PLANS AND DOCUMENTS
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DEVELOPMENT APPROVAL

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Water & Wastewater Precinct Network Plan ROL 1 Jimboomba Celestino

PREPARED FOR CELESTINO PTY LTD

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Executive Summary

Celestino Pty Ltd has engaged Calibre to prepare this Water and Wastewater PNP for the proposed Jimboomba Celestino development at Teviot Road, Jimboomba. The PNP was developed generally in accordance with Calibre Consulting *Water and Wastewater Infrastructure Master Plan report no. B13343.W-03 (C)*.

The primary objectives of the report are to:

1. Prepare a Precinct Network Plan for ROL 1 of the proposed Jimboomba Celestino Master Plan development that is generally in accordance with the previous strategies outlined within the *Calibre Consulting Water and Wastewater Infrastructure Master Plan report no. B13343.W-03 (C)*;
2. Assess capacity of the internal water network for ROL 1;
3. Assess capacity of the internal wastewater network up to RB1 SPS;
4. Develop a water and wastewater service strategy for the proposed development; and
5. Document the preferred water and wastewater service strategies.

For the proposed interim and ultimate servicing strategies refer to Appendix C.

Water network modelling was undertaken for two planning horizons, which are outlined below:

- Interim Water Supply Strategy servicing 1,750 EP of existing network capacity;
    - Proposed DN300 and DN225 connection to the existing water main within Teviot Road;
- Ultimate Water Supply Strategy servicing up to 20,468 EP of the ultimate development;
    - Proposed DN450 and DN525 water trunk main along Teviot Road connection to the future Flagstone Trunk Water infrastructure located to the west of the site;
    - Proposed internal Pressure reducing valve to service low elevations within the master plan site;
    - Proposed internal Water Booster Pump Station to service higher elevations within the master plan site; and
    - Proposed localised Water Booster Pump Station to service a higher elevation pocket within the northern portion of the master plan site.

Wastewater servicing strategy was prepared for two horizons:

- Interim Wastewater Strategy servicing 3,300 EP;
    - Proposed RB1 Wastewater Pump Station and associated rising main pumping to the Cedar Grove WWTP;
    - Proposed DN525 trunk wastewater main;
    - Internal gravity network as required.
- Ultimate Wastewater Strategy servicing 20,500 EP;
    - Upgrade of existing RB1 wastewater pump station and associated rising main;
    - Proposed RB3 wastewater pump station and associated rising main;
    - Proposed RB4 wastewater pump station and associated rising main;
    - Proposed DN525, DN450, DN375, DN300 and DN225 wastewater trunk mains as required;
    - Internal gravity network as required.

Water supply and wastewater network modelling has demonstrated that the proposed infrastructure has been sized adequately to service the Jimboomba Celestino development ROL 1 and all future stages. It is recommended that the strategies proposed within this Precinct Network Plan be adopted for ROL 1 and further refined for future applications.

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Glossary

AD	Average Day
ADWF	Average Dry Weather Flow
AHD	Australian Height Datum
C_2	Coefficient for Dry Weather Flow
DN	Diameter Nominal
Du	Dwelling Units
DSS	Desired Standards of Service
EDQ	Economic Development Queensland
EP	Equivalent Persons
FF	Fire Flow
LCC	Logan City Council
LWIA	Logan Water Infrastructure Alliance
MDMM	Mean Day Max Month
NRW	Non-Revenue Water
PD	Peak Day
PDA	Priority Development Area
PDWF	Peak Dry Weather Flow
PH	Peak Hour
PRV	Pressure Reducing Valve
PWWF	Peak Wet Weather Flow
RIGS	Reduced Inflow Gravity Sewer
RL	Relative Level
SEQ Code	South East Queensland Water Supply and Sewerage Design & Construction Code
SPS	Sewage Pump Station
SRWP	Southern Regional Water Pipeline
ULDA	Urban Land Development Authority
WWTP	Wastewater Treatment Plant

1 Introduction

Celestino Pty Ltd has engaged Calibre Professional Services Pty Ltd to prepare this Water and Wastewater Precinct Network Plan (PNP) in support of the proposed Jimboomba Celestino residential development at Teviot Road, Jimboomba. The proposed development forms part of the Flagstone Priority Development Area (PDA) located within Logan City Council (LCC) local government area. The proposed development layout is outlined on Veris drawings *30697-GP01 Sheet 1 and 2* attached within Appendix A.

In March 2017, Calibre prepared the Water and Wastewater Infrastructure Master Plan (IMP) (*B13343.W-03* Revision C) which identified water and wastewater service strategies proposed in support of the Jimboomba Celestino development. The Jimboomba Celestino PNP has been developed generally in accordance with the previous IMP network analyses with particular focus on the Reconfiguration of Lot (ROL) 1.

2 Site Characteristics

The proposed development is located at Teviot Road, Jimboomba within the LCC local government area. The ROL 1 development has a total area of 41.87 hectares within the master planning area of 560 hectares. The total proposed master plan area contains the real property descriptions of Lot 101, 102, 105 and 106 on SP254145 and Lot 800 on SP247625. The master plan area is bound by Teviot Road to the west, Logan River to the east and rural properties to the north and south.

The approximate development extents are outlined in Figure 2-1.

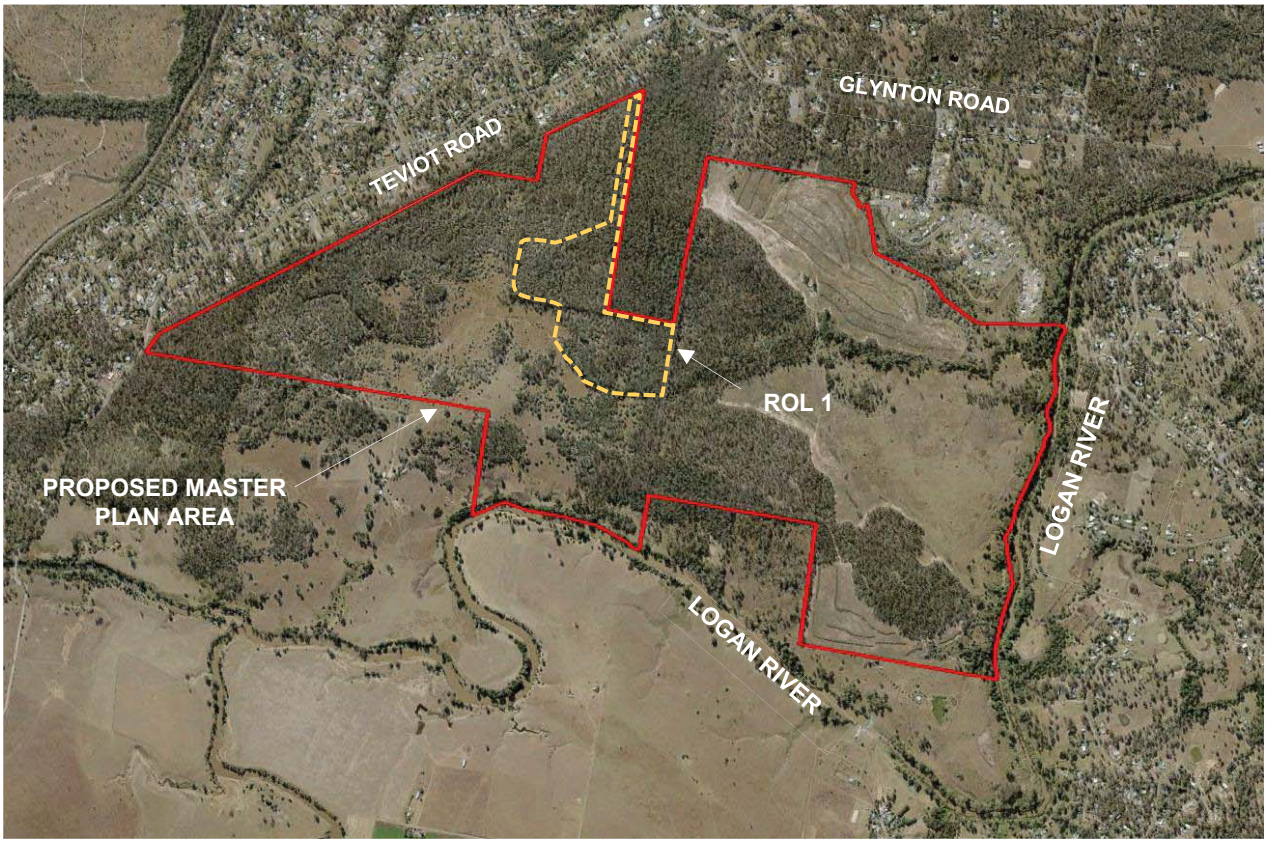


Figure 2-1: Site Location (Source: Google Earth)

The natural surface elevations of the site range from approximate 17 metres Australian Height Datum (mAHD) at the south western corner to 25 mAHD at the eastern boundary of the site adjacent Logan River. The site generally falls from east to west.

3 Assumptions

This section provides an overview of the key assumptions made in developing the Jimboomba Celestino PNP.

3.1 Previous Investigations

For the purpose of this assessment, the Calibre Consulting *Jimboomba Celestino Water and Wastewater Infrastructure Master Plan report no. B13343.W-03 (C)* has been adopted as the overarching Jimboomba Celestino service strategy. The network investigation and servicing strategies were endorsed with mark-ups on the 15th June 2017 by Economic Development Queensland (EDQ), Logan Water Infrastructure Alliance (LWIA) and Logan Water. The following sections outline the water and wastewater servicing strategies proposed within the master planning.

3.1.1 Interim Water Supply Service Strategy

It is proposed that the initial stages of the Jimboomba Celestino development will be serviced via connection to the existing DN375 trunk water main located at the intersection of Homestead Drive and Teviot Road. Network modelling has indicated that it is possible to service up to 1,750 EP from existing infrastructure. For the purpose of this PNP the interim development horizon will set with a maximum demand of 1,750 EP.

3.1.2 Ultimate Water Supply Servicing Strategy

It is proposed that beyond 1,750 EP, external upgrades will be required to service up to the ultimate population of 20,468 EP. Three catchment District Metered Zones (DMAs) are proposed as follows:

- Low Level Zone (LLZ): An extension of the LLZ downstream of the PRV proposed during the initial stages of the Jimboomba Celestino development. The LLZ is generally east of Bushland Road.
- High Level Zone 1 (HLZ1): An extension of the HLZ downstream of the booster pump station proposed during the initial stages of the Jimboomba Celestino development. The HLZ is generally west of Bushland Road.
- High Level Zone 2 (HLZ2): Construction of a new booster pump station at the northern corner of the development adjacent Bushland Road. This booster pump station will service the high elevation areas at the north-eastern portion of the site.

The proposed DMAs are shown on Calibre drawing no. 16-003350-403 attached within Appendix B. Internal infrastructure will be staged subject to the timing of individual development applications.

3.1.3 Interim Wastewater Servicing Strategy

As outlined in Calibre Consulting report no. B13343.W-03 (C), an internal gravity network conveying wastewater to the proposed RB1 Sewage Pumping Station (SPS) is proposed for the interim wastewater servicing strategy. The proposed SPS capacity for the interim horizon is 3,300 EP. The SPS is expected to pump towards the Cedar Grove Wastewater Treatment Plant (WWTP). Liaison with EDQ has indicated that the Cedar Grove WWTP will be operational by 2021.

3.1.4 Ultimate Wastewater Service Strategy

It is proposed that beyond 3,300 EP, the RB1 SPS will be upgraded and additional pump stations will be required to service up to 20,500 EP associated with the Jimboomba Celestino development and surrounding areas. Internal gravity sewer infrastructure will be constructed as required with reference to the ultimate proposed trunk mains proposed within the IMP.

3.2 Information Sources

The following information was provided by third parties in support of the water and wastewater assessment undertaken for the proposed development and are referenced in this report:

- Jimboomba Celestino Water & Wastewater Infrastructure Master Plan report no. B13343.W-03 (Calibre Consulting, March 2017) – EDQ Approved 15 June 2017
- Greater Flagstone PDA Wastewater Servicing Strategy (Cardno, 2 November 2016)
- SRWP Water Supply Zone Master Planning Figures 1, 2, 10 and 13 (LWIA, November 2014)
- H2OMap Water Supply Network Model entitled Logan_Comb11.H2O (LCC, 25 May 2016)

For the purpose of this PNP, it is assumed that the information provided above is the most accurate and appropriate at the time of reporting.

3.3 Desired Standard of Service

The desired standards of service (DSS) have been adopted in accordance with the LWIA Master Planning Review of Desired Standards of Service (Ref 90-12-43 Rev 2) dated April 2014 attached within Appendix B. Amendments to the DSS have been made in accordance with the previously approved IMP report dated 15 June 2017. These amendments include adopting 1000 L/EP/Day and 75% depth at PWWF for wastewater.

3.3.1 Water DSS

- Average Day Demand (AD): 165 L/EP/Day;
- Non-Revenue Water (NRW): 25 L/EP/Day;
- Residential Fire Flow: 15 L/s for 2 hours;
- Commercial Service Pressure (Peak Hour Conditions): 22 m;
- Minimum Residual Pressure (Fire Flow Conditions): 12 m; and
- Desired Maximum Service Pressure: 55 m.

3.3.2 Wastewater DSS

- Average Dry Weather Flow (ADWF): 165 L/EP/Day
- Peak Wet Weather Flow (PWWF): 1000 L/EP/Day
- Pipe Roughness ‘k’ value: 1.5mm; and
- Maximum depth of flow in new pipes: 75% depth at PWWF

3.4 Development Projections

Stage 1 development demands have been calculated in accordance with the Veris ROL 1 layout plan *30697-GP01 Sheet 1 and 2* included within Appendix A. Ultimate development projections for the proposed development have been estimated in accordance with the Veris proposed *Master Plan Layout* drawing no. *30697-GP08* included within Appendix A. Equivalent Persons (EP) were estimated using the density and areas proposed within this plan. The water and wastewater demands calculated are outlined in *Table 3-1*

Table 3-1: Jimboomba Celestino Demand Projections

Stage	Land use	Quantity	Units	Water		Wastewater	
				EP/Unit	Total EP	EP/Unit	Total EP
Stage 1	Residential Dwellings	581	Dw	2.8	1,626.8	2.8	1,626.8
Stage 2	Residential Dwellings	1,004.9	Dw	2.8	2,813.7	2.8	2,813.7
Stage 3	Residential Dwellings	395.5	Dw	2.8	1,107.4	2.8	1,107.4
Stage 4	Neighbourhood Centre	68.0	GFA m^2	0.7523	51.2	0.8219	55.9
	Residential Dwellings	300.9	Dw	1.78	535.6	1.78	535.6
	Residential Dwellings	1,198.0	Dw	2.8	3,354.4	2.8	3,354.4
Stage 5	Residential Dwellings	865.5	Dw	2.8	2,423.4	2.8	2,423.4
	Special Use	64.8	GFA m^2	0.7523	48.7	0.8219	53.3
Stage 6	Residential Dwellings	715.1	Dw	2.8	2,002.3	2.8	2,002.3
Stage 7	Residential Dwellings	768.5	Dw	1.78	1,368.0	1.78	1,368.0
Stage 8	Residential Dwellings	759.1	Dw	2.8	2,125.5	2.8	2,125.5
Stage 9	Residential Dwellings	333.1	Dw	2.8	932.7	2.8	932.7
Stage 10	District Centre	175.2	GFA m^2	0.7523	131.8	0.8219	144.0
	School	141	GFA m^2	0.6076	85.7	0.6638	93.6
Total				**18,607**		**18,637**	
Total inc. 10% Contingency				**20,468**		**20,500**	

The Interim Horizon is proposed to comprise primarily of Stage 1, with the Ultimate Horizon consisting of Stages 2 to 10. The total population for Stage 1 is 1,789.5 EP (including contingency) for water supply and wastewater. At the Ultimate Horizon, the total EP demand is 20,468 for water and 20,500 EP for wastewater.

4 Water Supply Servicing

The Jimboomba Celestino water and wastewater PNP has been developed generally in accordance with the servicing strategies identified within the Calibre Consulting IMP report no. B13343.W-03 and outlined within Section 3.1. A water supply network assessment was undertaken using InfoWater to assess the infrastructure and staging required to service the Jimboomba Celestino development. Assessment was undertaken for the ultimate population of 20,468 EP.

All proposed water service strategies are conceptual only and remain subject to detailed engineering design; however, the strategies are generally considered final.

4.1 Existing Water Supply Network Infrastructure

There is an existing DN375 trunk water main to the north of the site at the intersection of Homestead Drive and Teviot Road, supplied via Round Mountain Reservoir, approximately 6.9 km north-west. There is also an existing DN225 water main located immediately north-east of the site in Riverbed Boulevard.

4.2 Methodology

Assessment of the proposed water supply network was undertaken to confirm that the proposed development could be sufficiently serviced across all planning horizons. The following provides a description of the methodology adopted in developing the precinct network plan:

- Review the service strategies proposed within Calibre Consulting report no. B13343.W-03, dated March 2017;
- Investigate staging of Jimboomba Celestino water supply infrastructure;
- Undertake network modelling using InfoWater software; and
- Determine conceptual design parameters and planning considerations for proposed infrastructure.

Staging of infrastructure has been assumed to occur in accordance with the Veris *Master Plan Revised Stage 1 Layout* drawing no. *30697-GP1004* included within Appendix A. Two critical planning horizons have been identified for the purpose of this PNP:

- Interim Horizon – 1,750 EP (Stage 1); and
- Ultimate Horizon – 20,468 EP (Stage 2 – 10).

4.3 Water Supply Flow Projection

The design water supply demands were calculated based on the population projections in *Table 3-1* and the DSS outline in Section 3.3.1. These demands are presented in *Table 4-1*.

Table 4-1: Jimboomba Celestino Water Supply Demand Projections

Planning Horizon	Land Use	Demand	AD	NRW	MDMM	PD	PH	FF
Interim	Residential	1,750 EP	3.8 L/s	0.5 L/s	4.9 L/s	6.2 L/s	10.9 L/s	15 L/s
Ultimate	Residential	20,119 EP	44.2 L/s	5.8 L/s	55.8 L/s	71.1 L/s	124.9 L/s	15 L/s
	Commercial	349 EP	0.8 L/s	0.1 L/s	0.9 L/s	1.0 L/s	1.4 L/s	30 L/s
	Total	20,468 EP	45.0 L/s	5.9 L/s	56.7 L/s	72.1 L/s	126.4 L/s	30 L/s

Note: 30 L/s fire flow triggered by commercial land uses within the site.

The design water demands were estimated using an Average Day demand of 165 L/EP/day and a non-revenue water allowance of 15 L/EP/day. Peaking factors were not applied to the non-revenue water allowance. The Interim Jimboomba Celestino development demand of 1,750 EP has an estimated Peak Hour (PH) flow of 10.9 L/s. The interim horizon will require a minimum Fire Flow (FF) capacity of 15 L/s for 2 hours.

The ultimate development will have 20,468 EP with a PH flow of 126.4 L/s. The ultimate horizon will require a FF capacity of 30 L/s for 4 hour to cater for commercial land use.

4.4 Interim Water Supply Service Strategy

It is proposed that the interim horizon be serviced via connection to the DN375 located at the intersection of Homestead Drive and Teviot Road. It is proposed that a DN300 be extended along Teviot Road to the boundary of the site with a DN225 connection along the proposed road to service the interim horizon. The proposed interim water supply service strategy is outlined within Calibre drawing no. *16-003350-401* included within Appendix C.

Initially, the development is proposed to be serviced from the existing DMA via an internal reticulation network consisting of DN225, DN150 and DN100 water mains. This network is proposed to be part of the proposed HLZ1. Due to maximum pressure constraints on the existing external network, a maximum demand of approximately 1,750 EP can be serviced under the interim horizon.

4.5 Ultimate Water Supply Service Strategy

The ultimate water supply infrastructure is triggered once demands approach and exceed 1,750 EP. The ultimate development servicing strategy is outlined in Calibre drawing no. *16-003350-402* included within Appendix C.

To improve service pressures for the ultimate development, the DN300 main is proposed to be extended to the Harris Road and Teviot Road intersection. From there, a DN450 main will extend towards the Gittens Road and Teviot Road intersection. A DN525 then extends towards Wyatt road where it is proposed to provide a connection to future ultimate Flagstone PDA trunk water infrastructure. The proposed future ultimate Flagstone PDA trunk water infrastructure is outlined on Calibre drawing no. *16-003350-404* included within Appendix C. When this infrastructure has been established, connections provided for the interim service strategy will be shut as DMA boundary valves.

To maintain levels of service across the three DMAs, two booster pump stations and a single pressure reducing valve (PRV) are proposed. The proposed booster pump station for the HLZ1 is located adjacent to Harris Road. The booster pump station for HLZ2 is located towards the north east corner of the site. The proposed PRV is located adjacent to Harris Road and draws from the proposed DN450 main along Harris Road. The three proposed DMAs are outlined in Calibre drawing no. *16-003350-403 included within Appendix C.*

It is proposed that the site be serviced via an internal reticulation network comprising of DN225, DN150 and DN100 water mains. For security of supply purposes, it is proposed that additional connections with normally closed boundary valves will be made to the existing trunk water mains located at the Homestead Drive-Teviot Road intersection and Riverbend Boulevard.

4.6 Proposed District Meter Area Demands

The ultimate development is proposed to be serviced via a new trunk main connected to future ultimate Flagstone PDA trunk water infrastructure as outlined in Section 4.5. The total EPs associated within each DMA are outlined in *Table 4-2*.

Table 4-2: DMA EP Breakdown

DMA	Land Use	EP
HLZ 1	Residential	9,352
	District Centre	145
	School	94
	Total HLZ 1	*9,352*
HLZ 2	Residential	268
	Total HLZ 2	*268*
LLZ	Residential	10,523
	Neighbourhood Centre	56
	Special Use	54
	Total LLZ	*10,633*
Total		20,468

4.7 External Water Supply Network Performance

An external water supply network assessment was undertaken as part of the IMP. The external network analysis did not take into account commercial firefighting requirements. A summary of the external network analysis results are outlined in *Table 4-3*.

Table 4-3: External Network Performance – Interim Development Horizon

Additional Demand	Peak Hour		Fire Flow	
	Minimum PH Pressure	Number of Nodes Failing DSS	Minimum FF Pressure @ LS_J7467	Number of Nodes Failing DSS
0 EP	19.96 m	4	14.88 m	82
500 EP	19.76 m	4	14.61 m	82
1000 EP	19.50 m	4	14.45 m	82
1500 EP	19.21 m	4	14.22 m	82
1,750 EP	19.06 m	4	14.14 m	82
2,000 EP	18.90 m	5	13.99 m	82

Through network analysis, pre-existing network failures were identified within the external network as outlined within Table 4 3. It was determined that a maximum demand of 1,750 EP could be serviced without negatively impacting external levels of service.

Once demands exceed 1,750 EP the ultimate trunk infrastructure will be required to maintain levels of service both internal and external to the development.

Network modelling has confirmed that elevation up to 60m can be serviced with the proposed interim water supply strategy, as pressure constraints are associated with existing external network rather than internal development elevations.

4.8 Internal Network Performance

The internal network modelling results have been adopted from the IMP, as conservative EP demands were adopted for the site and potential surrounding development. Outcomes of water network modelling for the total development are outlined within *Table 4-4*.

Table 4-4: Internal Network Performance

Planning Horizon	Minimum PH Pressure	Minimum FF Pressure	Maximum Pressure
Interim	30.1 m	26.8 m	37.3 m
Ultimate	23.9 m	19.4 m	64.6 m

Note: Pressures shown at worst-case node within ultimate internal network.

Pressures under the interim and ultimate horizons indicate that the proposed water supply network has sufficient capacity to service the demands.

High service pressures up to 65m were identified within the IMP. Liaison with EDQ indicated that pressures up to 65m were acceptable if it allows for an overall better network. As a result internal nodes exceeding the 55m target maximum pressure have not been classified as network failures.

The internal network has been conceptually designed to increase interconnectivity, circulation and subsequently minimise water age. Based on preliminary water age modelling, it is not considered that excessive water ages will be experienced. It is recommended that investigations be undertaken during detailed design and submitted to Council for assessment in conjunction with future Operational Works applications.

Based on the network modelling analyses, it is considered that the internal and external water supply network can service the proposed Jimboomba Celestino development generally in accordance with the IMP and EDQ Flagstone PDA Water Supply Servicing Strategy.

5 Wastewater Service Strategy

The Jimboomba Celestino PNP has been developed in accordance with the servicing strategies identified within the IMP as outlined within Section 3.1. Assessment of the internal wastewater network was undertaken using InfoSWMM software to ensure sufficient capacity is provided to service the development.

Onsite wastewater infrastructure has been conceptually designed in accordance with the SEQ Code and Logan DSS and remains subject to detailed engineering design during individual development applications. The following outlined the concept design and assessment undertaken for the internal wastewater network infrastructure proposed within the IMP.

All proposed wastewater service strategies are conceptual only and remain subject to detailed engineering design, however the strategies are generally considered final.

5.1 Existing Wastewater Network

There is an existing gravity main located approximately 1.6 km north-west of the site in Homestead Drive, which conveys wastewater to the existing SPS adjacent to Blueberry Ash Court. From here, flows are directed north to the existing Jimboomba WWTP. Liaison with LWIA and LCC has indicated that the existing wastewater infrastructure and Jimboomba WWTP do not have residual capacity to service future development loads.

As outlined in the Flagstone PDA Wastewater Servicing Strategy, it is proposed that a new Cedar Grove WWTP will be constructed to the south of the site. Liaison with EDQ has indicated that the Cedar Grove WWTP will be operational by 2021.

5.2 Methodology

Assessment of the Jimboomba Celestino wastewater network was undertaken to confirm that the proposed development can be serviced across all planning horizons. The following provides a description of the methodology adopted in development the PNP:

- Review the service strategies proposed within Calibre Consulting report no. B13343.W-03, dated March 2017;
- Investigate staging of Jimboomba Celestino wastewater infrastructure;
- Undertake network modelling using InfoSWMM software; and
- Determine conceptual design parameters and planning considerations for proposed infrastructure.

Staging of infrastructure has been assumed to occur in accordance with the Veris *Master Plan Revised Stage 1 Layout* drawing no. *30697-GP1004* included within Appendix A. Two critical planning horizons have been identified for the purpose of this PNP:

- Interim Horizon – 1,750 EP (Stage 1); and
- Ultimate Horizon – 20,500 EP (Stages 2 – 10).

5.3 Wastewater Load Projection

The design wastewater loads were calculated based on the population projections in *Table 3-1* and the DSS outline in Section 3.3.2. The design wastewater loads are outlined in *Table 5-1*.

Table 5-1: Wastewater Load Projection

Planning Horizon	Demand	ADWF	C_2	PDFW	PWWF
Interim	1,750 EP	4.1 L/s	2.1	8.7 L/s	20.3 L/s
Ultimate	20,500 EP	47.45 L/s	1.7	78.6 L/s	237.3 L/s

The design wastewater loads were estimated using the Reduced Inflow Gravity Sewer (RIGS) ADWF load of 200 L/EP/d and EDQ recommended PWWF of 1000L/EP/Day. An ultimate Peak Wet Weather Flow (PWWF) rate of 237.3 L/s is estimated.

5.4 Interim Wastewater Service Strategy

It is proposed that initial stages of the Jimboomba Celestino development will be serviced via an internal gravity network which conveys wastewater to the proposed RB1 SPS as outlined in Calibre drawing no. *16-003350-301* included within Appendix C. It is anticipated that RB1 would be a regional pump station as outlined in the IMP and LWIA drawing no. *LS-008-14* attached within Appendix D.

RB1 SPS will convey wastewater to the Cedar Grove WWTP. The exact point of discharge will need to be confirmed by EDQ and LWIA prior to finalising the interim strategy. Liaison with EDQ has indicated that the Cedar Grove WWTP will be operational by 2021.

Should delays occur in availability of the Cedar Grove WWTP, a temporary holding tank and pump out facility may be required to service the development. Liaison with EDQ has indicated that the Cedar Grove WWTP will be operational by 2021.

5.5 Ultimate Wastewater Service Strategy

Under the ultimate horizon, two additional pump stations will be required to service the development, with RB3 SPS located toward the northern end of Harris Road, and RB4 SPS located in the south east. Internal trunk mains will be provided to service the site under the ultimate horizon as outlined within Calibre drawing no. *16-003350-302* included within Appendix C. The ultimate wastewater service strategy remains generally in accordance with the IMP.

5.6 Wastewater Network Assessment

Assessment of the internal wastewater network was undertaken using InfoSWMM software to ensure sufficient capacity is provided to service the proposed Jimboomba Celestino development. All infrastructure proposed as part of this PNP has been conceptually designed in accordance with the SEQ Code and remains subject to detailed engineering design during individual development applications. The following outlines the concept design and assessment undertaken for the internal wastewater network infrastructure proposed for the development.

5.6.1 Conceptual Gravity Sewer Design

The proposed conceptual gravity sewer design has been adopted in accordance with the IMP with adjustments made as required to align with the updated layout plan. It is proposed that the Ultimate development will be serviced via a gravity catchment consisting of DN675, DN600, DN525, DN450, DN375, DN300, DN225 and DN150 gravity mains. Gravity mains have been sized to service the ultimate development and structure plan area as outlined within the IMP. The design specifications for the proposed internal gravity network are outlined in Table 5-2.

Table 5-2: Internal Gravity Network Capacity Summary

Pipe Diameter (mm)	Minimum Grade (m)	Pipe Capacity @ 75% (L/s)	Pipe Full Capacity (L/s)
DN150	1:180	10.5	11.5
DN225	1:300	24.0	26.3
DN300	1:400	44.8	49.1
DN375	1:550	69.2	75.9
DN450	1:700	99.8	109
DN525	1:750	178	195
DN600	1:900	189	207
DN675	1:1050	240	263

The internal wastewater network has been planned with sufficient capacity to convey PWWF without exceeding 75% of pipe capacity. The internal reticulation network proposed to service the ultimate development is outlined on drawing no. *16-003350-302* within Appendix C.

The concept grading has determined that the ultimate proposed trunk gravity mains can service the entire development extents. It is anticipated that the gravity sewer design will be optimised as part of the detailed design. Conceptual long sections for the proposed trunk gravity mains are outlined on drawings *16-003350-320* to *16-00350*-352 attached within Appendix E.

During the initial phases of the development, the trunk gravity network may not achieve self-cleansing velocities. It is anticipated that during these initial development phases, the trunk network will need to be monitored and a maintenance program implemented to reduce the accumulation of sediment within the pipes. Self-cleansing requirements and potential maintenance programs are to be considered further during detailed design.

5.6.2 Pump Station Planning

The proposed locations of the pump stations are outlined on Calibre drawing no. *16-003350-302* attached within Appendix C. The ultimate capacity for RB1 has been adopted in accordance with the IMP to account for external demands associated with surrounding developments. The conceptual SPS operation requirements are outlined in *Table 5-3*.

Table 5-3: Pump Station Conceptual Operational Requirements

Pump Station	Catchment (EP)	ADWF (L/s)	PWWF (L/s)	Operational Storage (m^3)	Emergency Storage (kL)[1]
RB1 (Interim)	1,790	4.1	20.7	0.3	62.7
RB1 (Ultimate)	20,500	47.5	237.3	3.6	717.5
RB3 (Ultimate)	3,538	8.3	54.0	0.6	125.7
RB4 (Ultimate)	3,102	7.2	20.7	0.3	108.6

Note: 1) Minimum emergency storage calculated for 4 hours ADWF.

The proposed pump stations have been sized to cater for the ultimate catchment demands. It is proposed that RB1 SPS be staged to reduce upfront costs while also improving network performance during initial stages. The regional SPS RB1 capacity has been assessed by LWIA within a Detailed Planning and Preliminary Design Report for RB1, RB2-2 and FL-1 (Ref. LS-014) dated September 2017, attached within Appendix F.

5.6.3 Proposed Rising Main Operation

The rising main alignment is subject to detailed design by LWIA. The proposed alignment for each rising main is outlined in Calibre drawing no. *16-003350-302* attached within Appendix C. For the purpose of this assessment, the rising main has been assumed to discharge to the future GB2 SPS as identified in the "Logan South Wastewater Servicing Strategy" report and within the IMP. The proposed rising main performance during the ultimate horizon is outlined in *Table 5-4*.

Table 5-4: Proposed Rising Main Operation

Rising Main Description	Diameter	Approx. Length	Velocity (m/s)	Retention Time (hrs)
RB1 (Interim)	OD200 (ID 163)	950 m	0.91	0.17
RB1 (Ultimate)	OD500 (ID 406.8)	950 m	1.28	0.14
RB3 (Ultimate)	OD225 (ID 183)	600 m	1.29	0.08
RB4 (Ultimate)	OD200 (ID 163)	600 m	1.44	0.10

Note: 1) Length is indicative only and subject to LWIA investigations regarding trunk wastewater infrastructure and WWTP for Flagstone PDA.

The proposed rising main infrastructure has been planned to meet the preferred maximum velocity requirement of 1.5m/s as outlined in the SEQ Code.

6 Conclusion

Celestino Pty Ltd has engaged Calibre to prepare this Water and Wastewater PNP for the proposed Jimboomba Celestino development at Teviot Road, Jimboomba. The PNP was developed generally in accordance with Calibre Consulting *Water and Wastewater Infrastructure Master Plan report no. B13343.W-03 (C)*.

The Water and Wastewater PNP has concluded the following:

- The ROL 1 estimated development yield is 1,789.5 EP for both water and wastewater including a 10% contingency;
- The ultimate development estimated development yield is 20,468 EP for water and 20,500 EP for wastewater ;
- Two horizons for water infrastructure have been adopted in accordance with the IMP;
    - The interim water strategy services up to 1,750 EP with a proposed DN300 and DN225 water main connection to the existing water main within Teviot Road as outlined in *Calibre drawing 16-03350-401*, attached within Appendix C;
    - The ultimate water strategy services up to 20,468 EP with a proposed trunk DN450 and DN525 water mains extending along Teviot Road connecting to the future Flagstone Trunk Water infrastructure as outlined in *Calibre drawing 16-003350-402*, attached within Appendix C;
- Two horizons for wastewater infrastructure has been adopted in accordance with the IMP;
    - The interim wastewater strategy services up to 3,300 EP with the proposed internal gravity network conveying to the RB1 SPS, which they pumps directly to the Cedar Grove WWTP as outlined in *Calibre drawing 16-0003350-301*, attached within Appendix C;
    - The ultimate wastewater strategy includes two additional pump stations, RB3 located in the northern portion of the site above ROL 1 and RB4 located in the south-east portion of the site. Internal trunk and reticulation mains will be provided as illustrated in *Calibre drawing 16-003350-302*, attached within Appendix C;

Water supply and wastewater network modelling has demonstrated that the proposed infrastructure has been sized adequately to service ROL 1 and the ultimate Jimboomba Celestino development.

7 Recommendations

Calibre recommends that this Water and Wastewater Precinct Network Plan for the Jimboomba Celestino residential development be approved on the basis that the site provides suitable water and wastewater network infrastructure.

Calibre recommends the following be undertaken in support of the proposed development:

- Adopt the water supply service strategies as outlined on drawings no. *16-003350-401* to *402*;
- Adopt the wastewater service strategies as outlined on drawings no. *16-003350-301* to *302*; and
- Undertake detailed design in accordance with the proposed water and wastewater service strategies.

8 References

- Cardno (2 November 2015), Greater Flagstone PDA Wastewater Servicing Strategy.
- Logan City Council (2016), H2OMap Water Model entitled Logan_Comb11.H2O.
- Logan City Council (2015), Logan Planning Scheme SC6.2.5 Planning Scheme Policy.
- Logan City Council (2015), LWIA Water Network DSS entitled 90-13-08.
- Logan City Council (2014), LWIA Wastewater Network DSS entitled 90-12-43.
- SEQ Water (2013), SEQ Code – South East Queensland Water Supply and Wastewater Design & Construction Code.
- Calibre Consulting (March 2017), Water and Wastewater Infrastructure Master Plan

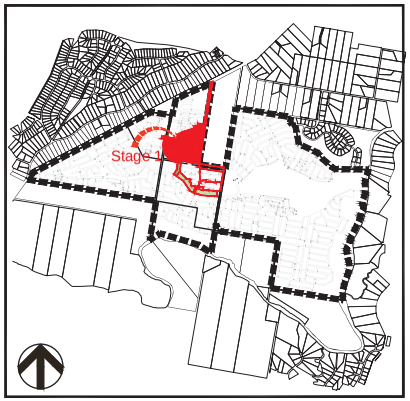
ROL 1 JIMBOOMBA CELESTINO

Appendix A Veris Layout Plan

CELESTINO PTY LTD

0 125m 50 mm 250m 100 mm 375m 150 mm SCALE BAR

CONTEXT PLAN (nts)



LEGEND	
Stage Boundary	
Preferred Garage Location	
On-street Car Park	

DEVELOPMENT SUMMARY	
Total Number Lots Under 12.5m	264
On street Car Parking Provision	0.75
Total Number of On Street Car Parks required	198
Shown - On street Car Parking	201

Teviot Road, Jimboomba

For

Riverside | CELESTINO

IMPORTANT NOTES:
(These notes are an integral part of this plan)

This plan was prepared to support a Reconfiguration of Lot application for the land situated at Jimboomba. The Contours, boundaries, creek line, flood lines, building locations have all been scaled and should not have any reliance based on them as they are only intended as a general guide. The configuration of allotments shown on this plan is subject to Local Authority (and relevant State Government Authority) approvals, detailed design, final survey, and registration of survey plans. Any comments contained on this plan should be confirmed by the relevant authorities. The dimensions, areas and total number of lots shown hereon are subject to field survey and also to the requirements of Council and any other authority which may have requirements under any relevant legislation. In particular, no relevance should be placed on the information on this plan for any financial dealings involving the land.

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Data Sources

Cadastral Boundaries	30697-BLE001-A
Contours / Topographic	DNR&M Lidar supplied data
Flood Level	Logan City Council shp file
Engineering Design	Calibre Consulting

Issue	Revisions	Date	Drawn
B	Add 2x mid-block pathways / orientate lots / add 10.0m lot / add LATM / new intersection / Bus Stop	14.11.2018	KS
A	Original	07.12.2017	KS/PS

Locality:	Jimboomba
Local Authority:	Logan City Council
Horizontal Meridian:	MGA
Vertical Level Datum:	AHD (Lidar)
Scale:	1:2,500@A3
Designed:	PS/KS
Drawn:	PS/KS
Checked:	PS/KS
Plot Date:	14 Nov, 2018
Computer File Ref:	30697-GP01-B.dwg

Car Parking Plan Stage 1

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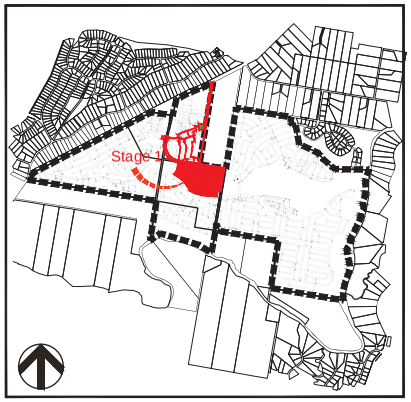
Drawing No 30697-GP01 (sheet 1 of 2)

Issue B

CONTEXT PLAN (nts)



LEGEND

Stage Boundary	
Preferred Garage Location	
On-street Car Park	

DEVELOPMENT SUMMARY

Total Number Lots Under 12.5m	264
On street Car Parking Provision	0.75
Total Number of On Street Car Parks required	198
Shown - On street Car Parking	201

Teviot Road, Jimboomba

For

Riverside | CELESTINO

IMPORTANT NOTES:
(These notes are an integral part of this plan)

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Locality:	Jimboomba
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Vertical Level Datum:	AHD (Lidar)
Scale:	1:2,500@A3
Designed:	PS/KS
Drawn:	PS/KS
Checked:	PS/KS
Plot Date:	14 Nov, 2018
Computer File Ref:	30697-GP01-B.dwg

Car Parking Plan Stage 1

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Drawing No: 30697-GP01 (sheet 2 of 2)
Issue: B

Teviot Road, Jimboomba

For

Jimboomba | CELESTINO

LEGEND

Site boundary	
Residential Density min.15dw/ha (up to 2 storey)	
Residential Density min. 25dw/ha (up to 5 storey)	
District Centre / Residential min. 25dw/ha (up to 5 storey)	
District Centre / Retail Core	
Active Edge - Active ground level uses such as: Retail Entertainment; Commercial; and Community Use.	
Neighbourhood Centre	
School	
Special use	
Park	
District sport park (Min. 7.5ha)	DS
Regional recreational park (Min. 10ha)	RR
Local Park	
Neighbourhood Park	

LEGEND

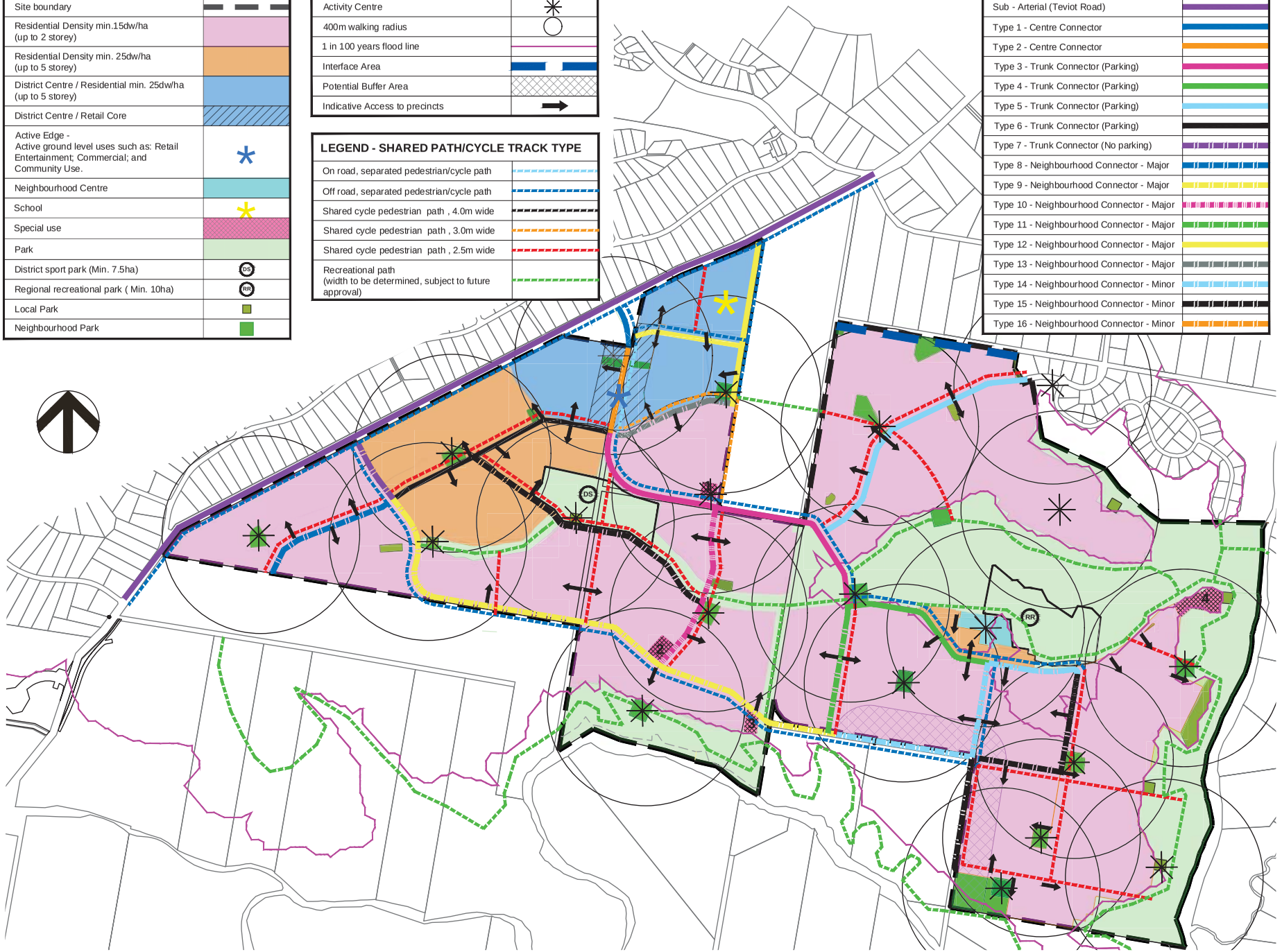
Activity Centre	
400m walking radius	
1 in 100 years flood line	
Interface Area	
Potential Buffer Area	
Indicative Access to precincts	

LEGEND - SHARED PATH/CYCLE TRACK TYPE

On road, separated pedestrian/cycle path	
Off road, separated pedestrian/cycle path	
Shared cycle pedestrian path , 4.0m wide	
Shared cycle pedestrian path , 3.0m wide	
Shared cycle pedestrian path , 2.5m wide	
Recreational path (width to be determined, subject to future approval)	

LEGEND - ROAD TYPES

Sub - Arterial (Teviot Road)	
Type 1 - Centre Connector	
Type 2 - Centre Connector	
Type 3 - Trunk Connector (Parking)	
Type 4 - Trunk Connector (Parking)	
Type 5 - Trunk Connector (Parking)	
Type 6 - Trunk Connector (Parking)	
Type 7 - Trunk Connector (No parking)	
Type 8 - Neighbourhood Connector - Major	
Type 9 - Neighbourhood Connector - Major	
Type 10 - Neighbourhood Connector - Major	
Type 11 - Neighbourhood Connector - Major	
Type 12 - Neighbourhood Connector - Major	
Type 13 - Neighbourhood Connector - Major	
Type 14 - Neighbourhood Connector - Minor	
Type 15 - Neighbourhood Connector - Minor	
Type 16 - Neighbourhood Connector - Minor	



0 750m 50 mm 1500m 100 mm 2250m 150 mm SCALE BAR

IMPORTANT NOTES:

This plan was prepared for Preliminary evaluation of the land situated at Riverbend & Teviot Brook. This plan is intended for discussion purposes only and should not be used for any other purposes. No investigations have been undertaken by Veris Australia Pty Ltd over this area of land other than to sight the registered plan and topographical map. The Contours, boundaries, creek line, flood lines, building locations have all been scaled and should not have any reliance based on them as they are only intended as a general guide. Any comments contained on this plan should be confirmed by the relevant authorities. The dimensions, areas and total number of lots shown hereon are subject to field survey and also to the requirements of Council and any other authority which may have requirements under any relevant legislation. In particular, no relevance should be placed on the information on this plan for any financial dealings involving the land.

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Data Sources

Cadastral Boundaries 30697-GP08-E

Issue	Revisions	Date	Drawn
F	Add road type 13	12/02/2019	PS/SI
E	Update road and cycle ways	7/02/2019	PS/SI
D	Original	8/09/2017	PS/KS/SI

Local Authority:	Logan City Council
Horizontal Meridian:	MGA
Vertical Level Datum:	AHD (Lidar)
Scale:	1: 15,000 @ A3
Drawn:	SI/KS
Checked:	SI/PS
Plot Date:	12 Feb, 2019
Computer File Ref:	30697-GP08-F.dwg

Master Plan
Lot 800 on SP247625, Lots 101, 102, 104, 105 & 106 on SP 254145

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Drawing No 30697-GP08

Issue F

ROL 1 JIMBOOMBA CELESTINO

Appendix B LWIA Desired Standard of Service

CELESTINO PTY LTD

Logan Water Alliance

LOGAN WATER ALLIANCE

MASTER PLANNING REVIEW

REVIEW OF DESIRED STANDARDS OF SERVICE

TASK NUMBER: 90-12-43

APRIL 2014

Approval Register

	Date
Project Manager Submitted	
Planning & Project Management Team Review	
Program Review	

Controlled Document – Change Register

Revision	Section Changed	Change Description	Initial	Date
A	All	Draft Report - Review	JT	02/04/2014
B	All	Draft Report - Review Comments	TP / JT	03/04/2014
C	All	Format of draft report	SS	03/04/2014
D	All	Incorporate comments	JT	10/04/2014
E	All	Format for Program Review	SS	14/04/2014
F	All	Program Review comments addressed	MS/JT	02/05/14
1	All	Final Report format	SS	02/05/2014
2	5	Emergency storage & Pump Design criteria clarified	JT/MP	25/6/2014

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APPENDICES

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Appendix B Existing Wastewater DSS
Appendix C Updated Water Supply Diurnal Patterns
Appendix D Recommended Water Desired Standards of Service
Appendix E Recommended Wastewater Desired Standards of Service

ABBREVIATIONS

AD	Average Day
ADWF	Average Dry Weather Flow
ARI	Average recurrence interval
C	Hazen Williams Coefficient
C1	Preferred Single Pump Rate Capacity Factor
CED	Common Effluent Discharge
CF	Constant Flow
DEWS	Department of Energy and Water Supply
DMA	district metered areas
DN	Diameter Nominal
DSS	desired standards of service
EP	Equivalent Person
ET	Equivalent Tenement
LCC	Logan City Council
LWA	Logan Water Alliance
MD	Max Day
MDMM	Mean Day Max Month
MH	Manhole
MOL	Minimum Operating Level
N	Number Pump Starts
OD	On Demand
PDWF	Peak Dry Weather Flow
PH	Peak Hour
PRV	Pressure Reducing Valve
PS	Pump Station
PWWF	Peak Wet Weather Flow
RIGS	Reduced Infiltration Gravity Sewers
SEQ Code	South East Queensland Water Supply and Sewerage Design and Construction Code
TWL	Top Water Level
V	Volume (operating)
VSD	Variable Speed Drive
WSAA	Water Service Association of Australia

EXECUTIVE SUMMARY

The Logan Water Alliance (LWA) is preparing to review the water and wastewater master planning previously undertaken and to facilitate this process a project plan has identified that a review of the Desired Standards of Service (DSS) is required.

Throughout the evolution of the planning process across Logan a number of DSS criteria have been challenged and where appropriate, a new criterion has been adopted. Contributing factors to situations that required a departure from strict adherence to the DSS include:

- prudency and efficiency measures adopted as part of the previous master planning exercise
- an increased understanding of Logan South demands and its variability and sensitivity to climatic conditions
- water quality challenges
- existing areas of network previously designed and constructed to lower standards

Key findings

- Water
    - **Average Day Consumption.** Detailed water demand analysis indicates that the current planning figure for residential demand of 260 L/EP/d (230 L/EP/d and 30 L/EP/d water loss) is overly conservative. A figure of 190 L/EP/d (165 L/EP/d and 25 L/EP/d water loss) provides for recently recorded demand levels and approximately 10% further rebound. This figure aligns with the current proposals put forward by Seqwater for regional planning.

      Constant Flow demand of 180 L/EP/d (130 L/EP/d and 50 L/EP/d water loss) is also considered to be too high based on the analysis of recent data which indicates average demand for these customers of 60 L/EP/d. Recognising that this demand component is extremely sensitive to climate conditions a factor of safety of 2 is considered prudent. A slightly higher water loss for these areas to account for the higher mains to property ratio typical for rural areas is also appropriate with a 150 L/EP/d (120 L/EP/d and 30 L/EP/d water loss) figure proposed.
    - **Diurnal Patterns.** A review of daily demand patterns indicate that the peak hour occurs in the evening and not the morning as the current pattern indicates. The new patterns are considered appropriate for existing and future planning.
    - **Peaking Factors.** A reduction to residential peaking factors has also been suggested as appropriate based on an analysis of recent data.
    - **Storage and Pumping.** The reduction in planning demands of 70 L/EP/d will impact on the sizing and timing of future infrastructure, particularly reservoirs and pump stations
    - **Roughness Calculations.** Hazen Williams' variance in headloss coefficient is considered unsuitable for accurate headloss prediction. Colebrook White provides a more accurate description of network conditions.

- Wastewater
    - **Average Dry Weather Flows**. Detailed analysis of dry weather wastewater flows by both treatment plant and pump station indicate the current average planning value of 200 L/EP/d is high with results suggesting a range of 158 to 163 L/EP/d. A figure of 165 L/EP/d is therefore considered prudent.
    - **Peak Dry Weather Flows**. An arbitrary factor (2*ADWF) is no longer considered suitable and the use of actual data or the WSAA formula is proposed.
    - **Wet Weather Containment**. The current approach to designing wastewater networks for wet weather flows is based on ensuring that the network can convey a theoretical PWWF. This approach is not appropriate for analysis of existing networks. In this situation, an evidence based containment standard that focuses on actual overflows rather than conveyance is considered prudent.

      A proposed containment standard of a 1 year ARI is recommended. Failure of this standard generally indicates severe wet weather problems that should be addressed. It is anticipated that this standard would only be applied to analysis of existing networks and if sufficient flow data has been collected to develop a calibrated model.

      Application of this standard will allow solutions to wet weather problems that may be more cost effective than conveyance capacity increase to be developed, e.g. I/I reduction and wet weather storage.
    - **Peak Wet Weather Flows.** A peak wet weather flow standard is still required for design of new networks and analysis of existing networks where insufficient flow data is available. An analysis of the existing PWWF standard of 1300 L/EP/day against actual flow data for LCC pump stations indicates that on average it caters a 1 year ARI in good soil and network conditions. This is reduced to a 6 month ARI in average network and soil conditions. Adopting a higher PWWF would impose unreasonable conveyance requirements upon LCC and could result in extensive and costly augmentations, when better value for money could be obtained through I/I remediation. On this basis, it is recommended that the current PWWF of 1300 L/EP/day be retained.

Recommendations

Following the investigations undertaken in this study it is recommended that LCC:

- adopt the proposed changes to the water and wastewater desired standard of service as outlined in Appendix D and E for the master planning review and subsequent planning activities
- forward adopted changes to the desired standard of service to the SEQ Code working group for inclusion in the next document revision
- continue to review the desired standard of service on a periodic basis and revise accordingly

1. INTRODUCTION

1.1 Objectives

The Logan Water Alliance is preparing to review the master planning previously undertaken as part of the priority infrastructure planning (PIP2) project in 2010-11. In order to facilitate the master planning process a project plan has been developed which identified the need to review the Desired Standards of Service (DSS) as part of this process.

Throughout the evolution of the planning process across Logan a number of DSS criteria have been challenged and where appropriate, a new criterion has been adopted. Contributing factors to situations that required a departure from strict adherence to the DSS include:

- prudency and efficiency measures adopted as part of the previous master planning exercise
- an increased understanding of Logan South demands and its variability and sensitivity to climatic conditions
- water quality challenges
- existing areas of network previously designed and constructed to lower standards

The recent release of the South East Queensland Water Supply and Sewerage Design and Construction Code (SEQ Code) now warrants a review of the DSS to ensure consistency and that no conflicts or ambiguity exists between the two documents.

This review will provide a single document that clearly outlines the standards to be applied. This will allow delineation between new and existing infrastructure as well as reticulation and trunk infrastructure. Work is also required to review the average and peak day demand for constant-flow and on-demand areas in Logan South region to better reflect actual usage. Clarity and guidance will also be provided on the options available for servicing elevated areas of the water network where either storage or pumping may be considered.

The development of guidance notes will be undertaken to ensure a prudent and efficient approach is adopted at all levels of planning. It is envisaged that these guidance notes will be used to provide clarity of what standards are to be applied throughout the master planning process which is scheduled to commence in early 2014.

1.2 Scope

The scope of this task is as follows:

- Review of water supply DSS criteria including:
    - Average Day (AD) demands:
        - analysis of water meter readings from both residential and commercial properties
        - pre and post drought data
        - on-demand and constant flow scenarios
    - Peaking factors including Mean Day Maximum Month (MDMM), Maximum Day (MD) & Peak Hour (PH)
    - Diurnal curves - including on-demand and constant flow scenarios
    - Dynamic analysis criteria for bulk supply and reticulation
    - Fire Fighting pressures and background demands for different land-use types
    - Reservoir storage volumes - including ground level reservoir and elevated reservoir
    - Pumping capacity for reservoir pumps and reticulation booster pumps
    - Pipeline design criteria including maximum velocities & friction coefficients
    - Water quality parameters
- Review of wastewater DSS criteria. Criteria to be reviewed may include:
    - Average Dry Weather Flows (ADWF)
    - Peak Wet Weather Flows (PWWF)
    - Reduced Infiltration Gravity Sewers (RIGS)
    - Discussion on containment standards
- Stakeholder workshop with LCC Planning, LCC Operations and LCC Asset Management to discuss and agree outcomes of the review
- Preparation of draft and final reports

1.3 Business Drivers

The business drivers for this study are:

- Growth
    - Suitable planning standards will ensure appropriate scope and timing for new infrastructure based on current trends and forecasts will also ensure a consistent DS over time
- Improvement
    - Clarification and guidance notes will ensure a prudent and efficient approach is adopted to planning at every level and ensure existing network capacity is utilised efficiently
- Compliance
    - Compliance with statutory legislation under the Integrated Planning Act 1997
    - Compatibility with the SEQ Code to ensure relevant criteria for both reticulation and trunk infrastructure

2. STAKEHOLDER ENGAGEMENT

A summary of the stakeholder engagement relating to or undertaken as part of this study is presented in Table 2-1.

Table 2-1: Stakeholder Engagement Summary

Function	Name	Position	Level of Consultation
Water Infrastructure Branch	Sandy Veeren	Water Infrastructure Planning Program Leader	• 80% Meeting
Water Infrastructure Branch	Marco Bonotto	Water Development Services Program Leader	• 80% Meeting
LWA / Water Infrastructure Branch	Sandy Stewart	Senior Water Infrastructure Engineer	• 80% Meeting
Water Business	Daryl Ross	Water Business Manager	• Overflow Abatement Workshop. Discussions on containment standards
Water Operations Branch	Palith Siriwardana	Water Operations Manager	• Overflow Abatement Workshop. Discussions on containment standards
Water Operations Branch	Darshan Udayaratna	Program Leader - Mechanical & Electrical Operations	• Overflow Abatement Workshop. Discussions on containment standards
Water Operations Branch	Roland Wicks	Water Grid and Operations Support Coordinator	• Overflow Abatement Workshop. Discussions on containment standards
Water Business	Darren Moore	Water Asset Management Program Leader	• Overflow Abatement Workshop. Discussions on containment standards
Water Business	Jackie Clutten	Water Cycle Planning Coordinator	• Overflow Abatement Workshop. Discussions on containment standards
Water Business	Chris Pipe-Martin	Water Quality Program Leader	• Overflow Abatement Workshop. Discussions on containment standards

3. PLANNING CONTEXT

3.1 Background

The last review of the demand based DSS was undertaken as part of the preparation of PIP2. The key DSS criteria used for the PIP2 master planning are summarised in Table 3-1. These parameters are defined in *Review of Desired Standards of Service (LWA 2010).*

Table 3-1: Summary of current key demand based DSS parameters

Parameter	Value
WATER:	
Average day demand (including system losses)	260 L/EP/d
System water losses	30 L/EP/d
Residential Detached Max Day / Average Day factor	1.7
Residential Detached Mean Day Max Month / Average Day	1.4
Residential Detached Peak Hour / Average Day	3.8
WASTEWATER:	
Average Dry Weather Flow	200 L/EP/d
Peak Dry Weather Flow (2xADWF)	400 L/EP/d
Peak Wet Weather Flow (Residential and Commercial)	1300 L/EP/d

3.2 Previous Studies

The following reports and documents have been considered in the DSS review:

- *Review of Desired Standards of Service (LWA 2010). Task 90-10-29.*
- *Reservoir Reliability Risk Assessment (LWA2013). Task 90-12-12.*
- *South East Queensland Water Supply and Sewerage Design and Construction Code (SEQ Code), 2013*
- *Sewerage Code of Australia WSA-02 (*2002), *Water Supply Code of Australia WSA-03, (2011)* (WSAA)
- *Queensland Fire and Rescue Service, Service Level Recommendations for Semi-Rural Areas (LWA 2010). Task 90-10-14.*
- *Overflow Abatement Strategy (LWA 2014). Task 90-12-50.*
- *Planning Guidelines for Water Supply and Sewerage (Department of Natural Resources and Mines, 2005)*

- *Draft Amendments to Planning Guidelines for Water Supply and Sewerage (Department of Energy and Water Supply 2013)*
- *Logan North Wet Weather Performance (LWA 2012). Task 90-11-94*
- *Logan East and South Wet Weather Performance Assessment (LWA 2013). Task 90-12-10.*
- *Leakage Modelling & SCADA Reporting 2012-13 (LWA 2013). Task 90-12-16*

4. WATER SUPPLY

4.1 Water Demand

Table 4-1 displays the demands found in the existing DSS as well as the proposed demands.

Table 4-1: Water Demand DSS Criteria

Water Demand	Existing	Proposed
Average Day demand	On demand areas - 260 L/EP/d. Based on 230 L/EP/d + allowance for leakage/losses (30 L/EP/d).	On demand areas - 190 L/EP/d. Based on 165 L/EP/d + allowance for leakage/losses (25 L/EP/d).
	Constant flow areas - 180 L/EP/d. Based on 130 L/EP/d + allowance for leakage/losses (50 L/EP/d).	Constant flow areas - 150 L/EP/d. Based on 120 L/EP/d + allowance for leakage/losses (30 L/EP/d).

4.1.1 On Demand

With the amalgamation of the Queensland Water Commission, Seqwater, Water Grid Manager and LinkWater into a single entity, the new Seqwater is now responsible for undertaking water demand analysis and forecasting for the region.

In an effort to develop a single baseline for which all water supply planning is undertaken, Seqwater recently presented the outcomes of their detailed demand analysis and demand forecasting for the region to the distribution entities. The assessment indicated the following consumption figures for the Logan region:

- Actual Residential Consumption (2013) – 122 litres per person per day
- Rebounded Residential Consumption (2018) – 135 litres per person per day

A review of the demands put forward by Seqwater was carried out by the LWA in order to assess whether the Seqwater demands were appropriate for planning use. Seqwater has forecast a rebound in water usage from 2018 of approximately 10%.

The review (LWA Task 90-12-22-63) reached the conclusion that some aspects, including the constant flow properties in Logan South, had not been considered and the following consumption figures are more appropriate for LCC planning:

- Actual Residential Consumption (2013) – 150 litres per equivalent person per day
- Rebounded Residential Consumption (2018) – 165 litres per equivalent person per day

The basis for the proposed 150 L/EP/d is that the analysis indicated daily residual consumption for typical metropolitan residential properties is between 144 – 153 L/EP/day under a series of analysis approaches. With the average metropolitan residential consumption being between 147 and 150 L/EP/day

Seqwater was amenable to the LCC's response and adjusted their figures to reflect more closely those of Council.

Seqwater uses a whole of grid approach to system losses and not L/EP/d. The latest analysis of water loss for Logan presented in *Leakage Modelling & SCADA Reporting 2012-13 (LWA 2013)* found that the average

water loss across the system was between 21 and 23 L/EP/d. A conservative figure of 25 L/EP/d is therefore recommended, resulting in a reduction of 5 L/EP/d from the existing DSS.

The Logan north historical consumption data for the last two years together with climate data is presented in Figure 4-1. Existing and proposed residential planning demands have been calculated for the same period and overlayed for comparison.

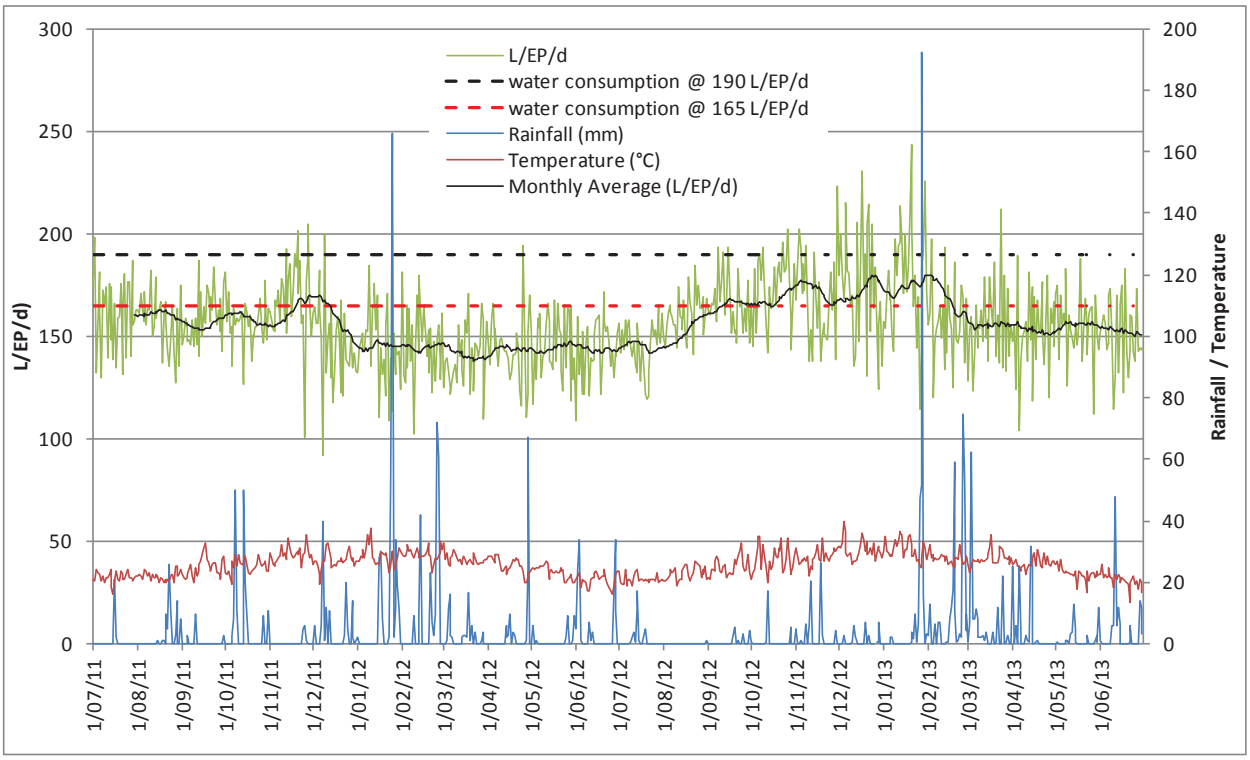


Figure 4-1: Logan North Historical Water Use

4.1.2 Constant Flow

The data for constant flow areas, predominately in Logan South, was not as reliable or as readily available as the data for on demand areas in Logan North and East. However, based on the data available a yearly average for constant flow properties of 60 L/EP/d was derived. As stated, this is a yearly average not taking into account dry periods or periods of high rainfall which significantly impact on demand in this area. To account for dry periods, LWA proposes a planning demand of 120 L/EP/d for constant flow areas. For losses in these networks a figure of 30 L/EP/d is recommended, slightly more than on-demand areas due to the significantly higher length of pipes per connection.

Water carrier tanker filling has a considerable impact on customer service levels in constant flow areas and this component should be adequately considered in all modelling and planning tasks.

It is recommended that as more information on customer demand trends in constant flow areas becomes available that the constant flow planning demands are reassessed.

4.2 Peaking Factors

Adopting the recommended 190 L/EP/d demand is a reduction of 70 L/EP/d resulting in a lower peak hour flow. In order to ensure the current DSS patterns and peaking factors represent current field results, an analysis of field data was undertaken.

4.2.1 Diurnal Patterns

An analysis of the two most recent summers (2012/13 and 2013/14) was undertaken in order to assess the current patterns against field results. Data was obtained from the SCADA network of flow meter records from the flow meters at the pressure reducing valves (PRV) at the inlets of the district metered areas (DMAs) in Logan North. The 2012/3 and 2013/4 summers flows and maximum day ratios were comparable.

Figure 4-2 displays a graphical representation of the diurnal flows for each day of the 2013/14 summer.

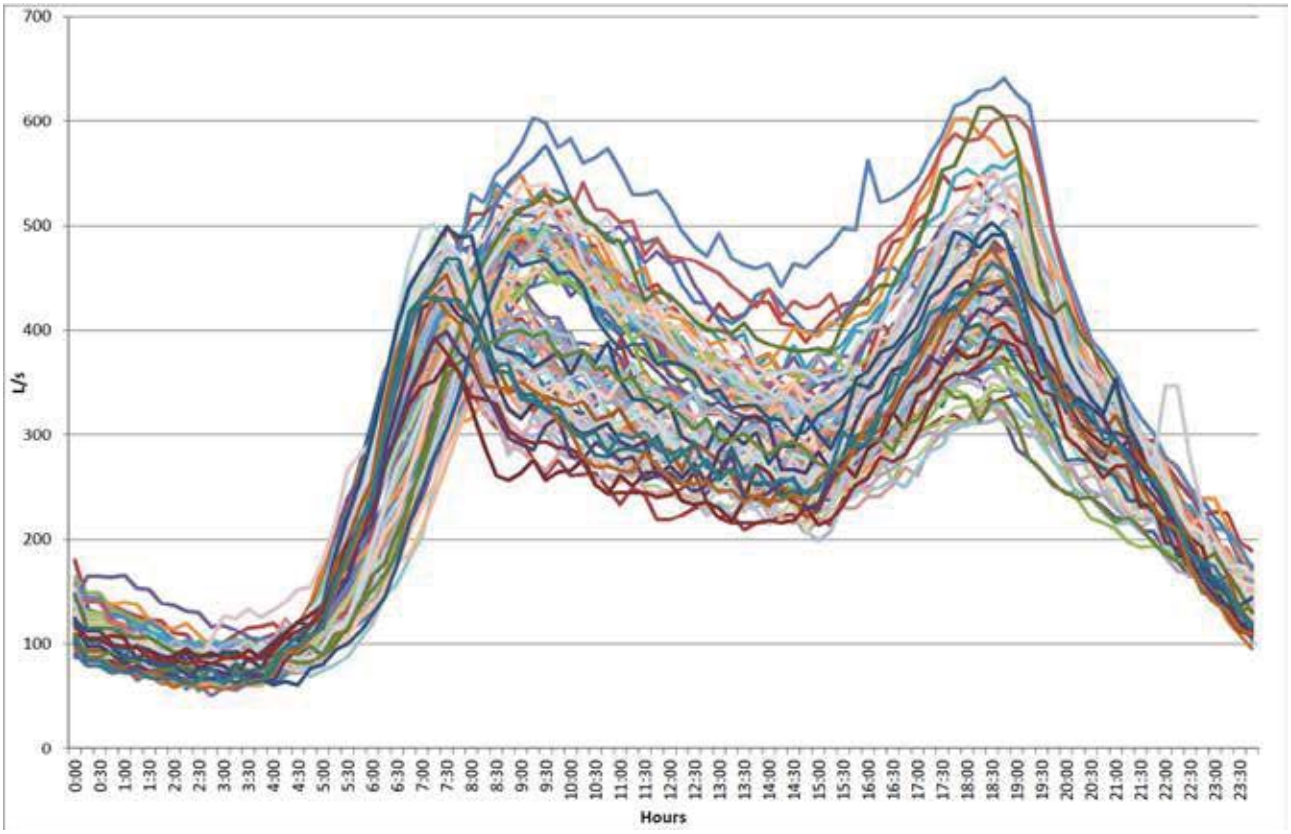


Figure 4-2: Logan North Diurnal Flows - Summer

In order to assess the current patterns against the field data the current DSS patterns were overlaid onto the field data. The current DSS patterns based on the existing demand of 260 L/EP/d.

Figure 4-3 displays the current DSS against the field data.

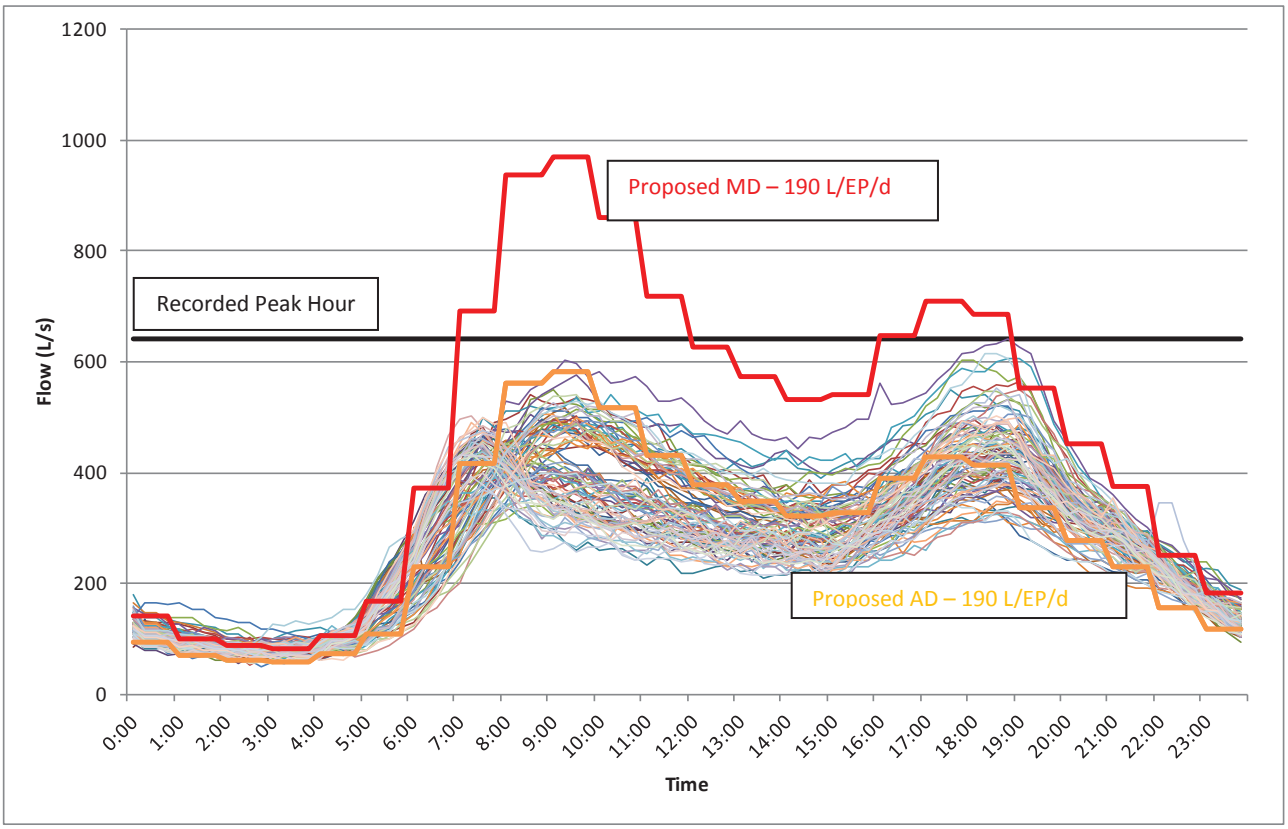


Figure 4-3: Current DSS against Logan North Diurnal Flows

As demonstrated in Figure 4-3, the change to a lower demand per EP alone would not align the current patterns to the field results as both

One of the major differences from the current pattern to the field results is the time of peak hour. The current pattern has a morning peak, while the field results indicate an evening peak. The second peak in the current pattern has approximately the same duration, where as the field results indicate that a morning peak, while not as high as the evening peak, has a longer duration. The evening peak is shorter in length, but has a higher instantaneous peak. The two peaks maximum demands are also closer than the current pattern indicates.

A new residential pattern was developed to account for the differences in the current pattern and the field results. As the Logan North network is made up of 85% residential properties and no flow data for individual commercial or industrial properties was available, only the residential patterns were adjusted to account for the difference between the current pattern and the field data.

Figure 4-3 below displays the current and new residential pattern. The new pattern has a lower peak hour in order to facilitate the two peaks aligning more closely.

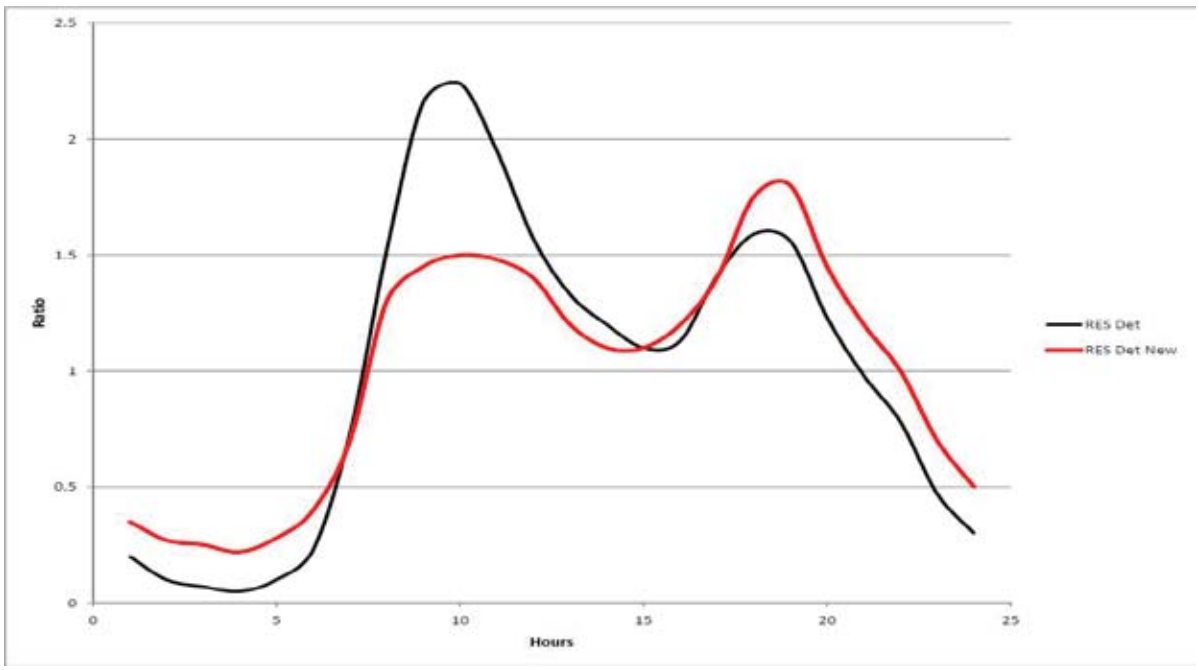


Figure 4-4: Current and new residential patterns

Figure 4-5 displays an average and a maximum day using the 190 L/EP/d with the new residential patterns.

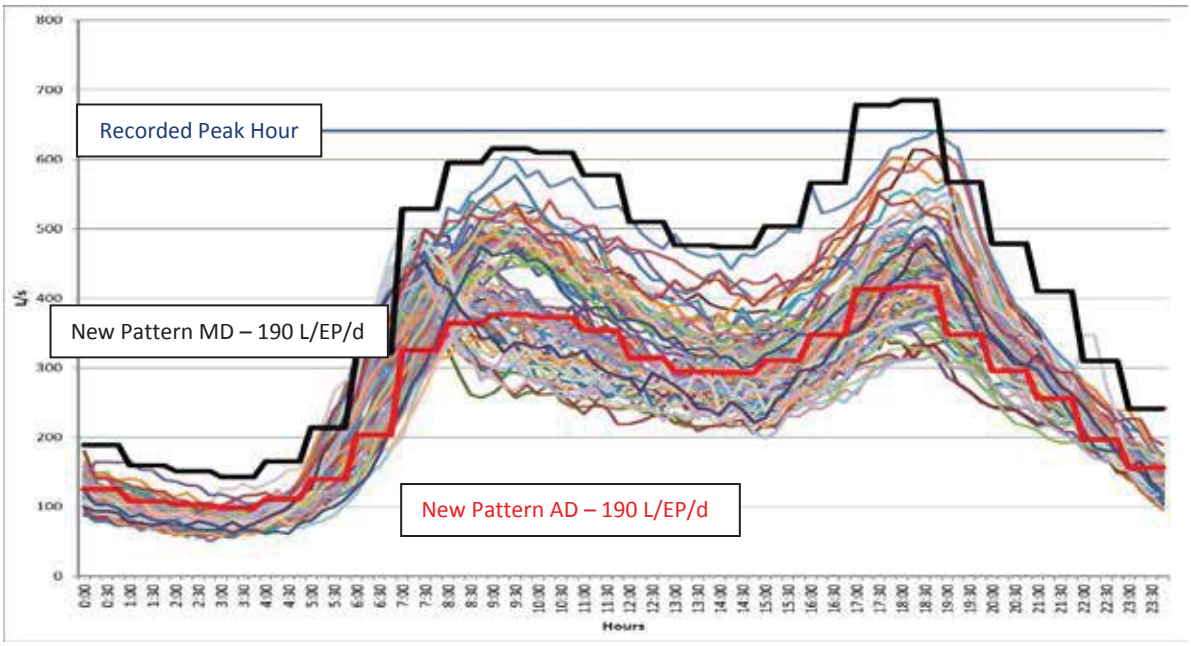


Figure 4-5: Current and new residential patterns

The recommended patterns can be found in Appendix C.

To inform future demand analysis and management it is recommended that a statistical sample of Logan's residential customer groups and all major customers are monitored. This would required monitoring or logging of existing suitable meters or replacement with smart meters.

4.2.2 Peaking Factors

For clarity and to avoid confusion with MH the abbreviation for manholes in the wastewater DSS, maximum hour (MH) will be replaced with Peak Hour (PH) throughout the water DSS.

The existing and proposed peaking factors are displayed in Table 4-2.

Table 4-2: Existing Peaking Factors

Peaking factors		MDMM/AD	MD/AD	PH/AD
Residential detached	Existing	1.4	1.7	3.8
	Proposed	**1.3**		**3.1**
Residential attached	Existing	1.3	1.6	3.1
	Proposed	1.3		**2.6**
Rural residential	Existing	1.4	1.7	CF services 70 L/hr/ET
	Proposed	**1.3**		
Commercial	Existing	1.2	1.3	2
	Proposed			
Industry	Existing	1.2	1.3	1.7
	Proposed			
Parks / Open Space	Existing	1.2	1.3	1.7
	Proposed			

Note: AD = Average Day, MD = Max. Day, MDMM = Mean Day Max Month and PH = Peak Hour.

The WSAA guidelines recommend a peaking factor of a least 1.5 for a maximum day and a peak hour factor of at least 2 for areas with a population over 10,000. Whilst the new pattern, designed to fall in line with actual recorded flows, lowers the peak hour when compared to the existing peak hour (3.8 to 3.1) the peak is still above the recommended ratio of 2.

The maximum day peaking factor of 1.7 aligned reasonably close to the recorded maximum day. No changes to the 1.7 maximum day peaking factor have been proposed.

The reduction in the residential detached MDMM factor aligns the total L/EP/d closer to the recorded MDMM flows.

4.3 System Planning

Table 4-3 displays the existing and proposed system planning DSS.

Table 4-3: System Planning DSS Criteria

System planning	Existing	Proposed
Bulk supply and reticulation	3 days of MDMM. Reservoirs to have a net positive inflow and capable of continuous operation and not fall below the emergency level. 3 days of MD. Reservoirs should not fall below the emergency level. 5 days of AD. Reservoirs should fill from empty to full	No changes proposed

No changes to the system planning DSS have been proposed. The reduction in average day demand will influence the outcome of the system planning DSS but there are no changes to the actual system planning criteria required.

4.4 Service Pressures

The proposed amendments to the water service pressure criteria are presented in Table 4-4.

Table 4-4: Service Pressure DSS Criteria

Service pressure	Existing	Proposed
Minimum operating pressure at PH	On demand areas – 22m at the property boundary based on reservoir at minimum operating level (MOL). MOL defined as 15% of storage height or top of emergency storage	No changes proposed
	Constant flow areas – 10m at the property boundary based on reservoir at minimum operating level (MOL) MOL defined as 15% of storage height or top of emergency storage.	
Maximum operating pressure	80 m at the boundary based on the reservoir at TWL	
Target pressure	55 m at the property boundary based on the reservoir level at TWL	

No review of the service pressures has been undertaken. The 22 m minimum is 2 m above the WSAA guidelines of 20 m, however, the higher 22 m was proposed and adopted during the formation of the SEQ Code and aligns with other councils in South East Queensland. The maximum pressure aligns with the WSAA guidelines therefore currents standards will remain.

4.5 Fire Fighting

As part of the water reform process Logan inherited large areas of rural network incorporating both constant flow and rural residential reticulation networks constructed under significantly different historic guidelines and standards. Due to the difficulties that exist in providing effective and practical fire flow provision to areas of sparse residential development, the application of Logan's urban fire fighting standard would prompt considerable infrastructure upgrades.

As part of the 2010 DSS review Council determined that existing constant flow areas that are not intended to change to an urban residential zone will no longer have a fire flow provided.

In addition to this provision and in line with WSAA, SEQ Code and the DEWS Planning Guidelines it is recommended that the DSS include a fire fighting provision for rural residential and commercial areas. All other criteria relating to fire fighting align with the relevant guidelines and codes.

The number of fires criteria is not considered necessary to the Logan network and an appropriate metric for sizing trunk mains. Furthermore there are no requirements under the *Planning Guidelines for Water Supply and Sewerage (DEWS, 2014)* to provide for simultaneous fires.

Table 4-5 displays the existing and proposed fire fighting DSS. It is recommended that the fire flow provisions adopted across the Logan network are mapped and the information provided to QFRS.

Table 4-5: Fire Fighting DSS Criteria

		Existing Criteria	Proposed Criteria
System pressure	On demand areas	12 m minimum at property boundary	No changes proposed
	Constant flow areas	no fire service provided	
Fire flow	Constant flow areas	no fire service provided	
	Rural Residential	-	7.5 L/s for 2 hrs
	Rural Commercial / Industrial	-	15 L/s for 2 hrs
	Residential	15 L/s for 2 hrs	No changes proposed
	Commercial & Industrial	30 L/s for 4 hrs	
	Special risk / hazard	to be advised by Logan City Council	
Background demand	2/3 PH (as per DEWS requirements)		
Reservoir level	MOL		
Number of fires	<5000 EP – single fire for residential or industrial / commercial >5000 EP – simultaneous single residential fire plus single commercial / industrial fire for sizing trunk mains		Remove

4.6 Reservoir Storage

The proposed amendments to the criteria for reservoir sizing are presented in Table 4-6.

Table 4-6: Reservoir Storage DSS Criteria

Reservoir storage		
	Existing	**Proposed**
Ground level storage capacity	3 (MD - MDMM) + (Greater of emergency storage/fire fighting storage) Emergency Storage = the greater of 4 hr MDMM demand in zone or 0.5 ML whichever is greater. Where Fire Fighting Reserve is: **> 4800EP:** • CF services for rural residential 72 kL (2 x 5 L/s x 2hrs); • for industry, business 540 kL (i.e. 30 L/s x 4hrs + 15 L/s x 2hrs) otherwise 210 kL (2 x 15L/s x 2hrs). **< 4800 EP:** • CF services 36 kL (15L/s x 2 hrs); for business & industry 432 kL (30L/s for 4 hrs); otherwise 150kL (= 15 L/s for > 2hrs).	3 (MD - MDMM) + (Greater of emergency storage/fire fighting reserve) Emergency Storage = 4 hr MDMM demand in zone or 0.5 ML whichever is greater. *Fire Fighting* reserve = ***> 4800EP:*** • rural residential 108 kL • industry 540 kL • otherwise 216 kL **< 4800 EP:** • rural residential 54 kL • industry 432 kL • otherwise 108 kL
Elevated storage capacity Guidance Note: In-line pressure booster pumping without high-level storage shall be considered where satisfactory pressures can be provided by gravity flow during low demand periods or booster failure can be mitigated to an acceptable level (risk assessment) and where there is no other feasible means of providing adequate pressure.	6 (MH - 1/12 MDMM) + minimum fire fighting reserve. Where fire fighting reserve is: > 4800EP: • CF services for rural residential 72 kL (2 x 5 L/s x 2hrs); • for industry, business 540 kL (i.e. 30 L/s x 4hrs + 15 L/s x 2hrs) • otherwise 210 kL (2 x 15L/s x 2hrs). < 4800 EP: • CF services 36 kL (15L/s x 2 hrs); • for business & industry 432 kL (30L/s for 4 hrs); • otherwise 150kL (= 15 L/s for > 2hrs).	6 (PH - 1/12 MDMM) + minimum fire fighting reserve. *Fire Fighting reserve* = ***> 4800EP:*** • rural residential 108 kL • industry 540 kL • otherwise 216 kL **< 4800 EP:** • rural residential 54 kL • industry 432 kL • otherwise 108 kL

4.6.1 Ground Reservoir Storage

As LCC will no longer allow new constant flow areas and there is no current provision for fire fighting in these areas it is recommended to remove these references from the water storage criteria.

The criteria presented are in line with those recommended in *Planning Guidelines for Water Supply and Sewerage (DEWS, 2014).* Fire fighting reserves have been determine by previous assessment and are considered appropriate.

Maximum hour references (MH) have been replaced with peak hour (PH) to avoid ambiguity with the manhole abbreviation used elsewhere in the DSS.

These criteria have been reviewed using the model and satisfy the minimum of 8-24 h consumption at peak day demand. Other than the minor changes already discussed and some grammatical corrections there are no further amendments recommended to these criteria.

4.6.2 Elevated Reservoir Storage

As addressed above for ground level storage, the deviation of peaking factors between residential and commercial has an impact on the range of supply. The WSAA guidelines recommend between 0.5 and 2 hours storage. The existing DSS allows for approximately 1.5 hours storage and adheres to the WSAA guidelines.

Additional advice is provided under the elevated reservoir criteria to ensure a risk based approach is considered when deciding between elevated storage and a boosted pump station solution.

4.7 Pumping Capacity

The proposed amendments to the pumping capacity criteria are presented in Table 4-7.

Table 4-7: Pumping Capacity DSS Criteria

Pumping capacity	Existing	Proposed
Pump supplying a ground level reservoir	MDMM over 20 hr	No change
Pump supplying an elevated reservoir	(6MH – operating volume) / (6 x 3600)	(6xPH – operating volume) / (6 x 3600)
Standby pump capacity	To match duty, except where more than one duty pump or as determined by risk assessment	No change
Reticulation booster pumps/pumped system	MH + fire flow	PH + fire flow

Excepting abbreviations, no changes are proposed for the pumping capacity DSS. All criteria adhere to the WSAA guidelines and the SEQ Code.

4.8 Pipeline Design

Previous planning exercises have identified that highly conservative pipeline roughness coefficients to allow for unknown losses and provided for the design of future networks have inadvertently been used as default values for existing infrastructure. This results in a significant reduction in existing system capacity particularly under fire flow conditions when hydraulic modelling is undertaken. Under these circumstances significant network augmentations can be proposed to meet DSS as actual capacity has been ignored.

To ensure this situation does not occur in future it is proposed to revise the DSS tables to include a set of roughness values based on material and age for existing infrastructure and refer to WSAA guidelines (WSA 03-2011 3.1.6.2.) on headloss limits for efficient design of new pipe assets.

Also due to the difficulty of manually solving the Colebrook-White equation to find f (friction factor), the use of the empirical ‘Hazen-Williams’ formulae for flow of water at 60º F (15.5º C) has been used for many years. To use the Hazen-Williams formula a head loss coefficient must be used. Unfortunately the value of the head loss coefficient is empirical and can vary from around 80 to 130 and beyond, and this can make the ‘Hazen-Williams’ formulae unsuitable for accurate prediction of head loss.

With the advent of computer based modelling the Colebrook White equation is now more easily solved and more accurate than the Hazen Williams equation under a series of flow conditions. A change of the friction value from the Hazen Williams to the Colebrook White formula is therefore proposed. The existing and proposed pipeline design criteria are presented in Table 4-8.

Table 4-8: Pipeline Design DSS Criteria

<table>
<tr><th>Pipeline design</th><th>Existing</th><th>Proposed</th></tr>
<tr><td>Main capacity</td><td>Raw water mains & trunk feeding ground level reservoir : MDMM for a gravity supply and MDMM over 20hrs for a pumped supply
Trunk Mains feeding elevated reservoir: capacity of pumps
Reticulation Mains: MH + fire flow</td><td>Raw water mains & trunk feeding ground level reservoir : MDMM for a gravity supply and MDMM over 20hrs for a pumped supply
Trunk Mains feeding elevated reservoir: capacity of pumps
Reticulation Mains: PH + fire flow</td></tr>
<tr><td rowspan="2">Mains size</td><td>Constant flow areas - where mains terminate in short runs such as cul-de-sacs minimum sizes that may be acceptable are:
DN 63 up to 150 m from main
DN 80 up to 350 m from main</td><td>No change</td></tr>
<tr><td>On demand areas - minimum reticulation main 100mm in residential areas and 150mm in commercial and industrial areas</td><td>No change</td></tr>
<tr><td rowspan="2">Friction default value</td><td>Existing</td><td>Proposed</td></tr>
<tr><td>Hazen Williams C values

Distribution:
<300mm, C=110
300mm – 600mm, C=120
>600mm, C=125

Reticulation:
<= 150mm, C=100
200mm - 300mm, C=110
>300mm, C=120</td><td>Assessment of existing infrastructure
Colebrook White k values (mm)

<table><tr><th rowspan="2">Pipe Material</th><th colspan="3">Pipeline Age (years)</th></tr><tr><th>< 10</th><th>10 to 25</th><th>> 25</th></tr><tr><td>Asbestos Cement</td><td>0.15</td><td>0.3</td><td>0.3</td></tr><tr><td>Plastic (UPVC, MDPE, Hobas, etc)</td><td>0.06</td><td>0.06</td><td>0.15</td></tr><tr><td>MSCL/DICL</td><td>0.3</td><td>0.3</td><td>0.6</td></tr><tr><td>CICL</td><td>0.3</td><td>0.3</td><td>0.6</td></tr></table>
Assessment/design of future infrastructure
Refer Clause 3.1.6.2 of WSA03_2011</td></tr>
<tr><td>Maximum velocity</td><td>2.5 m/s</td><td>2.5 m/s
At peak hour for trunk mains and at peak hour including fire flow for reticulation</td></tr>
</table>

4.9 Water Quality

The proposed amendments to the drinking water quality criteria are presented in Table 4-9.

Table 4-9: Water Quality DSS Criteria

Water quality	Existing	Proposed
Drinking water quality	Drinking water to comply with the NHMRC Australian Drinking Water Guidelines – 2004.	No Changes proposed

Logan City Council has adopted the NHMRC Australian Drinking Water Guidelines 2004 as the applicable standard for drinking water quality. This standard was adopted as part of the previous DSS review. No change is proposed.

4.10 Summary of Proposed DSS for Water

The proposed changes to the Desired Standards of Service for Wastewater planning are outlined in Appendix D.

5. WASTEWATER DSS REVIEW

5.1 Dry Weather Flow

The proposed amendments to the dry weather flow criteria are presented in Table 5-1.

Table 5-1: Dry Weather Flow DSS Criteria

Sewage load	Existing	Proposed
ADWF	200L/EP/day	165 L/EP/day
PDWF	2 x ADWF	Use actual flow data; or *d x 165* *L/EP/d or;* *d x 0.00191 x EP* *L/s* *Where:* $d = 0.01(\log A)^4 - 0.19(\log A)^3 + 1.4(\log A)^2 - 4.66\log A + 7.57$ *d = dry weather peaking factor* *A = gross plan area of development catchment, in hectares* *Refer to WSAA Sewerage Code of Australia; Part 1 Planning and Design*

5.1.1 Average Dry Weather Flow (ADWF)

The South East Queensland response to the millennium drought which occurred in the period 2005-2007 has resulted in a significant reduction in both average day demand for water and average dry weather flows in the sewer network. Prior to the drought, the DSS for ADWF exceeded 300 L/EP/day. The current DSS adopted an ADWF of 200 L/EP/day in response to the observed reduction in dry weather flow. This was a conservative figure at the time as the actual ADWF was around 140 L/EP/day. However, a degree of demand and sewer flow rebound was predicted once water restrictions were lifted and therefore a higher ADWF was specified for the DSS. This DSS revision provides the opportunity to assess whether this rebound has occurred.

Calculation of ADWF was undertaken for the Loganholme, Beenleigh, Jimboomba and Flagstone Wastewater Treatment Plants. In addition, ADWF flows were determined from SCADA data analysis at Logan North, Logan East and Logan South pump stations. The results of these calculations for the 2012/13 financial year are summarised in Table 5-2 and Figure 5-1.

Table 5-2: ADWF Analysis by WWTP

	Loganholme WWTP	Jimboomba WWTP	Flagstone WWTP	Beenleigh WWTP	Combined WWTP Loads
ADWF 2013 (ML/d)	36.360	0.212	0.292	8.195	45.059
Estimated Residential EP 2013 (EP)	187,675	740	1,806	47,173	237,394
Estimated Non-Residential EP 2013 (EP)	35,900	568	41	4,053	40,562
Total EP 2013 (EP)	223,575	1,308	1,847	51,226	277,956
Estimated ADWF (L/EP/d)	163	162	158	160	162

Table 5-2 shows that the ADWF across four of LCC's treatment plants is 162 L/EP/day. This is higher than the 140 L/EP/day identified during the previous DSS review indicating that a degree of demand rebound has occurred. As restrictions have now been lifted for more than two years it can be assumed that there will be no further rebound in the sewer ADWF. There may be further rebound in the water demand, but this is likely to be for outdoor consumption and will not impact on sewer flows. On this basis, it is recommended that 165 L/EP/day be adopted as the new DSS for ADWF.

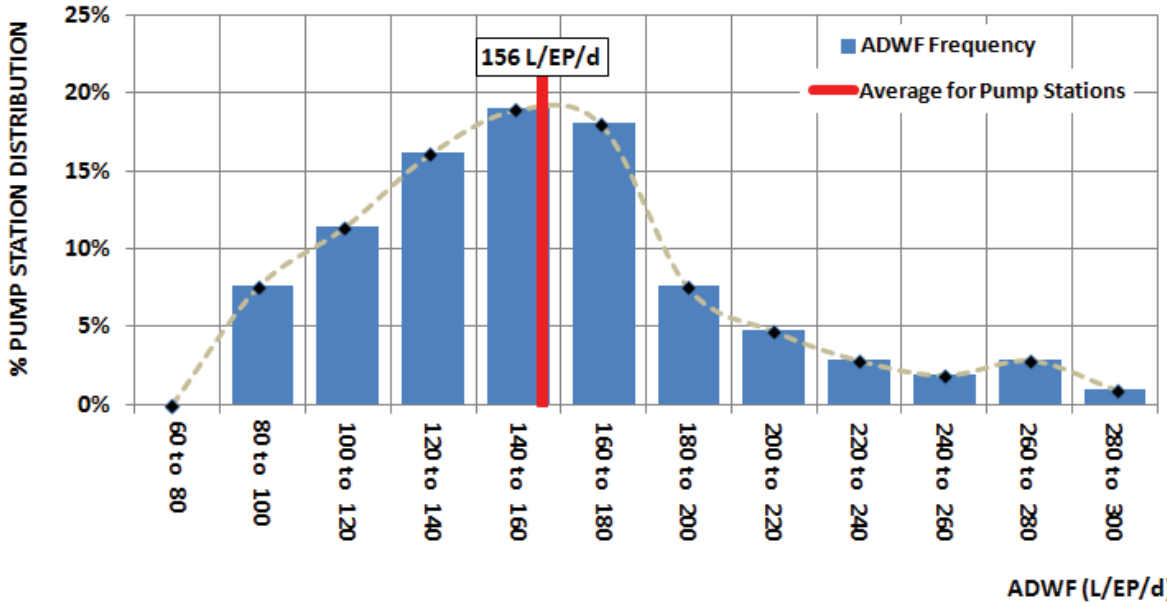


Figure 5-1: ADWF Analysis by pump station

The analysis of pump station flows shows a greater distribution of per capita dry weather flow as expected, however the average for all pump station is 156 L/EP/day which is consistent with the WWTP analysis.

5.1.2 Peak Dry Weather Flow PDWF

The current DSS for PDWF is 2xADWF. As an average, this is generally a reasonable approximation of PDWF. However, the PDWF is variable across catchments and has a strong correlation with catchment size and population density. Therefore, it is recommended that actual flow data should be used to determine the PDWF. If this data is not available, the PDWF equations provided by the WSAA Sewerage Code of Australia should be used. These equations take into account the relationship between average dry weather flow, catchment area and equivalent population.

5.2 Wet Weather Flow

The proposed amendments to the wet weather flow criteria are presented in Table 5-3.

Table 5-3: Wet Weather Flow DSS Criteria

Sewage load	Existing	Proposed
Containment standard for assessment of existing networks	Not specified	Network to contain a 1 year ARI storm event, for critical storm duration. This is an overflow containment standard rather than a conveyance standard. For those catchments that fail the standard, mitigation measures such as I/I reduction and wet weather storage should be considered in addition to any capacity increases.
PWWF; for design of new networks and assessment of existing networks where no calibrated model exists	Residential 1300 L/EP/d	No change
	For CED PWWF = 660 L/EP/d	No change
	Commercial 1300 L/EP/d	No change
	Light Industrial 1000 L/EP/d	No change
	Heavy Industrial 840 L/EP/d	No change

Note: CED = Common Effluent Discharge

5.2.1 Containment Standard

The current approach to wastewater network analysis, both for existing and new networks, is based on ensuring that the network can convey a theoretical PWWF, which is expressed as a multiple of ADWF, historically 5xADWF. This approach was originally developed in the United States in the 1930-40's. It was intended to be a design standard that would predict the maximum instantaneous flow during dry weather and was never intended to account for any wet weather flow.

Over time this has evolved into a wet weather peaking factor that has been widely used for the design and analysis of sewer systems, particularly in Queensland. Actual wet weather peaking factors in many sewer catchments, particularly those with older networks are significantly higher than this. For example the recent

wet weather performance studies undertaken by the LWA[1] demonstrated that the average peaking factor for pump station sewer catchments exceeds 10xADWF and in some areas is as high as 50xADWF. Given that the sewers have only been designed for 5xADWF, wet weather overflows are inevitable.

In addition, this approach can be inefficient. For example, the actual wet weather peaking factors in a relatively new system with good inflow and infiltration (I/I) management may be lower than the theoretical PWWF, resulting in an over sized network. Further, as network models are run in steady state to assess compliance with the theoretical PWWF, the benefits of network storage and attenuation which could reduce the size and extent of conveyance capacity augmentation requirements cannot be assessed. Finally the current approach does not allow for assessment of I/I problems and for solutions to wet weather problems that involve I/I remediation.

On this basis, it is recommended that for analysis of existing networks, an evidence based containment standard that takes account of actual flows in the network be adopted.

There are a number of approaches that can be taken in terms of adopting a containment standard, these include:

- **Single storm event**; which typically involves selection of a storm event for which overflows must not occur. For example a 5 year average recurrence interval (ARI) as is currently required for sewer networks in Victoria.
- **Long time series**; which involves running a rainfall time series through a model (typically 10 years or more) to assess against a standard for maximum allowable frequency, duration and volume of overflows. This is a more data intensive and time consuming process than a single event based standard. This is the approach adopted by many NSW water authorities, including Sydney Water.

Assessment of the performance of a sewer network against either of these two standards requires a calibrated hydraulic model. A calibrated model is a hydraulic model that accurately simulates the actual performance of a wastewater network under all flow conditions. The model is dynamic and allows the performance of the network to be assessed under actual wet weather conditions, from a single event through to a rainfall time series. Obtaining this degree of accuracy requires the model to be calibrated against a range of recorded flows in the network to ensure simulated performance matches actual performance.

Of the two frequency containment standards listed above, the long time series standard is industry best practice and is the standard water authorities in Victoria and NSW are aiming for over the long term. The key challenge to implementing this standard is the extensive data requirements, which includes rainfall data.

Therefore, it is recommended that a single event based containment standard be adopted initially, with a long term goal to move towards a long time series standard. This approach is also supported by the *Overflow Abatement Management Strategy, LWA 2014*.

[1] *Logan North Wet Weather Performance, LWA 2012; Logan East and South Wet Weather Performance Assessment, LWA 2013*)

It is proposed that a 1 year ARI be adopted as the single storm event. A discussion of the basis of the selection of this event is provided in the following section.

Containment Standard Selection

An analysis of flow and rainfall data for the Logan wastewater catchments has been undertaken to quantify current containment performance to help determine an appropriate containment standard. As discussed above, quantifying containment performance requires a calibrated model. Due to the lack of flow data and budgetary constraints only a limited number of calibrated models have been developed by LCC. These include models of 10 pump stations catchments as part of the *Logan North Wet Weather Performance Assessment, LWA 2013,* and two gravity network models (Greater Springwood and Tanah Merah). Due to limitations associated with the calibrated gravity network models, only the calibrated pump station catchment models are referred to in this assessment.

The 10 calibrated pump station catchment models are:

- Browns Plains (SPS09)
- Chambers Flat Road (SPS12)
- Edenlea Drive (SPS20)
- Coral Street (SPS16)
- Priestdale Road West (SPS41)
- Priestdale Road East (SPS42)
- St Andrews Drive 1 (SPS44)
- Trevallyan Drive 1 (SPS48)
- Tygum Road 1 (SPS50)
- Evergreen Ave. (SPS22)

These catchments represent a broad spectrum of wet weather performance and therefore provide a good representation of the overall containment performance of the whole of Logan.

Peaking Factor Vs Storm Event Frequency

The calibrated models were used to determine the performance of the 10 pump station catchments for design storm events varying return period from 1 to 6 year average recurrence intervals (ARI). Figure 5-2 summarises the outcomes of this assessment. It shows the peaking factor response of the catchments to increasing storm return period for the critical storm duration (i.e. storm duration that results in the highest peaking factor). At the low range is Browns Plain pump station where the maximum peak flow is estimated at 8 x ADWF for a 1 year ARI event. At the opposite range is Evergreen Avenue pump station which is estimated to experience peak flows of up to 66xADWF during a 6 year ARI event. This demonstrates the broad range of catchment conditions in the LCC sewerage network.

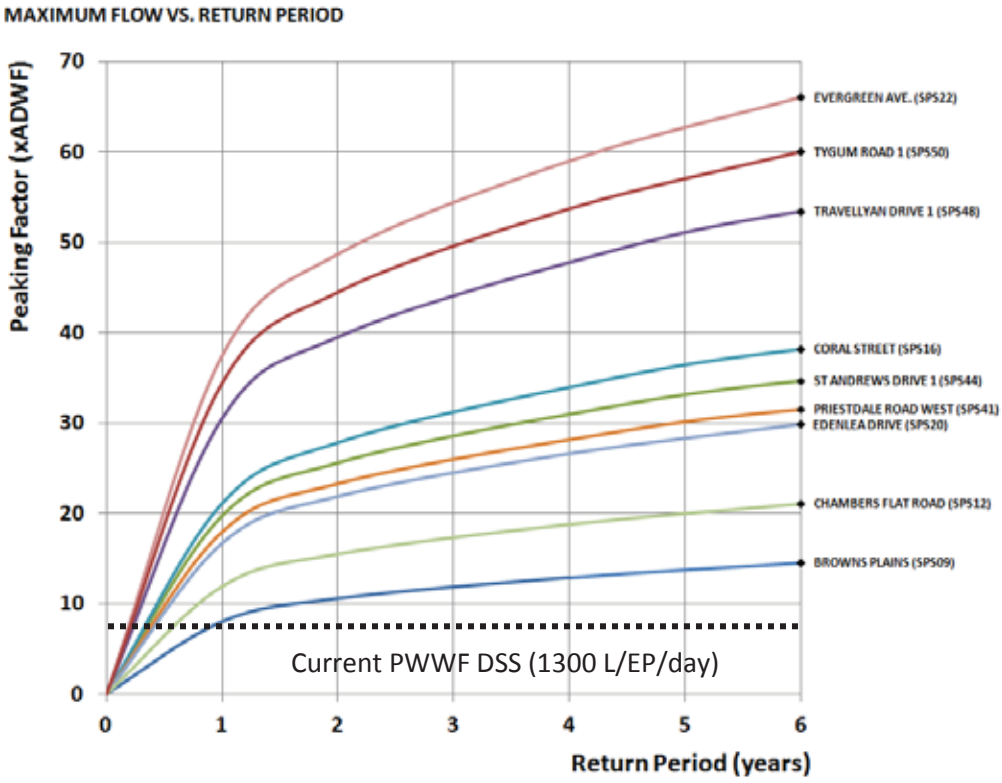


Figure 5-2: Range of Peaking Wet Weather Flows

Figure 5-2 shows that the relationship between increasing return period and peaking factor is logarithmic with peaking factors increasing rapidly up to a 1 year ARI return period with a reduced rate of increase beyond this point. It can therefore be concluded that catering for a 1 year ARI covers the broadest range of peaking factors with diminishing returns beyond the 1 year ARI.

It should be noted that at the current DSS wet weather peaking factor of 1300 L/EP/day (which equates to a peaking factor of 8xADWF based the proposed ADWF of 165 L/EP/d), this graph indicates that only the Browns Plains pump station would be able to cater for a 1 year ARI containment standard.

While this analysis provides useful information on the instantaneous performance of a wastewater network in terms of ability to convey peak flows from a particular storm event, the benefits afforded by network storage are not accounted for. An upstream gravity catchment can provide significant storage which will enhance containment performance, particularly for shorter duration events. The actual containment performance of these catchments is discussed in the following section.

Overflow Analysis

The calibrated models were then used to predict overflows for the same storm events. This combines conveyance and storage aspects of a wastewater network and provides the best overall assessment of containment performance. The results of this analysis are presented in Table 5-4.

Table 5-4: Design Event Containment performance

CATCHMENT	Population	Containment Performance	1 year ARI	2 Year ARI	4 Year ARI	6 Year ARI
	EP	ARI	Overflow Volume kL	Overflow Volume kL	Overflow Volume kL	Overflow Volume kL
Coral Street (SPS16)	9453	<1 yr	1,684	5,354	9,548	12,552
Tygum Road 1 (SPS50)	697	<1 yr	556	938	1,312	1,574
Evergreen Ave. (SPS22)	885	<1 yr	107	577	1,140	1,562
Priestdale Road East (SPS42)	1265	<1 yr	150	507	905	1,201
St Andrews Drive 1 (SPS44)	2095	<1 yr	71	290	656	937
Trevallyan Drive 1 (SPS48)	572	1 yr	0	85	234	397
Chambers Flat Road (SPS12)	11048	1 yr	0	34	196	320
Priestdale Road West (SPS41)	3309	2 yr	0	0	353	775
Edenlea Drive (SPS20)	1692	2 yr	0	0	96	200
Browns Plains (SPS09)	37575	4 yr	0	0	0	31

Of the ten catchments assessed, Coral Street had the greatest overflow volumes across all return period events analysed. Despite servicing a relatively small catchment (830 EP), Evergreen Avenue had the third greatest overflow volumes, which indicates the severity of the inflow and infiltration problems in this catchment.

Half of the catchments analysed, including Trevallyan Drive 1 which is generally considered to have poor wet weather performance all contain the 1 year ARI event. Three of these catchments actually cater for the 2 year ARI and one (Browns Plains) up to a 4 year ARI.

The other half which are all known to perform very poorly in wet weather events and are the source of regular customer complaints all overflow during a 1 year ARI.

This indicates that setting a 1 year ARI as a containment standard is a relatively low bar and failure of the standard does indicate fairly severe wet weather problems that should be addressed. On this basis it is recommended that a containment standard of a 1 year ARI be adopted. It is anticipated that this standard would only be applied to analysis of existing networks and if sufficient flow data has been collected to develop a calibrated model.

Application of this standard will enable the root cause of wet weather problems to be identified and cost efficient solutions to be developed that are not limited to conveyance capacity increases, which typically have high capital costs. Other solutions such as I/I reduction, network renewal, wet weather storages will also be able to be considered and may often provide a more cost efficient solution to wet weather problems than augmentation of conveyance capacity.

For design of new networks or assessment of existing networks where no flow data and/or no calibrated models exist it will still be necessary to specify a PWWF conveyance standard. This is discussed in more detail in the following section.

5.2.2 Peak wet weather flow design factor for new or data poor networks

An assessment has been undertaken to identify the actual peaking factors experienced in the Logan wastewater networks for a range of storm event intensities and identify the peaking factor that corresponds to a 1 year ARI event. This assessment was undertaken using flow data derived from pump station SCADA data for all LCC pump stations. The outcomes of this assessment are shown in Figure 5-3:

The red and green diamonds represent the average peak flow for all pump stations experienced for a particular event. The green diamonds are the average peak factors in Logan North and red are Logan East and South. A trend line was fitted to the average peak flow data (purple dashed line). The trend line provides a reasonable approximation of the average performance of the Logan pump station catchments across a range of storm ARI's. In this case the trend line has a Correlation Coefficient of R=0.78 and a Coefficient of Determination of R^2 = 0.61.

The Correlation Coefficient (R) and Coefficient of Determination (R^2) are a measure of how well the regression line represents the data. If the regression line passes exactly through every point on the scatter plot, it would be able to explain all of the variation. The further the line is away from the points, the less it is able to explain. A Correlation Coefficient (R) greater than 0.8 (Coefficient of Determination R^2 of 0.64) is generally described as strong, whereas a correlation coefficient less than 0.5 (Coefficient of Determination R^2 of 0.25) is generally described as weak.

The trend line shows that for a 1 year ARI, the average peaking factor is about 11xADWF (based on the proposed ADWF of 165 L/EP/day).

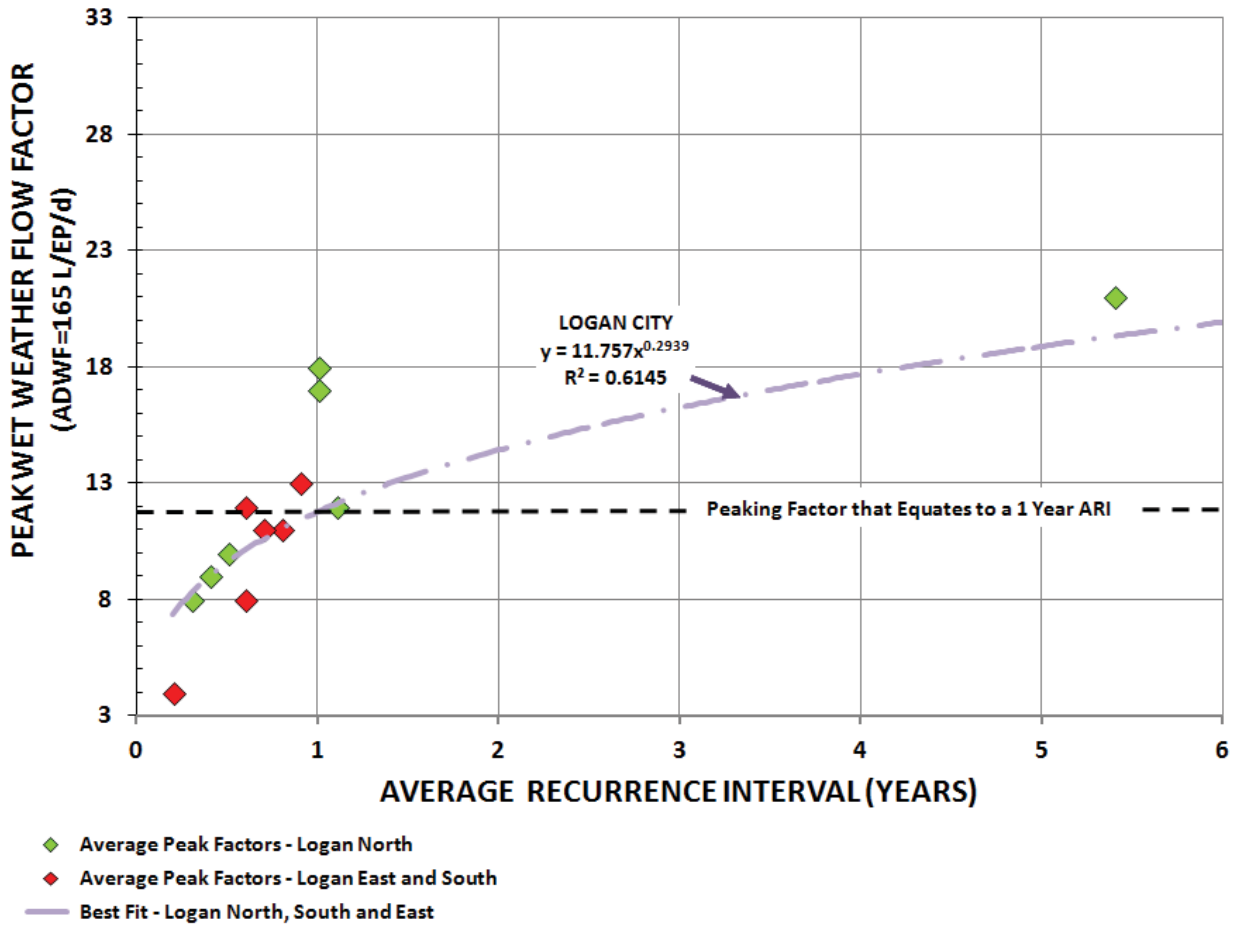


Figure 5-3: Pump station peak wet weather flows versus ARI

The WSAA Sewerage code provides a method for predicting peak wet weather flows for varying loads, ARI's and soil and network conditions. These methods have been used to forecast likely peaking factors for a range of ARI's, and network and soil conditions specific to rainfall patterns in the Logan region. The outcomes of this process are compared against the trend line for the pump station catchments on Figure 5-4.

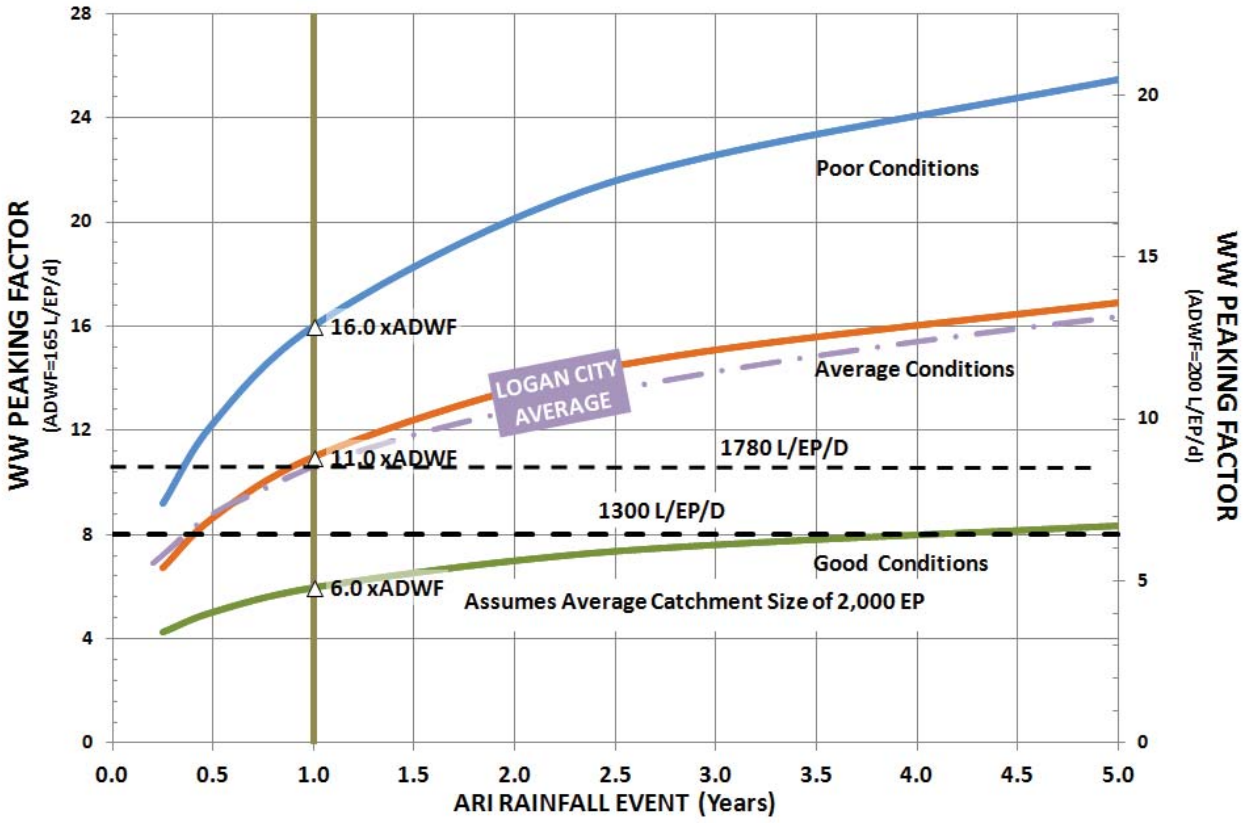


Figure 5-4: PWWF versus ARI: WSAA predictions

The Logan pump stations trend line shows a close correlation with the WSAA derived values for average conditions (i.e. average network condition and average soil condition). This confirms that the WSAA methods are broadly applicable in the Logan context and therefore provide a reasonable basis for specifying a wet weather peaking factor that is tied to a storm event ARI.

Figure 5-4 shows that 1 year ARI event correlates to an 11xADWF or 1780 L/EP/day PWWF for both the Logan average trend line and the WSAA average conditions prediction. On face value this indicates that to contain a 1 year ARI the wastewater network needs to be able to convey a peak wet weather flow of 11xADWF or 1780 L/EP/day (based on the proposed ADWF of 165 L/EP/day). This neglects the attenuating impact of network storage, so it is likely that conveyance of a lower peaking factor would still meet the 1 year ARI without overflows occurring, particularly for larger networks. It also ignores the benefits that can be achieved through I/I reduction and other non infrastructure solutions. On this basis, it is considered that adopting a PWWF of 1780 L/EP/day is likely to be unreasonably high and would impose significant capital cost on the business as much of the existing network is unlikely to meet this PWWF and would require augmentation.

Figure 5-4 also shows that the current DSS PWWF of 1300 L/EP/day which equates to a peaking factor of 8xADWF based on the reduced ADWF of 165 L/EP/day caters for a 0.4-0.5 year ARI assuming average network and soil conditions. If the network and soil conditions are good, however the current DSS would

actually cater for a 4 year ARI. Allowing for network storage benefits, the actual containment performance is likely to be better than this.

Given the significant impact of network storage on containment performance, it is considered that this assessment is not sufficient to recommend a peaking factor that is tied to the 1 year ARI.

Further, as the current DSS for PWWF of 1300 L/EP/day performs reasonably well against the WSAA derived values and lies somewhere between good and average network conditions for the 1 year ARI, it is recommended that the current PWWF of 1300 L/EP/day be retained. In the future as calibrated models are developed for more of the network and the impact of storage on containment performance and associated peak flows become better understood, this PWWF can be revised. Until there are sufficient calibrated models and flow data, however it is considered that there is not sufficient information to make an informed judgement on an appropriate peaking factor that can be universally applied to meet the 1 year containment standard.

Future DSS revisions could also consider adopting different PWWF for different catchments that are tied to catchment/network size, and network and soil conditions.

5.3 Gravity Sewer Design

The proposed amendments to the gravity sewer design criteria are presented in Table 5-5.

Table 5-5: Gravity Sewer DSS Criteria

Gravity sewer design	Existing	Proposed
Flow equation	Manning's 'n' or Colebrook White can be used. For modelling Colebrook - White is preferred.	Colebrook White to be used for all applications – design and modelling
Pipe roughness general	Manning's 'n' =0.013 or Colebrook White k = 1.5mm	Colebrook White k = 1.5mm
Minimum velocity @ PDWF	0.35m/s	Delete
Maximum velocity @ PWWF	3 m/s	No change
Depth of flow @ PWWF - Existing	Up to 1m below MH surface level and no spillage through overflow structures	No change
Depth of flow @ PWWF – Proposed/ new sewers	75% of pipe depth	100% of pipe depth
Depth of flow @ PDWF – existing and new sewers	Not specified	Maximum 60% of pipe depth
Slime Stripping	Not specified	It is desirable where gradients permit to achieve slime stripping. A wall shear of 3.85 Pa is required to achieve slime stripping.

5.3.1 Flow Equation

The Colebrook White formulae is considered to be the best available for determining head loss for turbulent flow in commercial pipe and is the preferred standard in modelling of gravity and pressure mains and should be used in preference to the Manning's or Hazen Williams equations. Use of the Colebrook White formulae will also ensure a consistent approach is adopted for both Water and Wastewater modelling. A Colebrook White roughness of 1.5 mm is proposed for gravity sewers; this is in keeping with WSAA standards and allows for aging and / or slime build up in gravity mains.

5.3.2 Pipe Depth

It is proposed that the maximum depth of flow at PWWF shall be flowing full or 100% of pipe internal diameter rather than the 75% currently specified. This is consistent with the latest WSAA standards.

The maximum depth of flow at PDWF is not to exceed 60% of the pipe internal diameter. This is required to ensure the efficiency of natural ventilation of the sewer and prevent adverse impacts on water traps at property connections.

5.3.3 Slime Stripping

Slime build up occurs in gravity pipelines that have low velocities. This adversely impacts friction coefficients and can lead to odour and septicity problems. As a result, all sewers should be laid at a minimum gradient that will ensure slime stripping occurs at PDWF. This requires calculation of wall shear stress.

The minimum wall shear at PDWF is that required to achieve self cleansing (1.6 Pa) based on the following:

$$\tau = \rho.g.R_p.S$$

Where:

τ	=	Wall Shear Stress (minimum of 1.6 Pa required for self cleansing)		
S	=	Grade of Sewer (m / 100m)		
ρ	=	Density of Water	=	$1000\ kg/m^3$
g	=	Gravitational constant	=	$9.8\ m^2/s$
R_p	=	Hydraulic Radius Partial Full Pipe (m) = Wetted Perimeter / Cross sectional Flow Area		

Where gradients permit the pipes should be constructed at gradients sufficient to achieve slime stripping wall shear of 3.85 Pa at PDWF.

5.4 Sewer minimum gradients

The proposed amendments to the minimum sewer gradient criteria are presented in Table 5-6.

Table 5-6: Sewer Gradient DSS Criteria

	Diameter	Grades (mm, 1 in X)	
		Existing	Proposed
Minimum grades	150	150 (80 in last section between last manholes or to an end)	180 (80 in last section between last manholes or to an end)
	225	290	300
	300	400	400
	375	500	Minimum Grades are to be based on achieving Self Cleansing Shear Stress of 1.6 Pa at PDWF, refer to figure 5-5.
	450	600	
	525	700	
	600	850	
	675	925	
	750	1000	
	825	1000	
	900	1090	
	1050	1270	
	1200	1450	

It is proposed to adopt the Minimum Grades specified in the WSAA Sewerage Code for DN150, DN225 and DN300 pipes.

Minimum Grades for Self Cleansing: The minimum grades are to be based on achieving self cleansing shear stress of 1.6 Pa at PDWF. The following equation shall be used in determining the minimum grades:

$$S_{min} = \frac{\tau}{\rho . g . R_p}$$

Where:

S_{min}	=	Minimum Grade		
τ	=	Self Cleansing Shear Stress	=	1.6 Pa
ρ	=	Density of Water	=	1000 kg/m^3
g	=	Gravitational constant	=	9.8 m^2/s
R_p	=	Hydraulic Radius Partial Full Pipe (m) = Wetted Perimeter / Cross sectional Flow Area		

This equation shows that the minimum slope required to achieve self cleansing is dependent on the hydraulic radius or depth to diameter ratio of the flow in the pipe. This equation has been solved for the primary pipeline diameters in Figure 5-5. The minimum slope shall not be less that that determined from this figure.

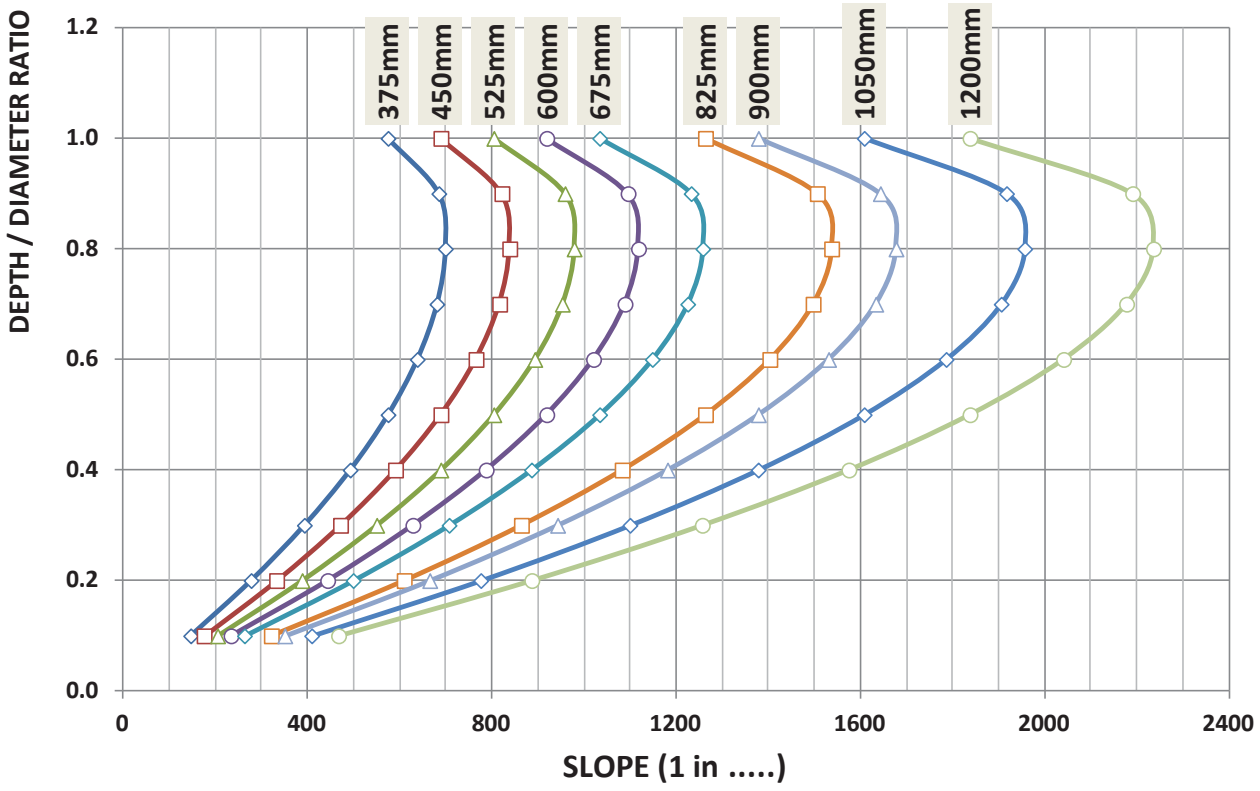


Figure 5-5: Minimum Slope of Gravity Pipes

Minimum grades for Slime control: For sewers ≥DN 300, the minimum self-cleansing velocity or wall shear may not be sufficient to prevent the build-up of slimes. However, increased grades may not be economical and alternative hydrogen sulphide control measures may be necessary. Where specified by LCC, the minimum grade shall ensure that a slime stripping velocity is achieved (Refer to Hydrogen Sulphide Control Manual Volume 1, Monograph 5.1). For branch and trunk sewers, the Designer, in consultation with LCC, shall address minimum grades and velocities and any other requirements necessary to effectively control slime build-up and hydrogen sulphide release.

5.5 Sewer Pump Station

The proposed amendments to the sewer pump station criteria are presented **in Table 5-7**

Table 5-7: Wastewater Pump Station DSS Criteria

Sewerage pump stations	Existing	Proposed
Wet well operating requirements	V (m^3) = 0.9 x pump rate (L/s) / N Where N is the acceptable number of starts per hour Pump Rate (L/s) = capacity of the largest duty pump N = 12 for motors <= 15 kW N = 8 for motors 15kW - 200 kW N = 5 for motors > 200 kW Control levels shall be based on Table 5.1 of WSA04-2005. The minimum depth between duty start and duty off is 100mm and ideally should be 300mm or greater.	No change
Emergency storage	4 hours of ADWF (local gravity catchment only) Except where otherwise specified to meet the requirements of the overflow risk assessment If there is an overflow pipe - storage in the system is measured between the alarm level and overflow pipe invert level. If there is no overflow pipe - storage is measured between the alarm level and 300mm below the lowest upstream manhole or top of wet well.	Emergency storage volume requirement to be based on containment of 2 hours of PDWF. Where actual measured flow data is available then volume should be based on containing 2 hrs measured PDWF provided it takes into account any future growth. Except where otherwise specified to meet the requirements of the overflow risk assessment If there is an overflow pipe - storage in the system is measured between the alarm level and overflow pipe invert level. If there is no overflow pipe - storage is measured between the alarm level and 300mm below the lowest upstream manhole or top of wet well.
Pump capacity	If PWWF <20L/s, then 2 pumps (duty/standby arrangement) shall be provided, with each pump being capable of delivering PWWF. If PWWF >20L/s and <200L/s, then 2 pumps (duty/duty assist arrangement) shall be provided, with each pump being sized such that the two pumps running in parallel are capable of delivering PWWF. It is desirable that a single pump be sized on C1 if appropriate, i.e. From QDNRM. C1 = 15 x EP-0.1587 minimum 3.5, maximum 6.5. If PWWF >200L/s, then minimum of 3 pumps (duty/duty assist/standby arrangement) shall be provided. Duty/duty assist pumps being capable of delivering PWWF with each pump being sized in consultation with Allconnex Water.	If PWWF <20L/s, then 2 pumps (duty/standby arrangement) shall be provided, with each pump being capable of delivering PWWF. If PWWF >20L/s and <200L/s, then 2 pumps (duty/duty assist arrangement) shall be provided, with each pump being sized such that the two pumps running in parallel are capable of delivering PWWF. The minimum single pump capacity shall be PDWF. It is desirable that a single pump be sized on C_1 if appropriate, i.e. From DEWS. $C_1 = 15 \times EP^{-0.1587}$ minimum 3.5, maximum 6.5. If PWWF >200L/s, then minimum of 3 pumps (duty/duty assist/duty assist/standby arrangement) shall be provided. Minimum single pump capacity shall be PDWF. All pumps operating shall be capable of delivering PWWF with each pump being sized in consultation with Logan City Council.
Pump operation	VSD shall only be used, subject to special approval from Allconnex Water. If approved by Allconnex Water, the VSD operating regime shall be selected to ensure sufficient self-cleansing velocities, and shall meet the pump supplier's requirements.	VSD shall only be used, subject to special approval from Logan City Council. If approved by Logan City Council, the VSD operating regime shall be selected to ensure sufficient self-cleansing velocities, and shall meet the pump supplier's requirements.
NPSH	***Refer Clause 6.4 of WSA04-2005***	No Change

5.5.1 Pump Design and Selection

The pump shall be selected to operate at its best efficiency during single pump operation (where it operates mostly during ADWF conditions). Pump selection shall be based on the roughnesses included under Section 5.6 of this report. However, manufacturer's roughness for new pipe should also be considered to ensure that the pump is able to operate (not necessarily at design flow) under all operating conditions including surcharging.

Pump Stations shall be sized such that the pump station is capable of meeting or exceeding all possible peak diurnal dry weather flows with one pump out of service. The combined pumps duty and assist pumps (i.e. all pumps operating) shall be able to meet or exceed PWWF.

5.5.2 Emergency Storage

The current volume requirement for emergency storage is four hours of average dry weather flow. This is based on an assumed operational response time with a safety margin to ensure that no dry weather spills will occur in the case of a major failure at a pump station. The issue with this approach is that PDWF, particularly for small catchments may be higher than 2xADWF. In this case, an emergency storage volume of 4 hours ADWF may be insufficient to prevent a dry weather spill within the expected response time.

To demonstrate this point, actual flow data from a range of wastewater pump stations throughout LCC have been analysed, refer to Table 5-8. All flow rates referred to in this table are derived from pump station SCADA data.

Table 5-8: Actual PDWF volume vs. 4 hours ADWF

ID	Name	EP	ADWF L/s	PDWF L/s	PDWF duration hrs	4 hrs ADWF volume kL	Actual PDWF volume kL	Difference kL
SPS124	Merrow Street 1	463	1.48	3.50	3.0	21.3	37.8	-16.5
SPS79	Flagstone 2	1060	1.67	4.00	2.5	24.0	36.0	-12.0
SPS24	Hanlon Street	1545	2.61	6.80	2.0	37.6	49.0	-11.4
SPS21	Elm Park	1707	1.88	4.50	2.0	27.1	32.4	-5.3
SPS47	Third Avenue	2776	6.35	13.00	2.0	91.4	93.6	-2.2
SPS16	Coral Street	5722	19.16	45.00	2.0	275.9	324.0	-48.1
SPS12	Chambers Flat Road 1	10570	16.21	35.00	2.5	233.4	315.0	-81.6
SPS09	Browns Plains	36233	68.41	180.00	2.5	985.1	1620.0	-634.9

Table 5-8 shows that for all pump stations analysed the actual PDWF volume exceeds 4 hours of dry weather flow. If the emergency storage was to be sized on 4 hours ADWF at these pump stations, there is a

high likelihood that a dry weather spill will occur if a pump station failure occurs. On this basis, it is recommended that the PDWF be used as the basis for determining emergency storage requirements.

Discussions with operational staff indicate that their response time for dealing with a pump station failure is around one hour. This allows for setting up and starting a bypass pump. This assumes ideal conditions and that there are no other emergencies underway that requires operators' attention. Therefore, it is recommended that all emergency storage should be designed to fully contain the actual peak dry weather flow volume. For the pump stations assessed in the above table, this equated to between 2 and 3 hours. As a general rule of thumb, it is recommended that the volume should be a minimum of 2 hours PDWF, although the calculation should be based on actual flow data where available.

5.6 Rising Mains

The proposed amendments to the rising mains criteria are presented in **Table 5-9**

Table 5-9: Rising Mains DSS Criteria

Rising mains	Existing	Proposed
Flow equation	Hazen Williams or Colebrook-White. For modelling Colebrook - White is preferred	Colebrook White to be used for all applications
Friction mains	100mm – 300mm diameter, C =110 > 300mm diameter, C =130 Colebrook-White to follow method in **WSA04**. The pump curve shall include the minimum static head with C=140 to confirm that the pump operates over the full range (i.e. to the overflow level) in the wet well.	For preliminary design a Colebrook White Roughness (k) of 1 mm is to be used. For detailed design roughness shall be based on velocity and pipe roughness at end of pump design life. However, pumps should be able to operate (not necessarily meet design flow) for Colebrook White roughness's between manufacturers specified roughness for new pipes and 1.0mm
Minimum velocity	Minimum velocity shall be not less than 0.9m/s, but preferred minimum is 1.5m/s. ***Refer Clause 10.3.5 of WSA04-2005***	Minimum Velocity to be based on achieving Slime Stripping Shear Stress of 3.85 Pa at Single Pump Flow
Maximum velocity	2.5 m/s proposed systems	3.0 m/s proposed systems Manufacturer's specifications for new systems
Max detention time	Maximum time of detention in pressure main and SPS wet well is 6 hours [2 hours in wet well and 4 hours in rising main] (based on daily average flows) to minimise potential for odour / hydrogen sulphide generation. Where high retention times are likely to occur, odour / sulphide control measures will be required to the satisfaction of Allconnex Water. ***(Refer WSA 07-2007-1.1 section 3.15)***	Maximum time of detention in pressure main and SPS wet well is 6 hours [2 hours in wet well and 4 hours in rising main] (based on daily average flows) to minimise potential for odour / hydrogen sulphide generation. Where high retention times are likely to occur, odour / sulphide control measures will be required to the satisfaction of LCC. ***(Refer WSA 07-2007-1.1 section 3.15)***

5.6.1 Flow Equation

The Colebrook White formulae is considered to be the best available for determining head loss for turbulent flow in commercial pipe and is the preferred standard in modelling of gravity and pressure mains. It will also ensure a consistent approach is adopted for both Water and Wastewater modelling.

5.6.2 Velocities

The Colebrook White roughness adopted is as per the WSAA Sewage Pump Station Code WSA04-2005 and applies to all materials. The roughness is dependent on main velocities; with increased roughness associated with lower velocities which is a result of slime build up on the internal walls of the pipe. The maximum velocity in the main has been increased to 3 m/s to reflect pipe manufacturer's specifications.

The minimum velocity shall be based on the wall shear at single pump capacity that is required to achieve self cleansing (1.6 Pa).

The wall shear is to be calculated using the following equation:

$$\tau = \rho.g.(D/4).(H_L/L)$$

Where:

τ	=	Wall Shear Stress (minimum of 1.6 Pa required for self cleansing)		
H_L	=	Frictional loss over main length of L based on Colebrook White (m)		
L	=	Length of main (m)		
$\boldsymbol{\rho}$	=	Density of Water	=	$1000\ kg/m^3$
g	=	Gravitational constant	=	$9.8\ m^2/s$
D	=	Internal Diameter of Pipe (m)		

5.7 Reduced Infiltration Gravity Sewers (RIGS)

The proposed amendments to the RIGS criteria are presented in Table 5-10.

Table 5-10: RIGS DSS Criteria

Reduced infiltration gravity sewers (RIGS)	Existing	Proposed
As per gravity sewers except	Not permitted without Allconnex Water approval	Not permitted without Logan City Council approval
PWWF	1000L/EP/d	Reticulation system to be designed for 1000 L/EP/day Trunk systems to be designed for minimum 1300 L/EP/day
Single Pump Capacity	Standard: C_1 x ADWF Where: $C_1 = 9.322 \times EP^{-0.1249}$ minimum 3.0, maximum 4.	Remove
Total PS Capacity	1000L/EP/d	1300 L/EP/day

The actual performance of RIGS systems have been assessed in the LCC wastewater system. Table 5-11 summarises the performance of pump station catchments that service RIGS gravity systems. All of the catchments selected are comprised of 100% of RIGS mains.

Table 5-11: RIGS Pump Station Catchment Performance

RIGS PUMPSTATIONS		6 month ARI PWWF (L/EP/day)	1 Year ARI PWWF (L/EP/day)	2 Year ARI PWWF (L/EP/day)	5 Year ARI PWWF (L/EP/day)	Remaining Life of Gravity Mains
RIGS PUMPSTATIONS						
SPS105	Bladensburg Drive	850	996	1,158	1,368	64 years
SPS120	Ravensbourne Circuit	869	1,002	1,150	1,340	66 years
SPS137	Teys Road	1,022	1,215	1,428	1,703	66 years
SPS116	Emerson Road	1,222	1,334	1,458	1,617	70 years
SPS72	Logan Village 3	1,555	1,862	2,201	2,638	69 years
	AVERAGE	***1,104***	***1,282***	***1,479***	***1,733***	***67 years***
LOGAN OVERALL						
	10 Percentile of Pump Stations	774	865	990	1,131	12 years
	90 Percentile of Pump Stations	2,543	3,315	4,171	5,276	64 years
	AVERAGE	***1,548***	***1,916***	***2,324***	***2,849***	***38 years***

Table 5-11 shows that for a 6 month ARI event PWWFs at SPS137, SPS116 and SPS72 exceed 1000 L/EP/d. For a 1 in 1 year ARI event, SPS105 is the only RIGS station that has a PWWF less than 1000 L/EP/D. For ARI events greater than 1 year all of the RIGS pump station catchments exceed 1000 L/EP/D.

The data in this table has been plotted in Figure 5-6 to compare the RIGS catchment performance with that of the LCC average.

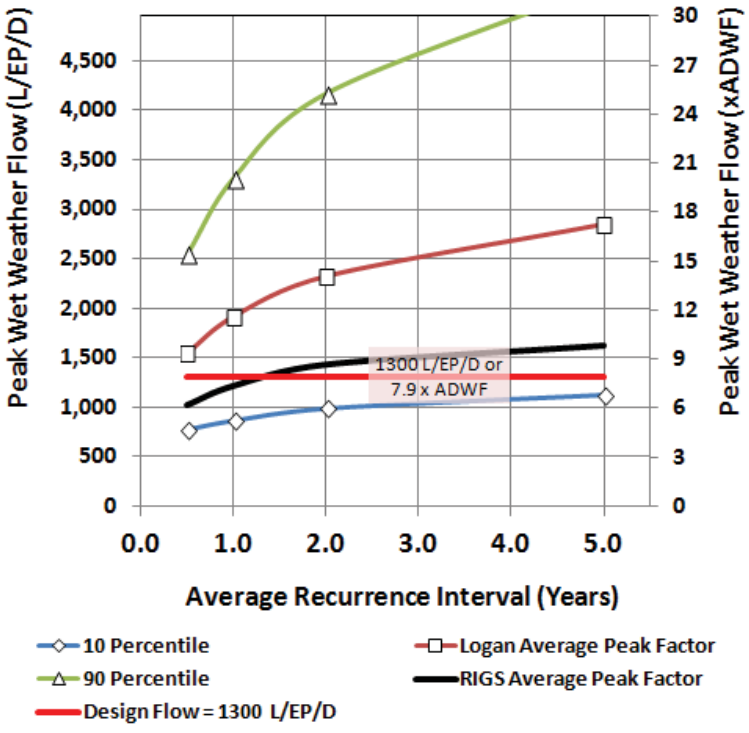


Figure 5-6: Performance of RIGS Sewers

Figure 5-6 shows that in general the RIGS sewers have performed better than the overall Logan wastewater system. However, the RIGS sewers were constructed after 2005 and the average remaining life is significantly higher than the average remaining life of the overall Logan wastewater system which should result in improved wet weather performance. Generally, I/I increase with the age of the mains and it is likely that the performance of the RIGS sewers will deteriorate.

The estimated 1 Year ARI PWWF of the RIGS sewers is estimated at 1282 L/EP/D. This is higher than the previous PWWF for RIGS of 1000 L/EP/D and the PWWF standard recommended by the SEQ Design Code. On this basis it is considered that the non-RIGS sewer PWWF of 1300 L/EP/day should be adopted for trunk mains downstream of RIGS systems.

It is recommended that that PWWF for RIGS in the SEQ Design Code should be reviewed and updated in line with this finding.

5.8 Summary of Proposed DSS for Wastewater

The proposed changes to the Desired Standards of Service for Wastewater planning are outlined in Appendix E.

6. CONCLUSIONS

The following key conclusions are drawn from this study:

- Water
    - **Average Day Consumption.** Detailed water demand analysis indicates that the current planning figure for residential demand of 260 L/EP/d (230 L/EP/d and 30 L/EP/d water loss) is overly conservative. A figure of 190 L/EP/d (165 L/EP/d and 25 L/EP/d water loss) provides for recently recorded demand levels and approximately 10% further rebound. This figure aligns with the current proposals put forward by Seqwater for regional planning.

      Constant Flow demand of 180 L/EP/d (130 L/EP/d and 50 L/EP/d water loss) is also considered to be too high based on the analysis of recent data which indicates average demand for these customers of 60 L/EP/d. Recognising that this demand component is extremely sensitive to climate conditions a factor of safety of 2 is considered prudent. A slightly higher water loss for these areas to account for the higher mains to property ratio typical for rural areas is also appropriate with a 150 L/EP/d (120 L/EP/d and 30 L/EP/d water loss) figure proposed.
    - **Diurnal Patterns.** A review of daily demand patterns indicate that the peak hour occurs in the evening. The current pattern has a morning peak indicative of the drought response to outdoor use which typically occurs in the evening. The new patterns are considered appropriate for existing and future planning.
    - **Peaking Factors.** A reduction to residential peaking factors has also been suggested as appropriate based on an analysis of recent data
    - **Storage and Pumping.** The reduction in planning demands of70 L/EP/d will impact on the sizing and timing of future infrastructure, particularly reservoirs and pump stations
    - **Roughness Calculations.** Hazen Williams' variance in headloss coefficient is considered unsuitable for accurate headloss prediction. Colebrook White provides a more accurate description of network conditions.
- Wastewater
    - **Average Dry Weather Flows**. Detailed analysis of dry weather wastewater flows by both treatment plant and pump station indicate the current average planning value of 200 L/EP/d is high with results suggesting a range of 158 to 163 L/EP/d. A figure of 165 L/EP/d is therefore considered prudent.
    - **Peak Dry Weather Flows**. An arbitrary factor (2xADWF) is no longer considered suitable and the use of actual data or the WSAA formula is proposed.
    - **Wet Weather Containment**. The current approach to designing wastewater networks for wet weather flows is based on ensuring that the network can convey a theoretical PWWF. This approach is not appropriate for analysis of existing networks. In this situation, an evidence based containment standard that focuses on actual overflows rather than conveyance is considered prudent.

      A proposed containment standard of a 1 year ARI is recommended. Failure of this standard generally indicates severe wet weather problems that should be addressed. It is anticipated that this standard would only be applied to analysis of existing networks and if sufficient flow data has been collected to develop a calibrated model.

Application of this standard will allow solutions to wet weather problems that may be more cost effective than conveyance capacity increase to be developed, e.g. I/I reduction and wet weather storage.

- **Peak Wet Weather Flows.** A peak wet weather flow standard is still required for design of new networks and analysis of existing networks where insufficient flow data is available. An analysis of the existing PWWF standard of 1300 L/EP/day against actual flow data for LCC pump stations indicates that on average it caters a 1 year ARI in good soil and network conditions. This is reduced to a 6 month ARI in average network and soil conditions. Adopting a higher PWWF would impose unreasonable conveyance requirements upon LCC and could result in extensive and costly augmentations, when better value for money could be obtained through other measures such as I/I reduction. On this basis, it is recommended that the current PWWF of 1300 L/EP/day be retained.

7. RECOMMENDATIONS

The Logan Water Alliance recommends that:

1. The modified water and wastewater DSS documented in Appendix D and E are adopted for the master planning review and subsequent planning activities.
2. The adopted changes to the DSS are forwarded to the SEQ Code working group for inclusion in the next document revision. Further review of discrepancies between DDS and code be undertaken in a formalised repose be provided the custodians of the SEQ Code to ensure appropriate criteria are adopted.
3. The DSS continue to be reviewed on a periodic basis and adjusted accordingly.

8. REFERENCES

Logan Water Alliance 2010, *Review of Desired Standards of Service (Task 90-10-29)*

Logan Water Alliance 2013, *Reservoir Reliability Risk Assessment (Task 90-12-12)*

Gold Coast City Council, Logan City Council Queensland Urban Utilities, Redland Water & Unity Water 2013, *South East Queensland Water Supply and Sewerage Design and Construction Code*

Water Services Association of Australia (WSAA) 2002*, Sewerage Code of Australia WSA-02*

Water Services Association of Australia (WSAA) 2011*, Water Supply Code of Australia WSA-03*

Logan Water Alliance 2010, *Queensland Fire and Rescue Service, Service Level Recommendations for Semi-Rural Areas (Task 90-10-14)*

Logan Water Alliance 2014, *Overflow Abatement Strategy (Task 90-12-50)*

Department of Energy and Water Supply (DEWS) 2014, *Planning Guidelines for Water Supply and Sewerage*

Department of Natural Resources and Mines (DNRM) 2005, *Draft Amendments to Planning Guidelines for Water Supply and Sewerage*

Logan Water Alliance 2012, *Logan North Wet Weather Performance (Task 90-11-94*)

Logan Water Alliance 2013, *Logan East and South Wet Weather Performance Assessment. (Task 90-12-10)*

Hydrogen Sulphide Control Manual Volume 1

WSA 02 -2002 Version 2.3, Sewerage Code of Australia; Part 1 Planning and Design

WSA 04 -2005 Version 2.1, Sewage Pump Station Code of Australia; Part 1 Planning and Design

Appendix A Existing Water DSS

WATER NETWORK DSS			
Parameter	**Criteria**		
Water Demand			
Average Day demand	On demand areas - 260 L/EP/d. Based on 230 L/EP/d + allowance for leakage/losses (30 L/EP/d).		
	Constant flow areas - 180 L/EP/d. Based on 130 L/EP/d + allowance for leakage/losses (50 L/EP/d).		
Peaking factors	**MDMM/AD**	**MD/AD**	**MH/AD**
Residential detached	1.4	1.7	3.8
Residential attached	1.3	1.6	3.1
Rural residential	1.4	1.7	CF services 70 L/hr/ET
Commercial	1.2	1.3	2.0
Industry	1.2	1.3	1.7
Parks / Open Space	1.2	1.3	1.7
System planning			
Bulk supply and reticulation	3 days of MDMM. Reservoirs to have a net positive inflow and capable of continuous operation and not fall below the emergency level. 3 days of MD. Reservoirs should not fall below the emergency level. 5 days of AD. Reservoirs should fill from empty to full		
Minimum service pressure			
Minimum operating pressure at MH	On demand areas – 22m at the property boundary based on reservoir at minimum operating level (MOL). MOL defined as 15% of storage height or top of emergency storage		
	Constant flow areas – 10m at the property boundary based on reservoir at minimum operating level (MOL) MOL defined as 15% of storage height or top of emergency storage.		
Maximum operating pressure	80 m at the boundary based on the reservoir at TWL		
Target pressure	55 m at the property boundary based on the reservoir level at TWL		
Fire fighting			
System pressure	On demand areas – 12 m minimum at property boundary		
	Constant flow areas – no fire service provided		
Fire flow	Residential – 15L/s for 2 hrs Commercial & Industrial – 30 L/s for 4 hrs Special risk / hazard to be advised by Allconnex Water		
Background demand	2/3 MH (as per DERM requirements)		
Reservoir level	MOL		
Number of fires	<5000 EP – single fire for residential or industrial / commercial >5000 EP – simultaneous single residential fire plus single commercial / industrial fire for sizing trunk mains		
Water quality			
Drinking water quality	Drinking water to comply with the NHMRC Australian Drinking Water Guidelines – 2004.		

Reservoir storage	
Ground level storage capacity	3 (MD - MDMM) + (Greater of emergency storage/fire fighting storage) Emergency Storage = the greater of 4 hr MDMM demand in zone or 0.5 ML whichever is greater. *Where Fire Fighting Reserve is:* ***> 4800EP:*** • CF services for rural residential 72 kL (2 x 5 L/s x 2hrs); • for industry, business 540 kL (i.e. 30 L/s x 4hrs + 15 L/s x 2hrs) otherwise 210 kL (2 x 15L/s x 2hrs). **< 4800 EP:** • CF services 36 kL (15L/s x 2 hrs); for business & industry 432 kL (30L/s for 4 hrs); otherwise 150kL (= 15 L/s for > 2hrs).
Elevated storage capacity	6(MH - 1/12 MDMM) + minimum fire fighting reserve. *Where fire fighting reserve is:* > **4800EP**: • CF services for rural residential 72 kL (2 x 5 L/s x 2hrs); • for industry, business 540 kL (i.e. 30 L/s x 4hrs + 15 L/s x 2hrs) • otherwise 210 kL (2 x 15L/s x 2hrs). < **4800 EP**: • CF services 36 kL (15L/s x 2 hrs); • for business & industry 432 kL (30L/s for 4 hrs); • otherwise 150kL (= 15 L/s for > 2hrs).
Pumping capacity	
Pump supplying a ground level reservoir	MDMM over 20 hr
Pump supplying an elevated reservoir	(6MH – operating volume) / (6 x 3600)
Standby pump capacity	To match duty, except where more than one duty pump or as determined by risk assessment
Reticulation booster pumps/pumped system	MH + fire flow
Pipeline design	
Main capacity	Raw water mains & trunk feeding ground level reservoir : MDMM for a gravity supply and MDMM over 20hrs for a pumped supply Trunk Mains feeding elevated reservoir: capacity of pumps Reticulation Mains: MH + fire flow
Mains size	Constant flow areas - where mains terminate in short runs such as cu- de-sacs minimum sizes that may be acceptable are: DN 63 up to 150 m from main DN 80 up to 350 m from main
	On demand areas - minimum reticulation main 100mm in residential areas and 150mm in commercial and industrial areas
Friction default value	Hazen Williams Distribution: <300mm, C=110 300mm – 600mm, C=120 >600mm, C=125 Reticulation: <= 150mm, C=100 200mm - 300mm, C=110 >300mm, C=120
Maximum velocity	2.5 m/s

Appendix B Existing Wastewater DSS

WASTEWATER NETWORK DSS		
Parameter	**Criteria**	
Sewage load		
ADWF	200L/EP/day	
PDWF	2 x ADWF	
PWWF	Residential 1300 L/EP/d For CED PWWF = 660 L/EP/d	
	Commercial 1300 L/EP/d	
	Light Industrial 1000 L/EP/d	
	Heavy Industrial 840 L/EP/d	
Gravity sewer design		
Flow equation	Manning's 'n' or Colebrook White can be used. For modelling Colebrook - White is preferred.	
Pipe roughness general	Manning's 'n' =0.013 or Colebrook White k = 1.5mm	
Minimum velocity @ PDWF	0.35m/s	
Maximum velocity @ PWWF	3 m/s	
Depth of flow @ PWWF - Existing	Up to 1m below MH surface level and no spillage through overflow structures	
Depth of flow @ PWWF - Proposed	75% of pipe depth	
Minimum grades	**Diameter**	**Grades (mm, 1 in X)**
	150	150 (80 in last section between last manholes or to an end)
	225	290
	300	400
	375	500
	450	600
	525	700
	600	850
	675	925
	750	1000
	825	1000
	900	1090
	1050	1270
	1200	1450

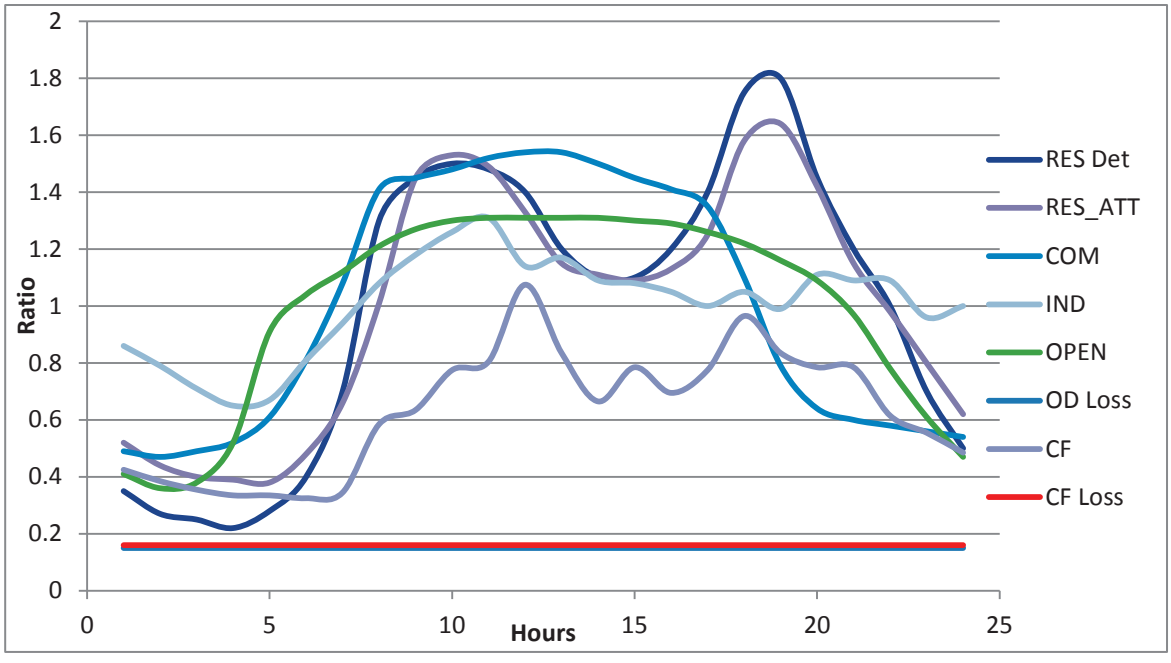
Sewerage pump stations	
Wet well operating requirements	V (m^3) = 0.9 x pump rate (L/s) / N Where N is the acceptable number of starts per hour Pump Rate (L/s) = capacity of the largest duty pump N = 12 for motors <= 15 kW N = 8 for motors 15kW - 200 kW N = 5 for motors > 200 kW Control levels shall be based on Table 5.1 of WSA04-2005. The minimum depth between duty start and duty off is 100mm and ideally should be 300mm or greater.
Emergency storage	4 hours of ADWF (local gravity catchment only) Except where otherwise specified to meet the requirements of the overflow risk assessment If there is an overflow pipe - storage in the system is measured between the alarm level and overflow pipe invert level. If there is no overflow pipe - storage is measured between the alarm level and 300mm below the lowest upstream manhole or top of wet well.
Pump capacity	If PWWF <20L/s, then 2 pumps (duty/standby arrangement) shall be provided, with each pump being capable of delivering PWWF. If PWWF >20L/s and <200L/s, then 2 pumps (duty/duty assist arrangement) shall be provided, with each pump being sized such that the two pumps running in parallel are capable of delivering PWWF. It is desirable that a single pump be sized on C_1 if appropriate. i.e. From QDNRM. $C_1 = 15 \times EP^{-0.1587}$ minimum 3.5, maximum 6.5. If PWWF >200L/s, then minimum of 3 pumps (duty/duty assist/standby arrangement) shall be provided. Duty/duty assist pumps being capable of delivering PWWF with each pump being sized in consultation with Allconnex Water.
Pump operation	VSD shall only be used, subject to special approval from Allconnex Water. If approved by Allconnex Water, the VSD operating regime shall be selected to ensure sufficient self-cleansing velocities, and shall meet the pump supplier's requirements.
NPSH	***Refer Clause 6.4 of WSA04-2005***
Rising mains	
Flow equation	Hazen Williams or Colebrook-White. For modelling Colebrook - White is preferred
Friction mains	100mm – 300mm diameter, C =110 > 300mm diameter, C =130 Colebrook-White to follow method in **WSA04**. The pump curve shall include the minimum static head with C=140 to confirm that the pump operates over the full range (i.e. to the overflow level) in the wet well.
Minimum velocity	Minimum velocity shall be not less than 0.9m/s, but preferred minimum is 1.5m/s. ***Refer Clause 10.3.5 of WSA04-2005***
Maximum velocity	2.5 m/s proposed systems
Max detention time	Maximum time of detention in pressure main and SPS wet well is 6 hours [2 hours in wet well and 4 hours in rising main] (based on daily average flows) to minimise potential for odour / hydrogen sulphide generation. Where high retention times are likely to occur, odour / sulphide control measures will be required to the satisfaction of Allconnex Water. ***(Refer WSA 07-2007-1.1 section 3.15)***
Reduced infiltration gravity sewers (RIGS)	
As per gravity sewers except	Not permitted without Allconnex Water approval
PWWF	1000L/EP/d
Single Pump Capacity	Standard: C_1 x ADWF
	$C_1 = 9.322 \times EP^{-0.1249}$ minimum 3.0, maximum 4.
Total PS Capacity	1000L/EP/d

Low pressure sewers / vacuum pumps	
Low pressure sewers / vacuum pumps	Are not preferred and will only be allowed subject to approval from Allconnex Water.
Private pump stations and rising mains	
	Private pump stations and rising mains are permitted for single users only and no connection or sharing will be permitted without approval from Allconnex Water
Common effluent discharge	
Common effluent discharge (CED)	Not permitted without approval from Allconnex Water.

Appendix C Updated Water Supply Diurnal Patterns

RES_DET	RES_ATT	COM	IND	OPEN	CF Loss	OD Loss	CF
0.35	0.52	0.49	0.86	0.41	0.16	0.15	0.42
0.27	0.44	0.47	0.79	0.36	0.16	0.15	0.38
0.25	0.4	0.49	0.71	0.38	0.16	0.15	0.35
0.22	0.39	0.52	0.65	0.52	0.16	0.15	0.33
0.28	0.38	0.61	0.67	0.91	0.16	0.15	0.33
0.4	0.48	0.81	0.81	1.04	0.16	0.15	0.32
0.7	0.66	1.08	0.94	1.12	0.16	0.15	0.34
1.3	1.01	1.41	1.08	1.21	0.16	0.15	0.58
1.45	1.45	1.45	1.18	1.27	0.16	0.15	0.63
1.5	1.53	1.48	1.26	1.3	0.16	0.15	0.77
1.48	1.49	1.52	1.31	1.31	0.16	0.15	0.80
1.4	1.33	1.54	1.14	1.31	0.16	0.15	1.07
1.2	1.15	1.54	1.17	1.31	0.16	0.15	0.83
1.1	1.11	1.5	1.09	1.31	0.16	0.15	0.66
1.1	1.09	1.45	1.08	1.3	0.16	0.15	0.78
1.2	1.13	1.41	1.05	1.29	0.16	0.15	0.69
1.4	1.25	1.35	1	1.26	0.16	0.15	0.77
1.75	1.58	1.1	1.05	1.22	0.16	0.15	0.96
1.8	1.64	0.79	0.99	1.16	0.16	0.15	0.83
1.45	1.42	0.64	1.11	1.09	0.16	0.15	0.78
1.2	1.15	0.6	1.09	0.97	0.16	0.15	0.78
1	0.98	0.58	1.09	0.78	0.16	0.15	0.61
0.7	0.8	0.56	0.96	0.61	0.16	0.15	0.55
0.5	0.62	0.54	1	0.47	0.16	0.15	0.48
1.0	**1.0**	**1.0**	**1.0**	**1.0**	**0.16**	**0.15**	**0.63**

RES_DET=residential detached, RES_ATT=residential attached, COM=commercial, IND=Industrial, OPEN=Open spaces, CF =Constant Flow, CF Loss= CF Leakage, OD Loss =On demand Leakage



Appendix D Recommended Water Desired Standards of Service

WATER NETWORK DSS			
Parameter	**Criteria**		
Water Demand			
Average Day demand	On demand areas - 190 L/EP/d. Based on 165 L/EP/d + allowance for leakage/losses (25 L/EP/d).		
	Constant flow areas - 150 L/EP/d. Based on 120 L/EP/d + allowance for leakage/losses (30L/EP/d).		
Peaking factors	**MDMM/AD**	**MD/AD**	**MH/AD**
Residential detached	1.3	1.7	3.1
Residential attached	1.3	1.6	2.6
Rural residential	1.3	1.7	CF services 70 L/hr/property
Commercial	1.2	1.3	2.0
Industry	1.2	1.3	1.7
Parks / Open Space	1.2	1.3	1.7
System planning			
Bulk supply and reticulation	3 days of MDMM. Reservoirs to have a net positive inflow and capable of continuous operation and not fall below the emergency level. 3 days of MD. Reservoirs should not fall below the emergency level. 5 days of AD. Reservoirs should fill from empty to full		
Minimum service pressure			
Minimum operating pressure at MH	On demand areas – 22m at the property boundary based on reservoir at minimum operating level (MOL). MOL defined as 15% of storage height or top of emergency storage		
	Constant flow areas – 10m at the property boundary based on reservoir at minimum operating level (MOL) MOL defined as 15% of storage height or top of emergency storage.		
Maximum operating pressure	80 m at the boundary based on the reservoir at TWL		
Target pressure	55 m at the property boundary based on the reservoir level at TWL		
Fire fighting			
System pressure	On demand areas	12 m minimum at property boundary	
	Constant flow areas	no fire service provided	
Fire flow	Constant flow areas	no fire service provided	
	Rural Residential	7.5 L/s for 2 hrs	
	Rural Commercial / Industrial	15 L/s for 2 hrs	
	Residential	15 L/s for 2 hrs	
	Commercial & Industrial	30 L/s for 4 hrs	
	Special risk / hazard	to be advised by Logan City Council	
Background demand	2/3 PH (as per DEWS requirements)		
Reservoir level	Minimum Operating Level (MOL)		

Water quality	
Drinking water quality	Drinking water to comply with the NHMRC Australian Drinking Water Guidelines – 2004.

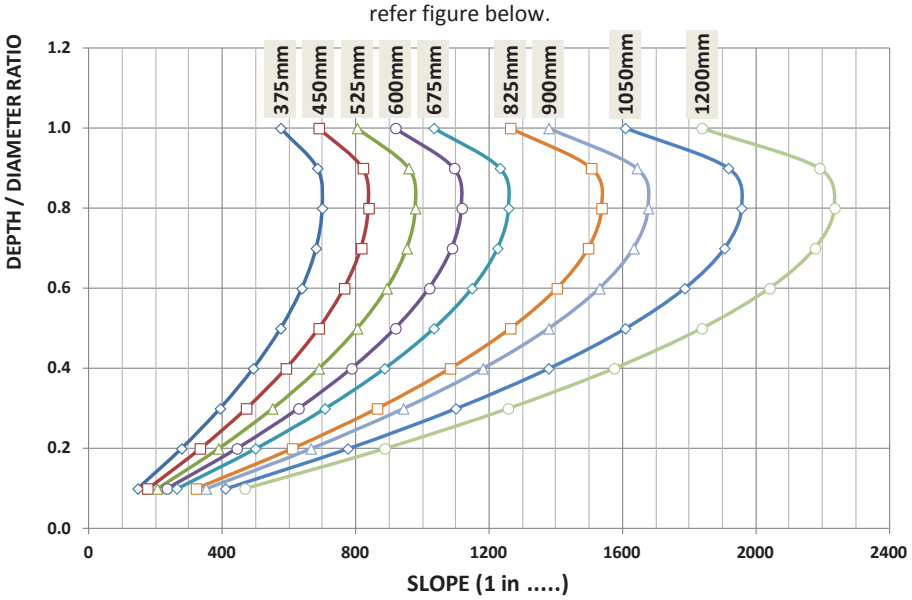
Reservoir storage	
Ground level storage capacity	3 (MD - MDMM) + (Greater of emergency storage/fire fighting reserve) Emergency Storage = 4 hr MDMM demand in zone or 0.5 ML whichever is greater. *Fire Fighting* reserve = ***> 4800EP:*** • rural residential 108 kL • industry 540 kL • otherwise 216 kL **< 4800 EP:** • rural residential 54 kL • industry 432 kL • otherwise 108 kL
Elevated storage capacity Guidance Note: In-line pressure booster pumping without high-level storage shall be considered where satisfactory pressures can be provided by gravity flow during low demand periods or booster failure can be mitigated to an acceptable level (risk assessment) and where there is no other feasible means of providing adequate pressure.	6 (PH - 1/12 MDMM) + minimum fire fighting reserve. *Fire Fighting reserve =* ***> 4800EP:*** • rural residential 108 kL • industry 540 kL • otherwise 216 kL **< 4800 EP:** • rural residential 54 kL • industry 432 kL • otherwise 108 kL

Pumping capacity	
Pump supplying a ground level reservoir	MDMM over 20 hr
Pump supplying an elevated reservoir	(6PH – operating volume) / (6 x 3600)
Standby pump capacity	To match duty, except where more than one duty pump or as determined by risk assessment
Reticulation booster pumps/pumped system	PH + fire flow

<table>
<tr><td colspan="3">Pipeline assessment & design</td></tr>
<tr><td>Main capacity</td><td colspan="2">Raw water mains & trunk feeding ground level reservoir : MDMM for a gravity supply and MDMM over 20hrs for a pumped supply
Trunk Mains feeding elevated reservoir: capacity of pumps
Reticulation Mains: PH + fire flow</td></tr>
<tr><td rowspan="2">Mains size</td><td colspan="2">Constant flow areas - where mains terminate in short runs such as cu- de-sacs minimum sizes that may be acceptable are:
DN 63 up to 150 m from main
DN 80 up to 350 m from main</td></tr>
<tr><td colspan="2">On demand areas - minimum reticulation main 100mm in residential areas and 150mm in commercial and industrial areas</td></tr>
<tr><td rowspan="2">Friction default value</td><td>Existing Infrastructure</td><td>Future Infrastructure</td></tr>
<tr><td>Colebrook White k values (mm)
<table>
<tr><th rowspan="2">Pipe Material</th><th colspan="3">Pipeline Age (years)</th></tr>
<tr><th>< 10</th><th>10 to 25</th><th>> 25</th></tr>
<tr><td>Asbestos Cement</td><td>0.15</td><td>0.3</td><td>0.3</td></tr>
<tr><td>Plastic (UPVC, MDPE, Hobas, etc)</td><td>0.06</td><td>0.06</td><td>0.15</td></tr>
<tr><td>MSCL/DICL</td><td>0.3</td><td>0.3</td><td>0.6</td></tr>
<tr><td>CICL</td><td>0.3</td><td>0.3</td><td>0.6</td></tr>
</table></td><td>The design shall be conducted to achieve less than the following head losses:
(a) 5 m head/km for ≤DN 150 (CIOD) or ≤DN 180 (ISO).
(b) 3 m head/km for ≥DN 200 (CIOD) or ≥DN 250 (ISO).
Head loss shall be calculated using computer models or hydraulic formulas.</td></tr>
<tr><td>Maximum velocity</td><td colspan="2">2.5 m/s
At peak hour for trunk mains and at peak hour including fire flow for reticulation</td></tr>
</table>

Appendix E Recommended Wastewater Desired Standards of Service

WASTEWATER NETWORK DSS	
Parameter	**Criteria**
Sewage load	
ADWF	165 L/EP/day
PDWF	Use actual flow data; or $d \times 165$ L/EP/d or; $d \times 0.00191 \times EP$ L/s *Where:* $d = 0.01(\log A)^4 - 0.19(\log A)^3 + 1.4(\log A)^2 - 4.66 \log A + 7.57$ *d = dry weather peaking factor* *A = gross plan area of development catchment, in hectares* *Refer to WSAA Sewerage Code of Australia; Part 1 Planning and Design*
Containment standard for assessment of existing networks	Network to contain a 1 year ARI storm event, for critical storm duration. This is an overflow containment standard rather than a conveyance standard. For those catchments that fail the standard, mitigation measures such as I/I reduction and wet weather storage should be considered in addition to any capacity increases.
PWWF	Residential 1300 L/EP/d
	For Common Effluent Discharge PWWF = 660 L/EP/d
	Commercial 1300 L/EP/d
	Light Industrial 1000 L/EP/d
	Heavy Industrial 840 L/EP/d
Gravity sewer design	
Flow equation	Colebrook White to be used for all applications – design and modelling
Pipe roughness general	Colebrook White k = 1.5mm
Maximum velocity @ PWWF	3 m/s
Depth of flow @ PWWF - Existing	Up to 1m below MH surface level and no spillage through overflow structures
Depth of flow @ PWWF - Proposed/ new sewers	100% of pipe depth
Depth of flow @ PDWF – existing and new sewers	Maximum 60% of pipe depth
Slime Stripping	It is desirable where gradients permit to achieve slime stripping. A wall shear of 3.85 Pa is required to achieve slime stripping.

<table>
<tr><td rowspan="13">Minimum grades</td><td>Diameter</td><td>Grades (mm, 1 in X)</td></tr>
<tr><td>150</td><td>180 (80 in last section between last manholes or to an end)</td></tr>
<tr><td>225</td><td>300</td></tr>
<tr><td>300</td><td>400</td></tr>
<tr><td>375</td><td rowspan="12">Minimum Grades are to be based on achieving Self Cleansing Shear Stress of 1.6 Pa at PDWF, refer figure below.
 
</td></tr>
<tr><td>450</td></tr>
<tr><td>525</td></tr>
<tr><td>600</td></tr>
<tr><td>675</td></tr>
<tr><td>750</td></tr>
<tr><td>825</td></tr>
<tr><td>900</td></tr>
<tr><td>1050</td></tr>
<tr><td>1200</td></tr>
<tr><td colspan="3">Sewerage pump stations</td></tr>
<tr><td colspan="2">Wet well operating requirements</td><td>$V\ (m^3) = \frac{0.9 \times \text{pump rate (L/s)}}{N}$
Where N is the acceptable number of starts per hour
Pump Rate (L/s) = capacity of the largest duty pump
N = 12 for motors <= 15 kW
N = 8 for motors 15kW - 200 kW
N = 5 for motors > 200 kW
Control levels shall be based on Table 5.1 of WSA04-2005. The minimum depth between duty start and duty off is 100mm and ideally should be 300mm or greater.</td></tr>
<tr><td colspan="2">Emergency storage</td><td>Emergency storage volume requirement to be based on containment of 2 hours of PDWF. Where actual measured flow data is available then volume should be based on containing 2 hrs measured PDWF provided it takes into account any future growth.
Except where otherwise specified to meet the requirements of the overflow risk assessment
If there is an overflow pipe - storage in the system is measured between the alarm level and overflow pipe invert level.
If there is no overflow pipe - storage is measured between the alarm level and 300mm below the lowest upstream manhole or top of wet well.</td></tr>
<tr><td colspan="2">Pump capacity</td><td>If PWWF <20L/s, then 2 pumps (duty/standby arrangement) shall be provided, with each pump being capable of delivering PWWF.
If PWWF >20L/s and <200L/s, then 2 pumps (duty/duty assist arrangement) shall be provided, with each pump being sized such that the two pumps running in parallel are capable of delivering PWWF. The minimum single pump capacity shall be PDWF. . It is desirable that a single pump be sized on C_1 if appropriate, i.e. From DEWS Planning Guidelines. C_1 = 15 x EP-0.1587 minimum 3.5, maximum 6.5.</td></tr>
</table>

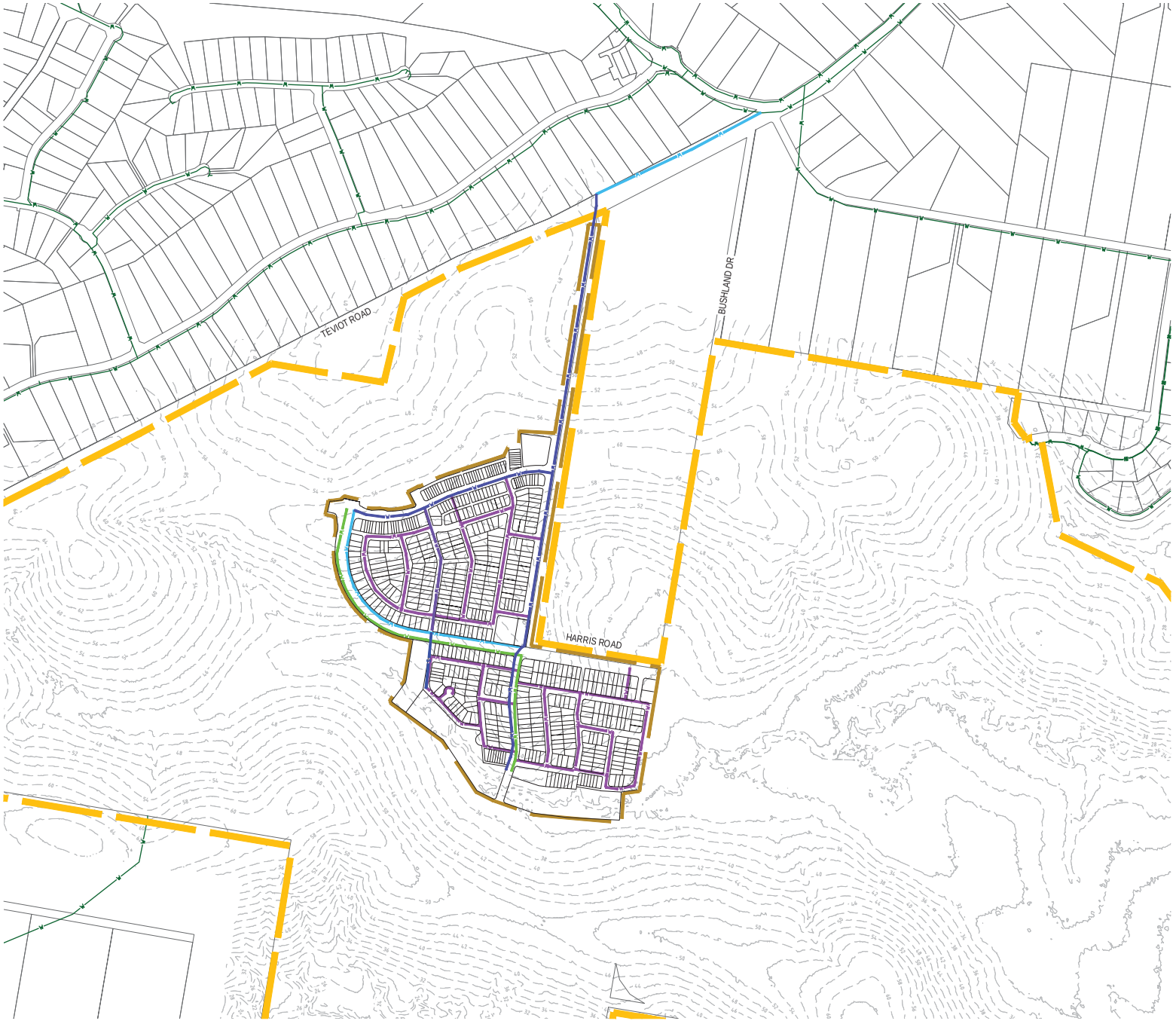
	If PWWF >200L/s, then minimum of 3 pumps (duty/duty assist/duty assist/…. arrangement) shall be provided. Minimum single pump capacity shall be PDWF. All pumps operating shall be capable of delivering PWWF with each pump being sized in consultation with LCC.
Pump operation	VSD shall only be used, subject to special approval from Logan City Council. If approved by Logan City Council, the VSD operating regime shall be selected to ensure sufficient self-cleansing velocities, and shall meet the pump supplier's requirements.
NPSH	***Refer Clause 6.4 of WSA04-2005***

Rising mains	
Flow equation	Colebrook White to be used for all applications
Friction mains	For preliminary design a Colebrook White Roughness (k) of 1 mm is to be used. For detailed design roughness shall be based on velocity and pipe roughness at end of pump design life. However, pumps should be able to operate (not necessarily meet design flow) for Colebrook White roughness's between manufacturers specified roughness for new pipes and 1.0mm
Minimum velocity	Minimum Velocity to be based on achieving Slime Stripping Shear Stress of 3.85 Pa at Single Pump Flow
Maximum velocity	3.0 m/s proposed systems Manufacturer's specifications for new systems
Max detention time	Maximum time of detention in pressure main and SPS wet well is 6 hours [2 hours in wet well and 4 hours in rising main] (based on daily average flows) to minimise potential for odour / hydrogen sulphide generation. Where high retention times are likely to occur, odour / sulphide control measures will be required to the satisfaction of Logan City Council. ***(Refer WSA 07-2007-1.1 section 3.15)***
Reduced infiltration gravity sewers (RIGS)	
As per gravity sewers except	Not permitted without Logan City Council approval
PWWF	Reticulation system to be designed for 1000 L/EP/day Trunk systems to be designed for minimum 1300 L/EP/day
Total PS Capacity	1300L/EP/d
Low pressure sewers / vacuum pumps	
Low pressure sewers / vacuum pumps	Are not preferred and will only be allowed subject to approval from Logan City Council.
Private pump stations and rising mains	
	Private pump stations and rising mains are permitted for single users only and no connection or sharing will be permitted without approval from Logan City Council
Common effluent discharge	
Common effluent discharge (CED)	Not permitted without approval from Logan City Council.

ROL 1 JIMBOOMBA CELESTINO

Appendix C Calibre Drawings

CELESTINO PTY LTD



LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- DN100 WATER MAIN
- DN225 WATER MAIN
- DN300 WATER MAIN
- DN375 WATER MAIN
- EXISTING WATER MAINS

CONCEPT PLAN ONLY
NOT TO BE USED FOR CONSTRUCTION PURPOSES

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
A	03.11.17	DRAFT	RH	JG
B	06.12.17	ROL 1 UPDATE	RH	JG
C	21.02.19	ROL 1 UPDATE	JM	JG
D				
E				
F				
G				
H				

DRAWN
DESIGN

STATUS: **NOT FOR CONSTRUCTION**
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SCALE
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1:10000 A3

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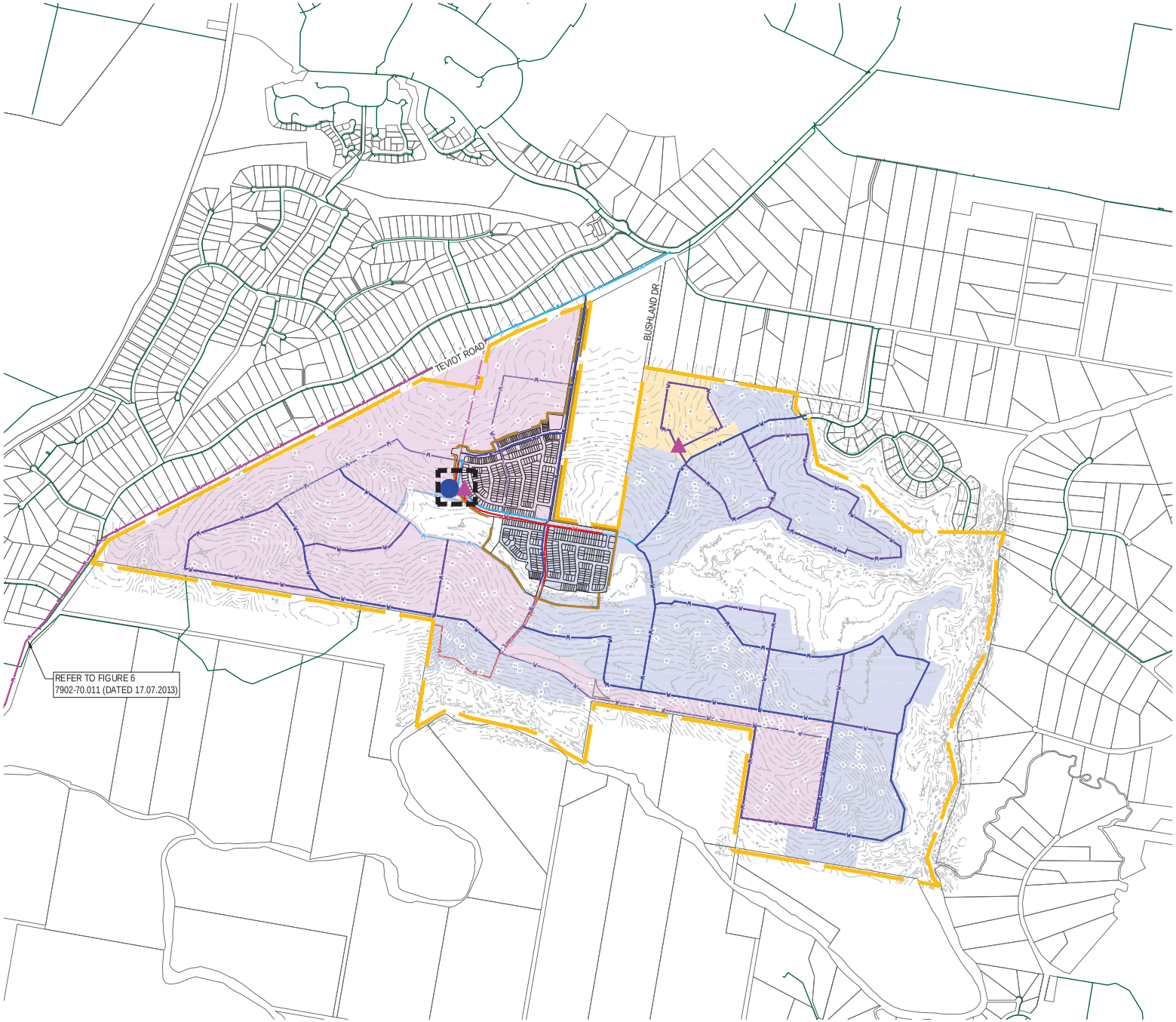
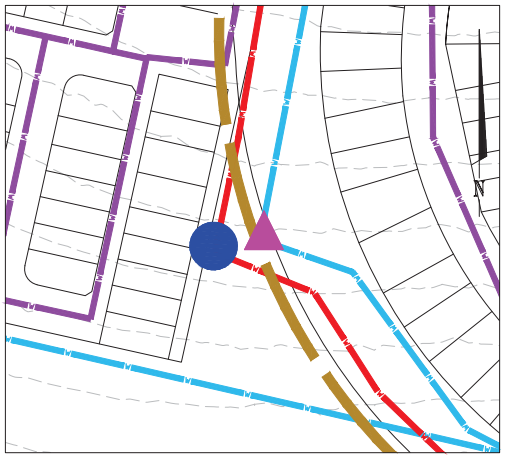
PROJECT No.	DRAWING No.	REVISION
16-003350	401	A





PUMP STATION AND PRV CONFIGURATION
SCALE 1:1000 (A1)
SCALE 1:2000 (A3)

LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- EXISTING CADASTRE
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- DN150 WATER MAIN
- DN225 WATER MAIN
- DN300 WATER MAIN
- DN375 WATER MAIN
- DN450 WATER MAIN
- EXISTING WATER MAINS
- BOOSTER PUMP STATION
- PRESSURE REDUCING VALVE
- HLZ1
- HLZ2
- LLZ

CONCEPT PLAN ONLY
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REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
A	03.11.17	DRAFT	RH	JG
B	06.12.17	ROL 1 UPDATE	RH	JG
C	21.02.19	ROL 1 UPDATE	JM	JG
D				
E				
F				
G				
H				

DRAWN
DESIGN

STATUS: **NOT FOR CONSTRUCTION**
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SCALE
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1:20000 A3

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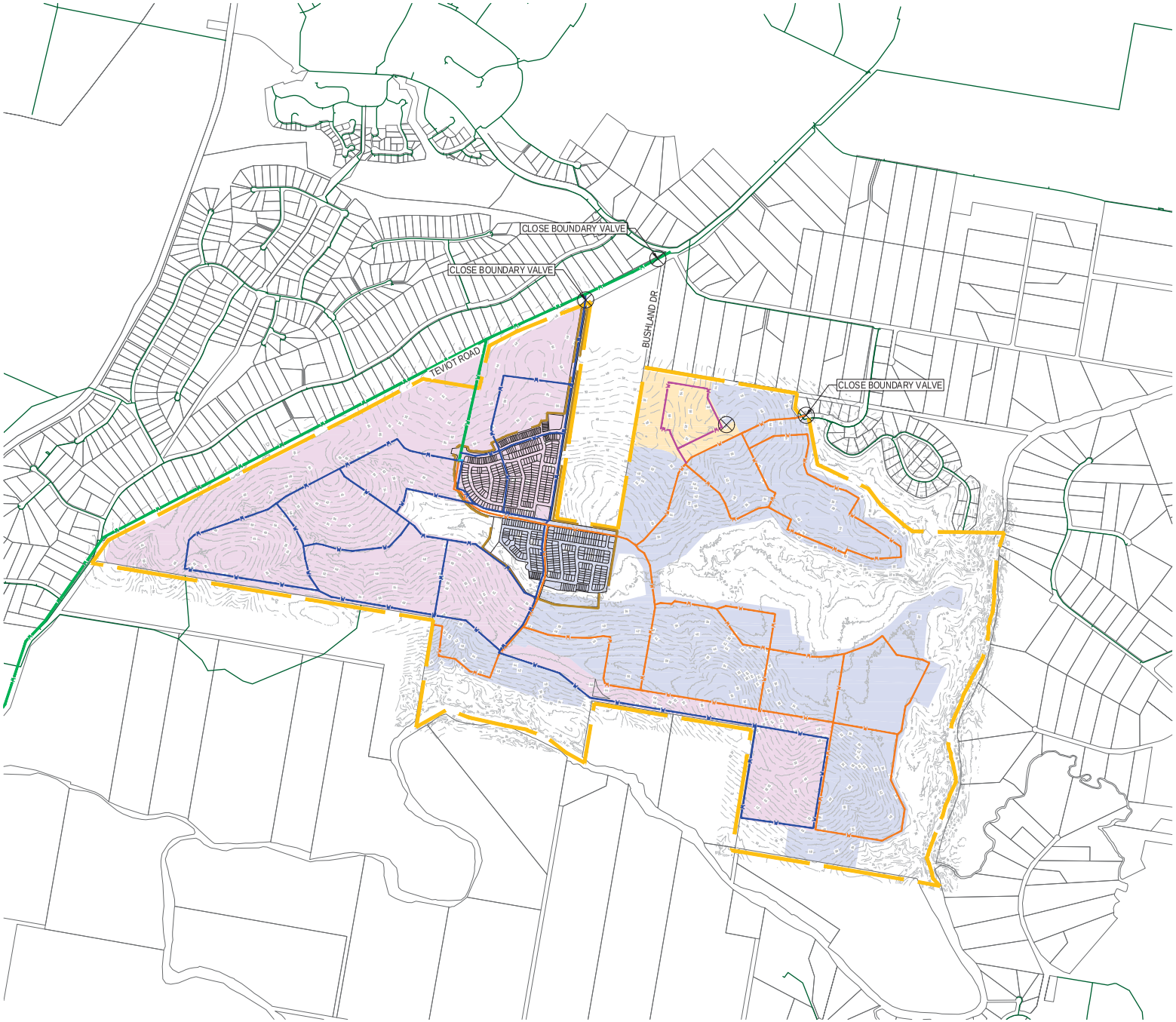
16-003350	402	A



LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- EXISTING CADASTRE
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- HLZ1 WATER MAINS
- HLZ2 WATER MAINS
- LLZ WATER MAINS
- EXTERNAL WATER MAINS
- EXISTING WATER MAINS
- HLZ1
- HLZ2
- LLZ
- CLOSE BOUNDARY VALVE

CONCEPT PLAN ONLY
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REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
A	03.11.17	DRAFT	RH	JG
B	06.12.17	ROL 1 UPDATE	RH	JG
C	21.02.19	ROL 1 UPDATE	JM	JG
D				
E				
F				
G				
H				

DRAWN
DESIGN

STATUS: **NOT FOR CONSTRUCTION**
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SCALE
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1:20000 A3

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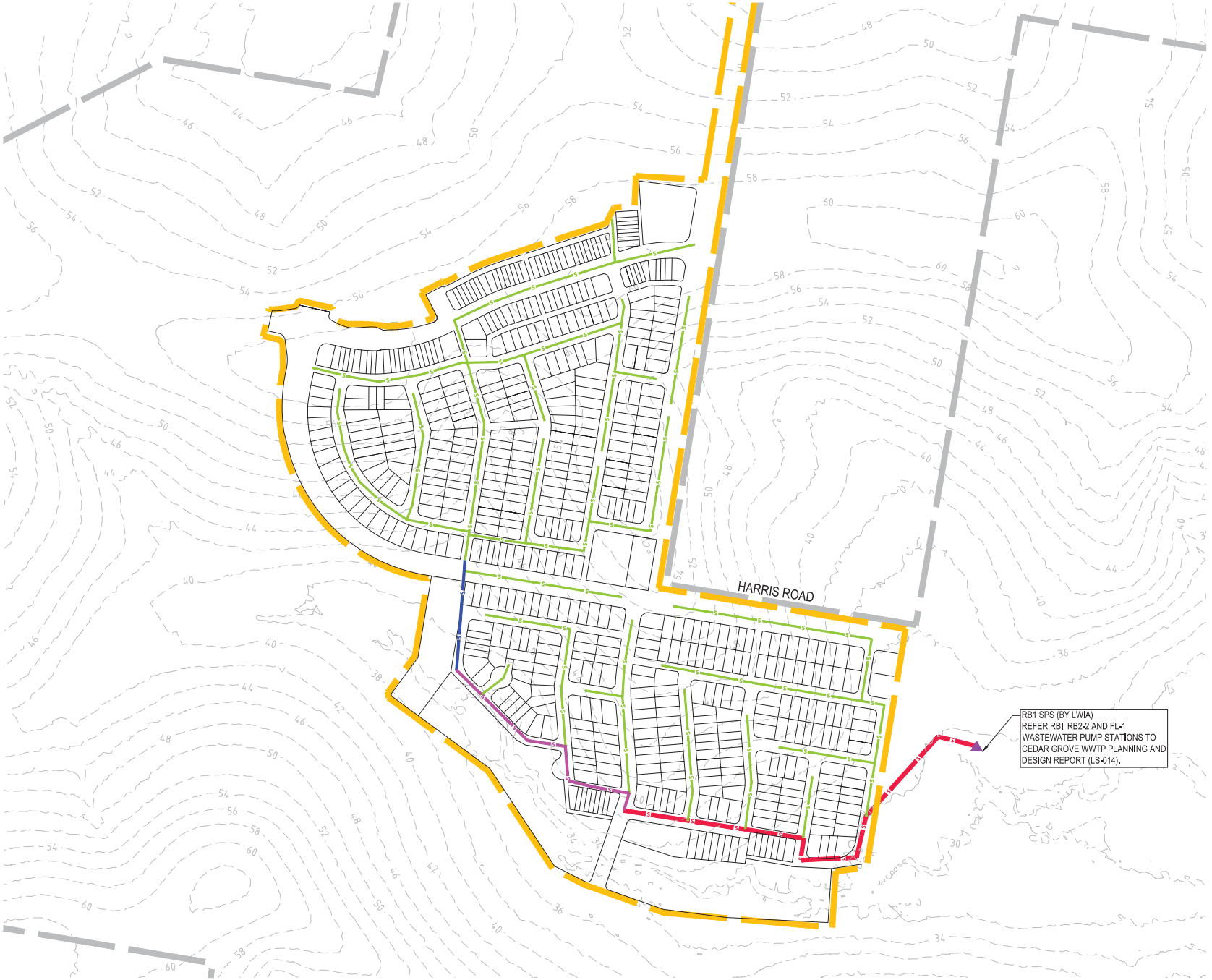
DRAWING TITLE: **PROPOSED WATER SUPPLY DMA'S**

PROJECT No.	DRAWING No.	REVISION
16-003350	403	A



LEGEND:

- MASTER PLAN BOUNDARY
- EXISTING LOTS BOUNDARY
- ROL 1 LOT BOUNDARY
- EXISTING CONTOURS
- INDICATIVE PUMP STATION LOCATION (BY OTHERS
- PROPOSED DN150 WASTEWATER MAIN
- PROPOSED DN300 WASTEWATER MAIN
- PROPOSED DN450 WASTEWATER MAIN
- PROPOSED DN525 WASTEWATER MAIN
- PROPOSED WASTEWATER RISING MAIN

NOTES:

1. WASTEWATER SERVICING STRATEGIES PROPOSED ARE CONCEPTUAL AND REMAIN SUBJECT TO DETAILED ENGINEERING DESIGN.
2. PROPOSED INFRASTRUCTURE ALIGNMENTS, DIAMETERS AND MATERIALS ARE SUBJECT TO DEVELOPMENT DENSITIES AND WILL BE CONFIRMED DURING DETAILED DESIGN.
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REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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B	04.12.17	NETWORK UPDATE	RH	JG
C	06.12.17	ROL 1 UPDATE	RH	JG
D	14.12.17	UPDATED RISING MAIN ALIGNMENT	JG	JG
E	21.02.19	UPDATED RISING MAIN ALIGNMENT	JM	JG
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G				
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DRAWN

DESIGN

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SCALE: 1:2000 20 0 20 40 60 80 100 A1
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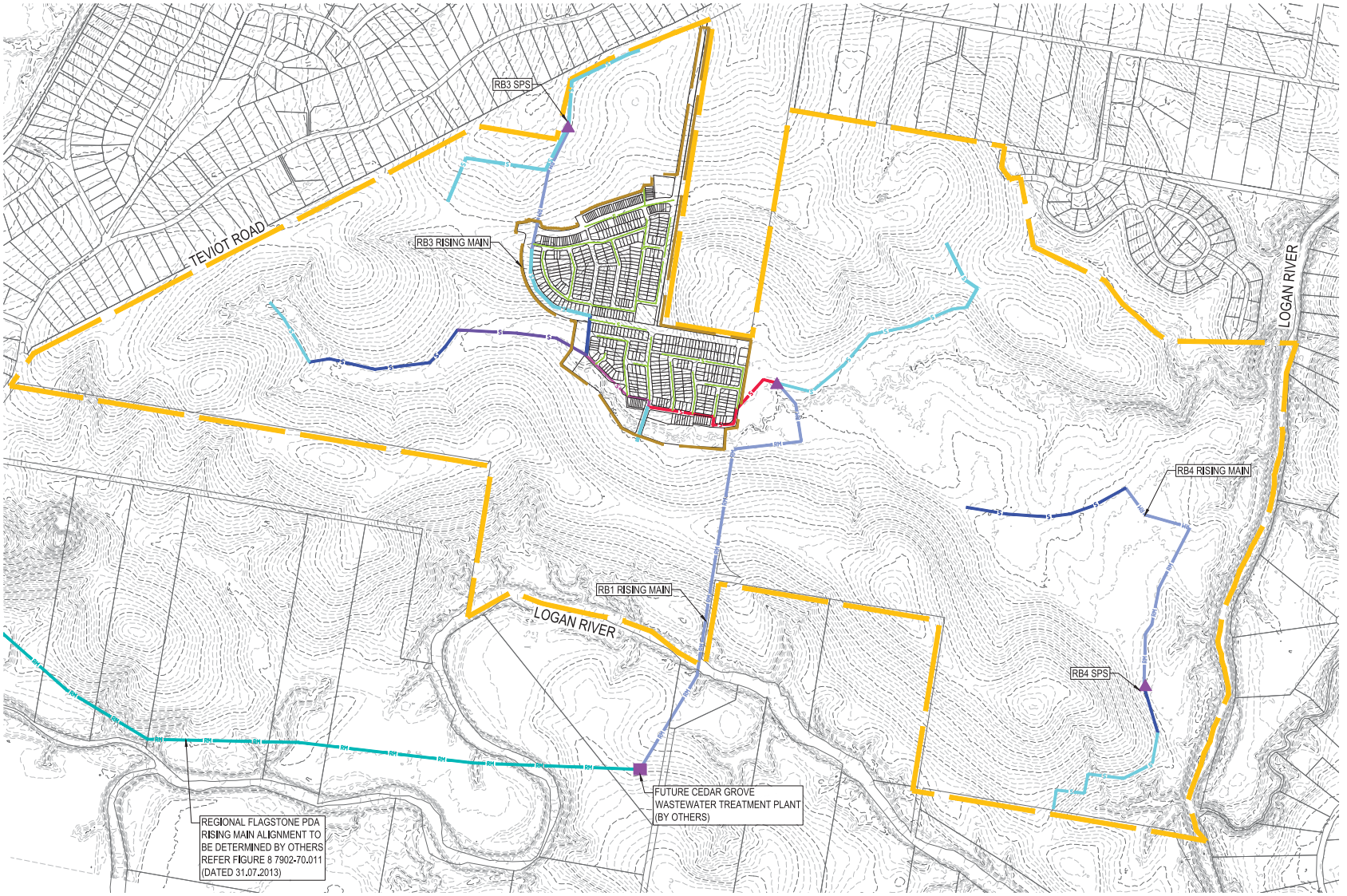
PROJECT No.	DRAWING No.	REVISION
16-003350	301	A



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LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- CADASTRE
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- WASTEWATER TREATMENT PLANT
- INDICATIVE PUMP STATION LOCATION
- PROPOSED DN150 WASTEWATER MAIN
- PROPOSED DN225 WASTEWATER MAIN
- PROPOSED DN300 WASTEWATER MAIN
- PROPOSED DN375 WASTEWATER MAIN
- PROPOSED DN450 WASTEWATER MAIN
- PROPOSED DN525 WASTEWATER MAIN
- PROPOSED WASTEWATER RISING MAIN
- REGIONAL RISING MAIN

NOTES:

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REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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C	06.12.17	ROL 1 UPDATE	RH	JG
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E	21.02.19	UPDATED RISING MAIN ALIGNMENT	JM	JG
F				
G				
H				

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DRAWING TITLE: **PROPOSED ULTIMATE WASTEWATER SERVICE STRATEGY**

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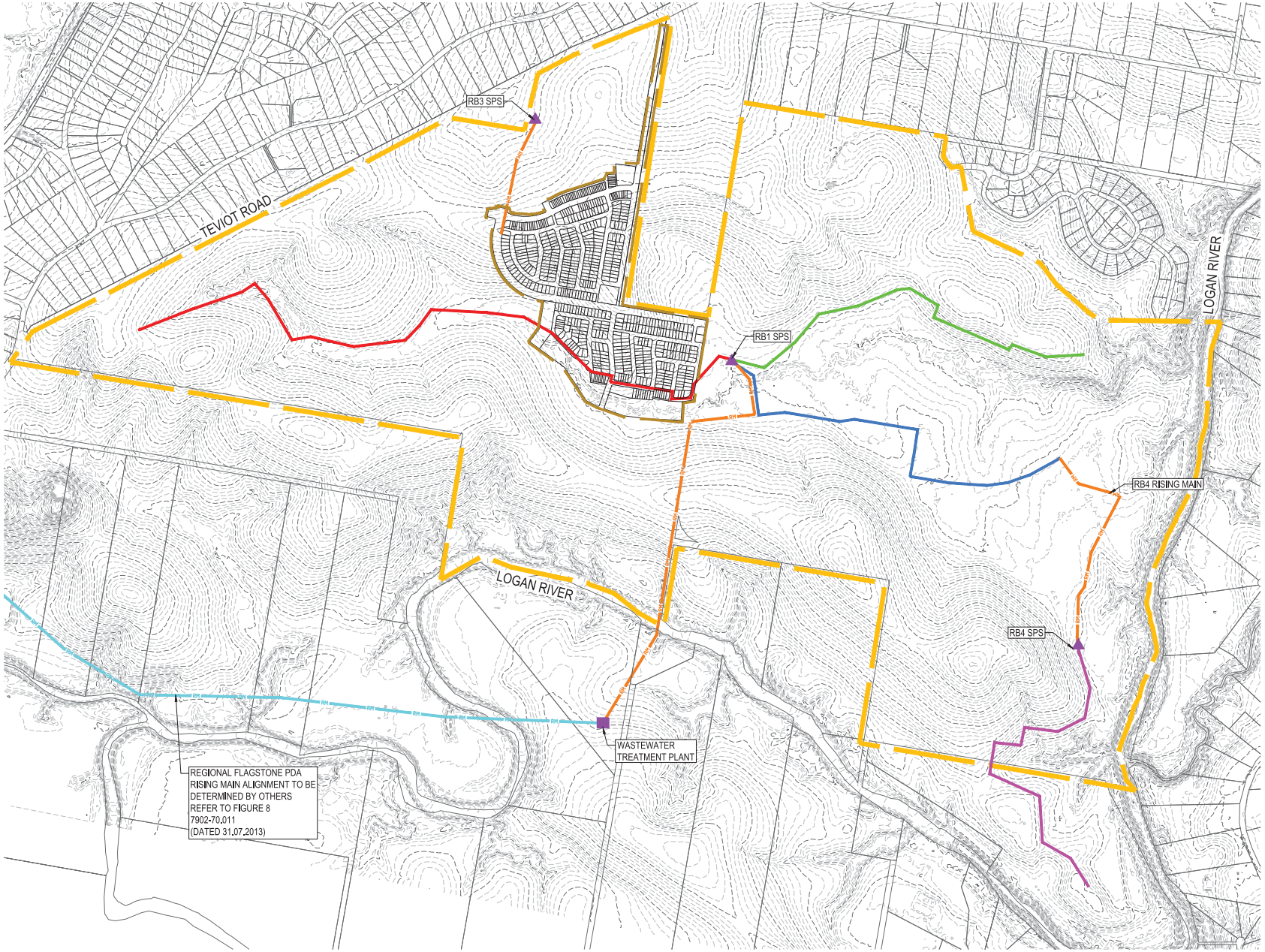
16-003350	302	A



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LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- CADASTRE
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- SEWER GRAVITY MAIN ALIGNMENT 1
- SEWER GRAVITY MAIN ALIGNMENT 2
- SEWER GRAVITY MAIN ALIGNMENT 3
- SEWER GRAVITY MAIN ALIGNMENT 4
- REGIONAL RISING MAIN
- RISING MAIN
- WASTEWATER TREATMENT PLANT
- INDICATIVE PUMP STATION LOCATION

NOTES:

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REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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DRAWING TITLE: PROPOSED TRUNK MAIN INFRASTRUCTURE ALIGNMENT

PROJECT No.	DRAWING No.	REVISION
16-003350	303	A

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Appendix D EDQ – Ultimate Trunk Infrastructure

CELESTINO PTY LTD

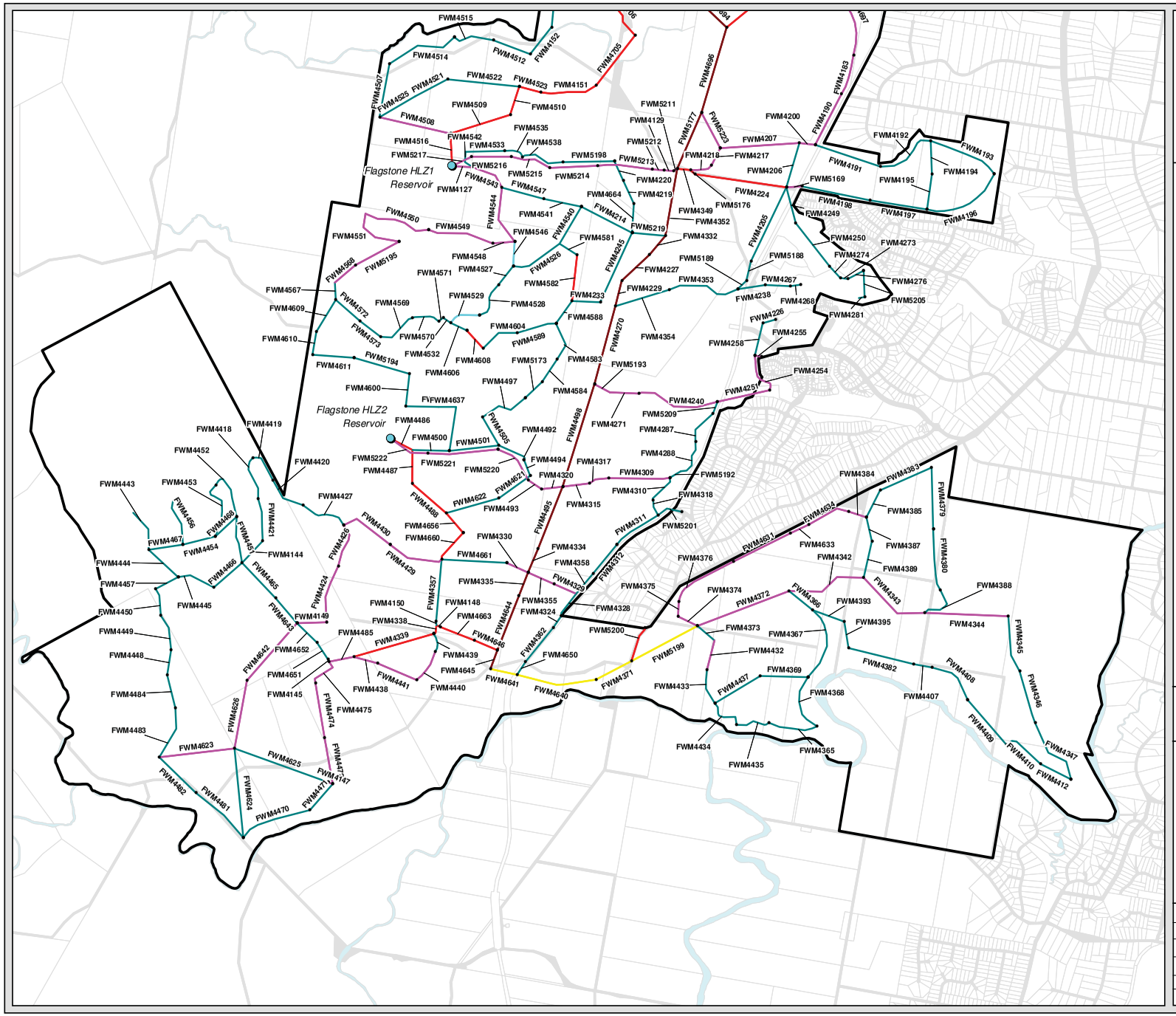


Figure 6
Water Supply Map South

LEGEND

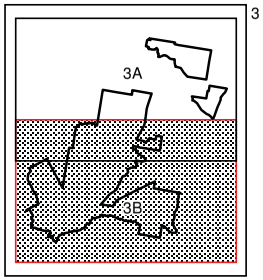
N
W E
S

- Reservoir
- Node
  (Infrastructure start / end point)

Water Supply Mains
(Diam)

- 250mm
- 300mm
- 375mm
- 450mm
- 475mm
- 600mm

Sheet Index



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Scale: 1:35,000

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Level 11, North Tower, 515 St Pauls Terrace
Locked Bag 4006, Fortitude Valley QLD 4006
Tel: 07 3369 9822 Fax: 07 3369 9722

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SHEET	A3
Project No:	7902-70.011
Date:	17/07/2013
Revision Number:	
Designed by:	
Client Name:	EDQ

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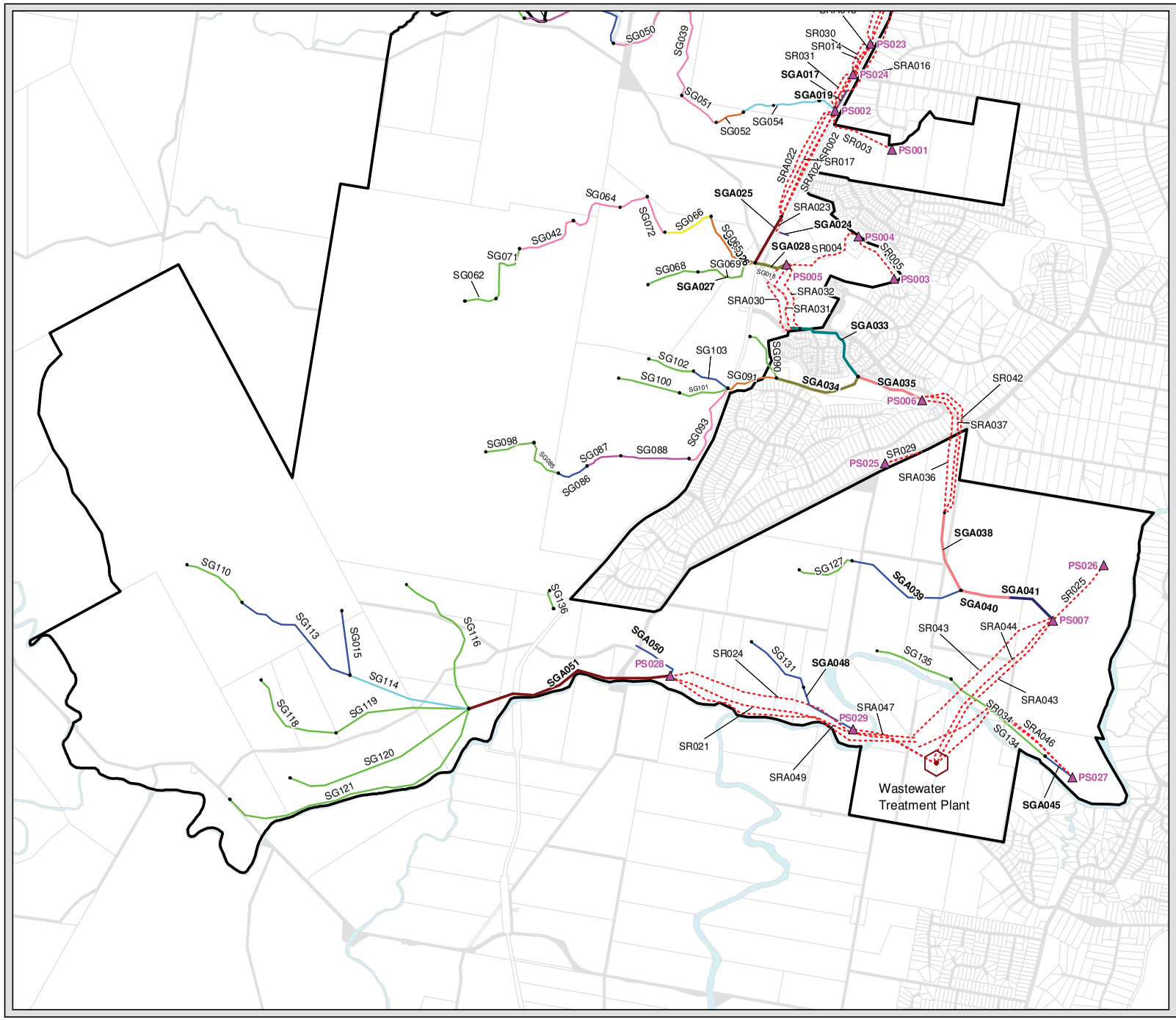


Figure 8
Sewerage Map South

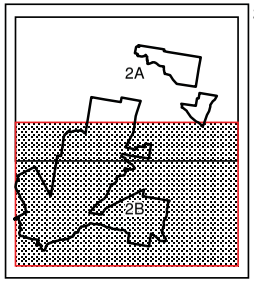
Pump Station
Node
(Infrastructure start / end point)
Wastewater
Treatment Plant

Proposed Gravity Mains (Diam)

300	600
375	675
400	750
450	800
500	900
525	1000
	1200

Rising Main
Development Area
(PDA) Boundary

Sheet Index



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Metres
Scale: 1:35,000

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Date:	31/07/2013
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Designed by:	
Client Name:	EDQ

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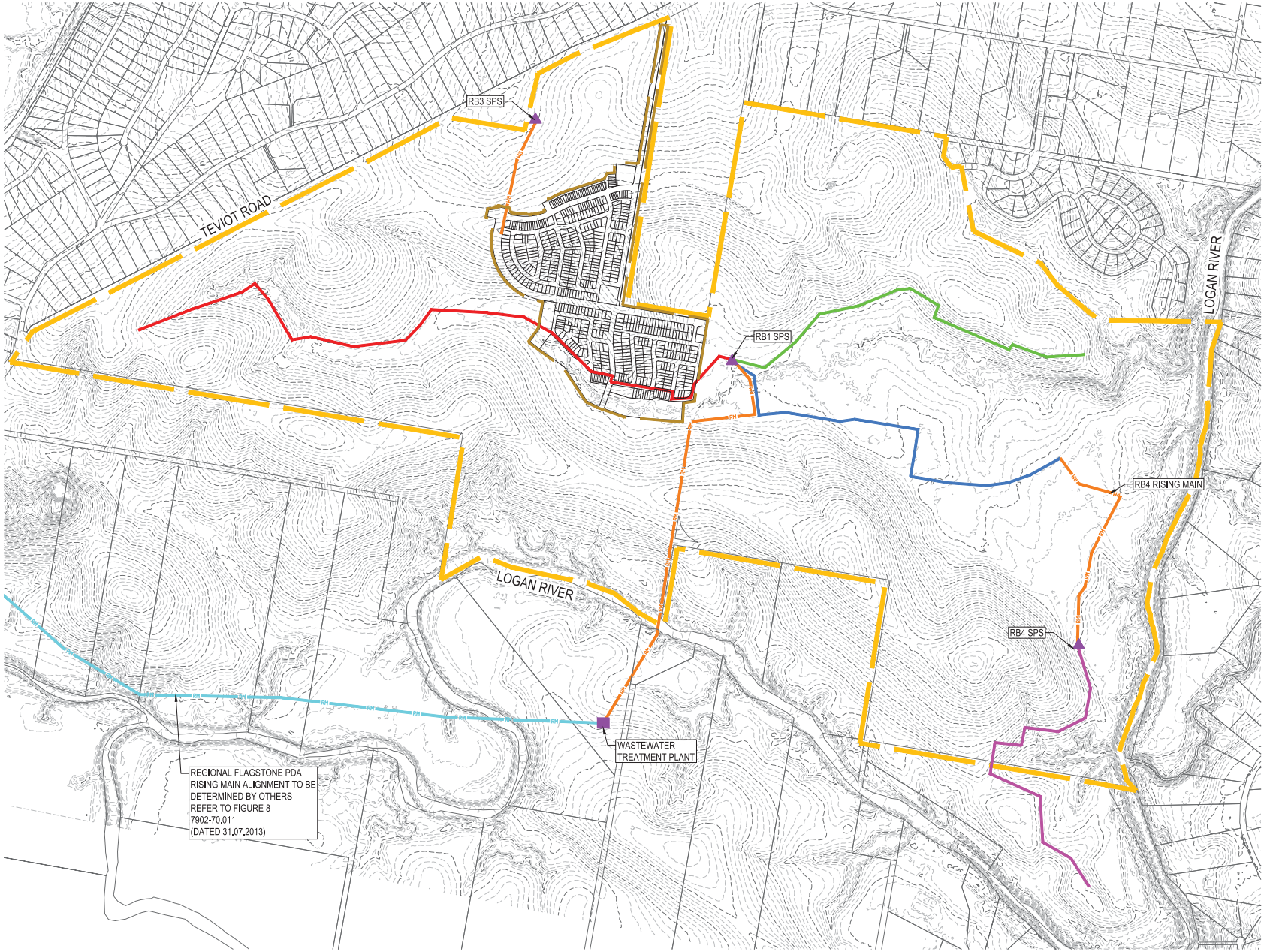
ROL 1 JIMBOOMBA CELESTINO

Appendix E Conceptual Trunk Gravity Long Section Design

CELESTINO PTY LTD



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LEGEND:

- MASTER PLAN BOUNDARY
- STAGE 1 SITE BOUNDARY
- CADASTRE
- PROPOSED LOT BOUNDARY
- EXISTING CONTOURS
- SEWER GRAVITY MAIN ALIGNMENT 1
- SEWER GRAVITY MAIN ALIGNMENT 2
- SEWER GRAVITY MAIN ALIGNMENT 3
- SEWER GRAVITY MAIN ALIGNMENT 4
- REGIONAL RISING MAIN
- RISING MAIN
- WASTEWATER TREATMENT PLANT
- INDICATIVE PUMP STATION LOCATION

NOTES:

1. WASTEWATER SERVICING STRATEGIES PROPOSED ARE CONCEPTUAL AND REMAIN SUBJECT TO DETAILED ENGINEERING DESIGN.
2. PROPOSED INFRASTRUCTURE ALIGNMENTS, DIAMETERS AND MATERIALS ARE SUBJECT TO DEVELOPMENT DENSITIES AND WILL BE CONFIRMED DURING DETAILED DESIGN.
3. DESIGN INFORMATION SHOWN ON PLAN IS INDICATIVE ONLY AND HAS BEEN GENERATED FROM DCDB, LIDAR CONTOURS, COUNCIL AND DIAL BEFORE YOU DIG RECORDS. FINAL DESIGN TO BE CONFIRMED AND WILL BE SUBJECT TO DETAILED SURVEY AND POT-HOLING.

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
A	07.11.17	DRAFT	RH	JG
B	06.12.17	ROL 1 UPDATE	RH	JG
C	21.02.19	ROL 1 UPDATE	JM	JG
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F				
G				
H				

DRAWN

DESIGN

STATUS: NOT FOR CONSTRUCTION

APPROVED

SCALE: 1:7500 150 0 150 300 A1; 1:15000 A3

CLIENT: 1:7500 1:15000

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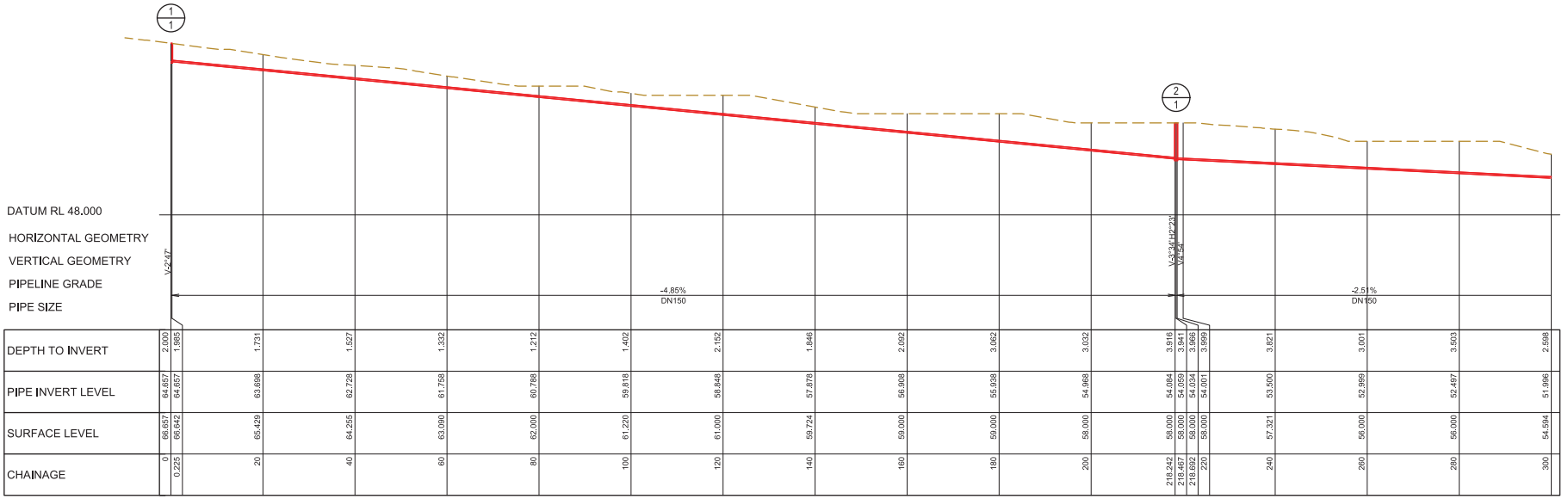
DRAWING TITLE: PROPOSED TRUNK MAIN INFRASTRUCTURE ALIGNMENT

PROJECT No.	DRAWING No.	REVISION
16-003350	303	A



W SGM01 - LONGITUDINAL SECTION (1 of 10)

W SGM01 - LONGITUDINAL SECTION (2 OF 10)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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C				
D				
E				
F				
G				
H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:2000 A3
VERTICAL

CLIENT: Riverside CELESTINO

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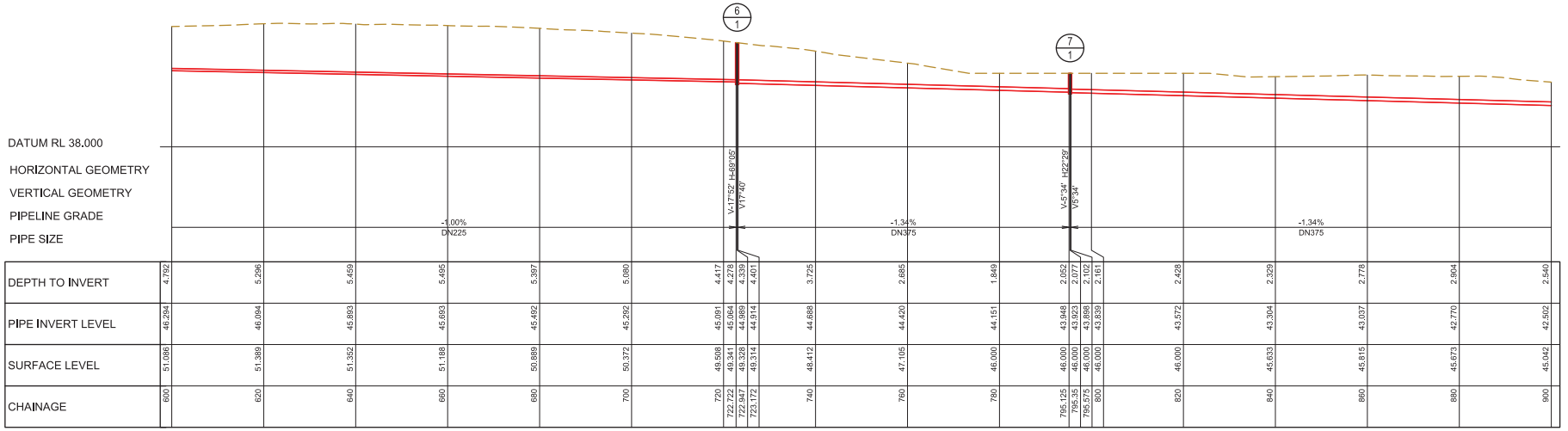
DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 1 LONG SECTIONS**

PROJECT No.	DRAWING No.	REVISION
16-003350	320	A



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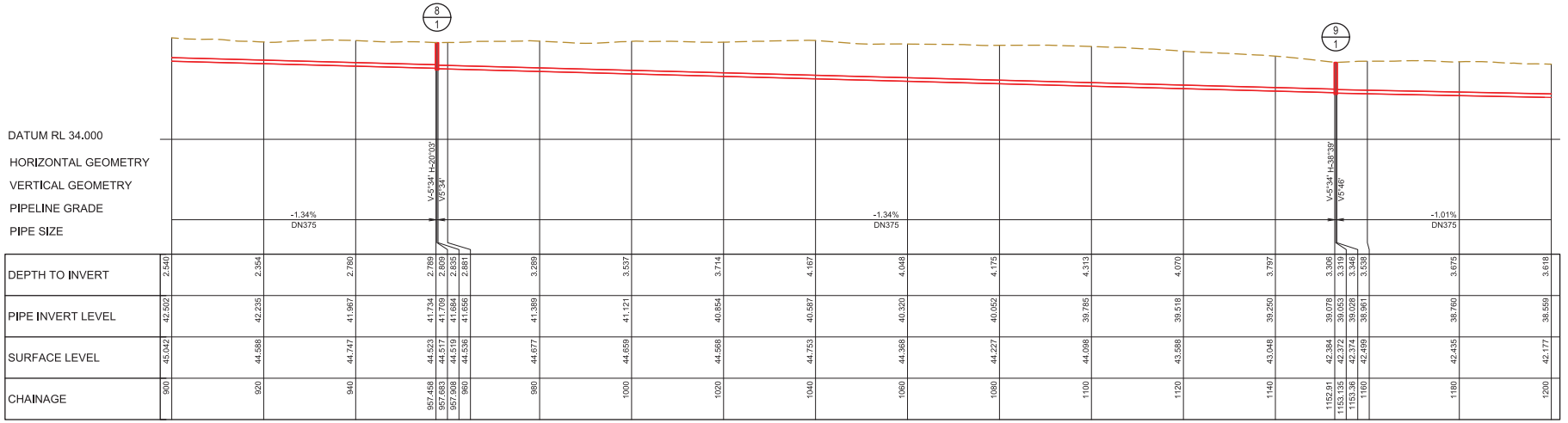

W SGM01 - LONGITUDINAL SECTION (4 OF 10)

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C				
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H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:1000 1:2000 20 10 0 20 40 A1 A3 VERTICAL

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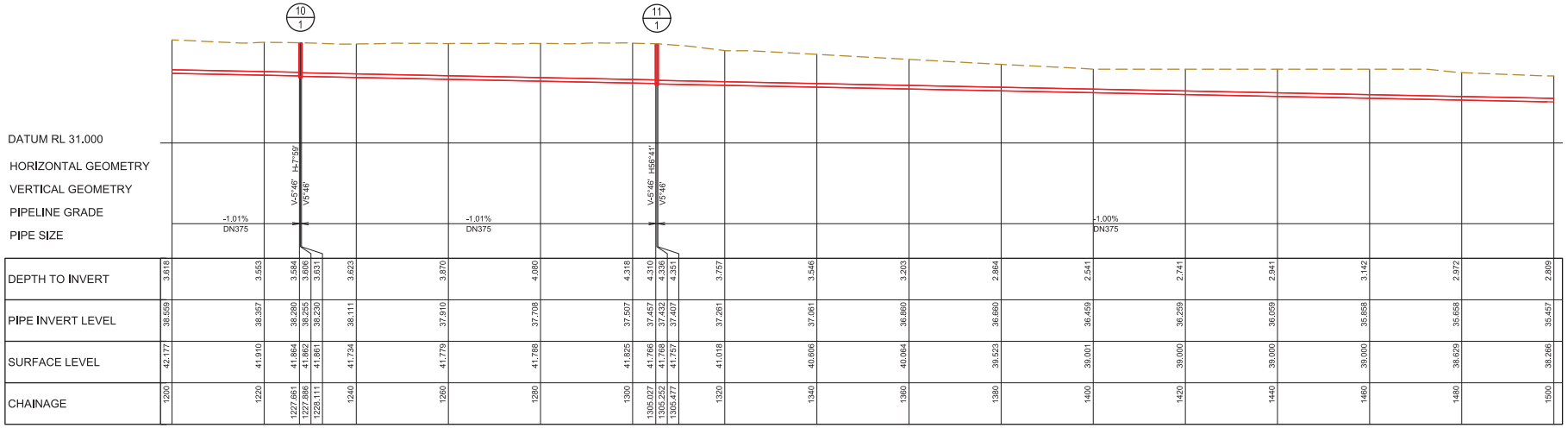
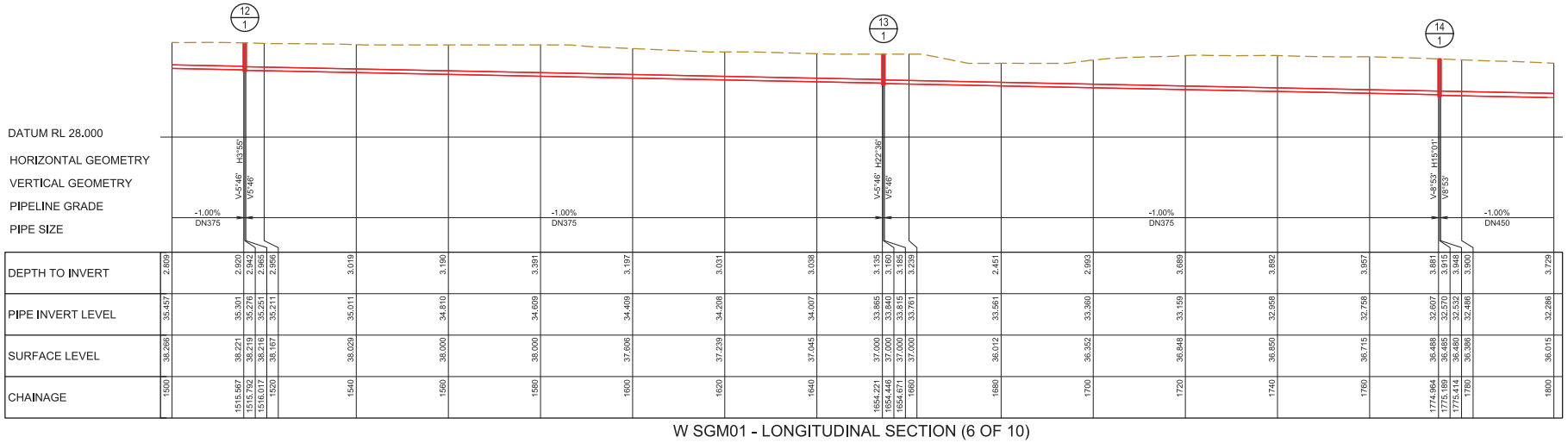
DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 1 LONG SECTION**

PROJECT No.	DRAWING No.	REVISION
16-003350	321	A



W SGM01 - LONGITUDINAL SECTION (5 OF 10)



W SGM01 - LONGITUDINAL SECTION (6 OF 10)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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B				
C				
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H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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HORIZONTAL

1:1000 20 10 0 20 40 A1
1:2000 A3
VERTICAL

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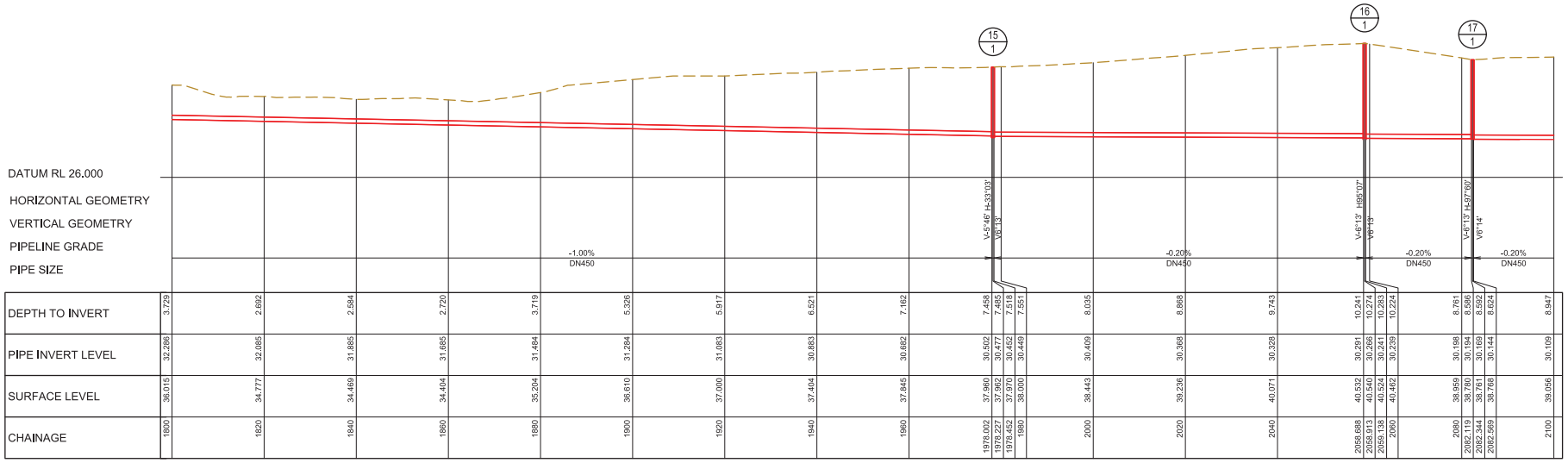
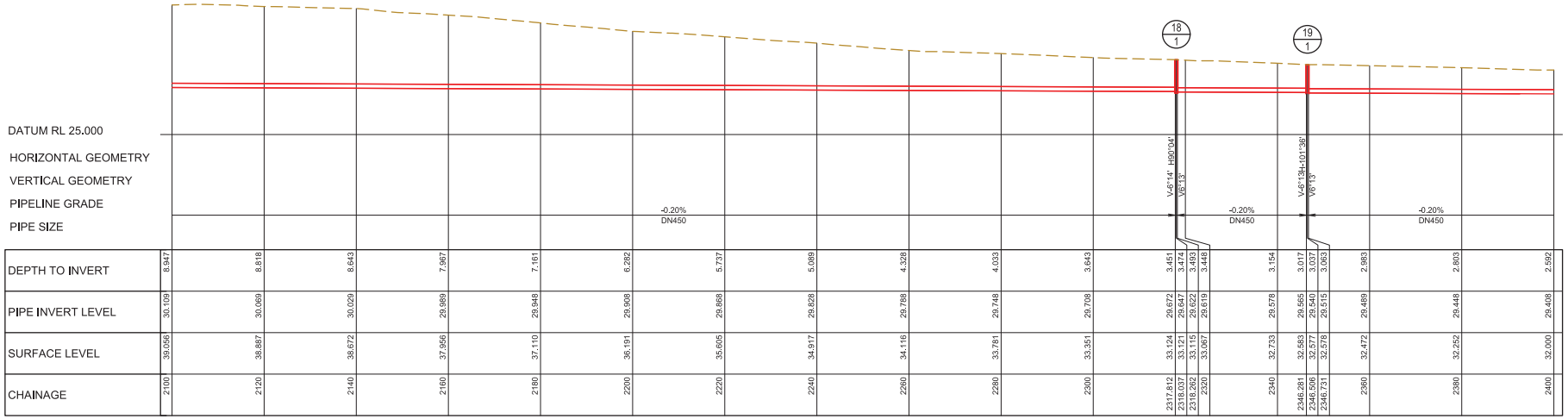
DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 1 LONG SECTION**

PROJECT No.	DRAWING No.	REVISION
16-003350	322	A



W SGM01 - LONGITUDINAL SECTION (7 OF 10)



W SGM01 - LONGITUDINAL SECTION (8 OF 10)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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C				
D				
E				
F				
G				
H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:1000 1:2000 20 10 0 20 40 A1 A3 VERTICAL

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DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 1 LONG SECTION**

PROJECT No.	DRAWING No.	REVISION
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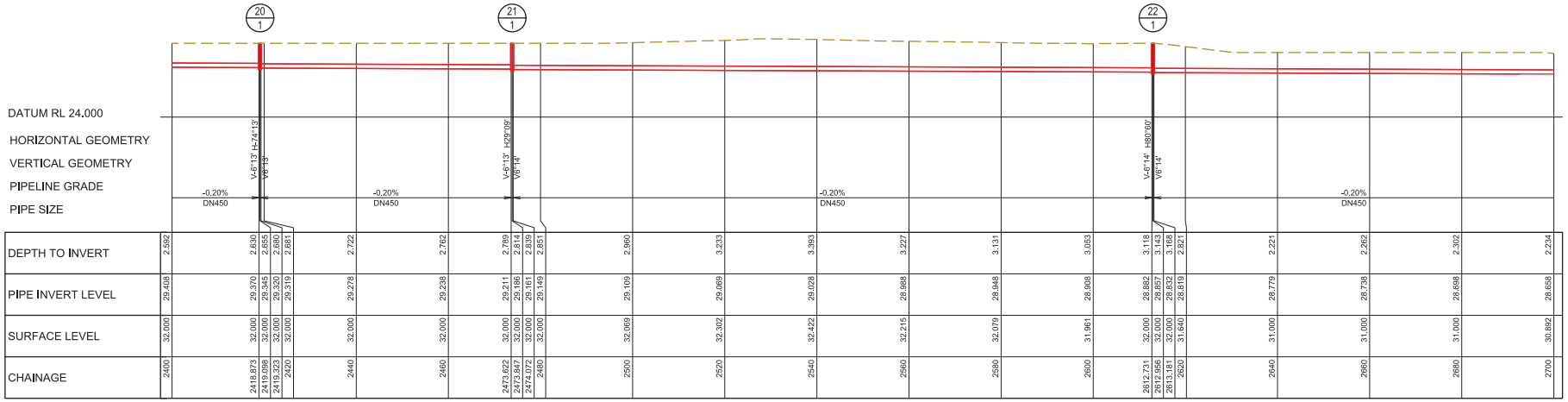
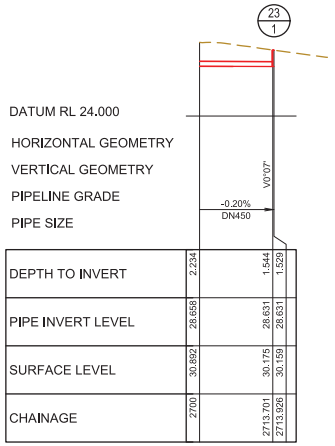


W SGM01 - LONGITUDINAL SECTION (9 OF 10)



W SGM01 - LONGITUDINAL SECTION (10 OF 10)

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APPROVED

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HORIZONTAL
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VERTICAL

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PROJECT No.	DRAWING No.	REVISION
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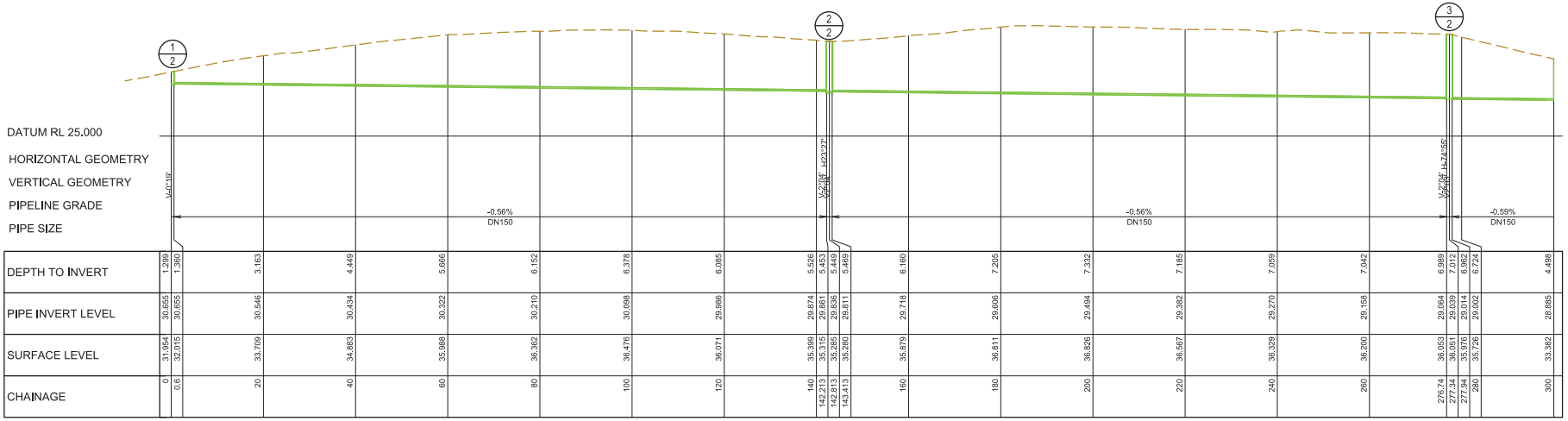
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2/2
3/2
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VERTICAL GEOMETRY
PIPELINE GRADE
PIPE SIZE
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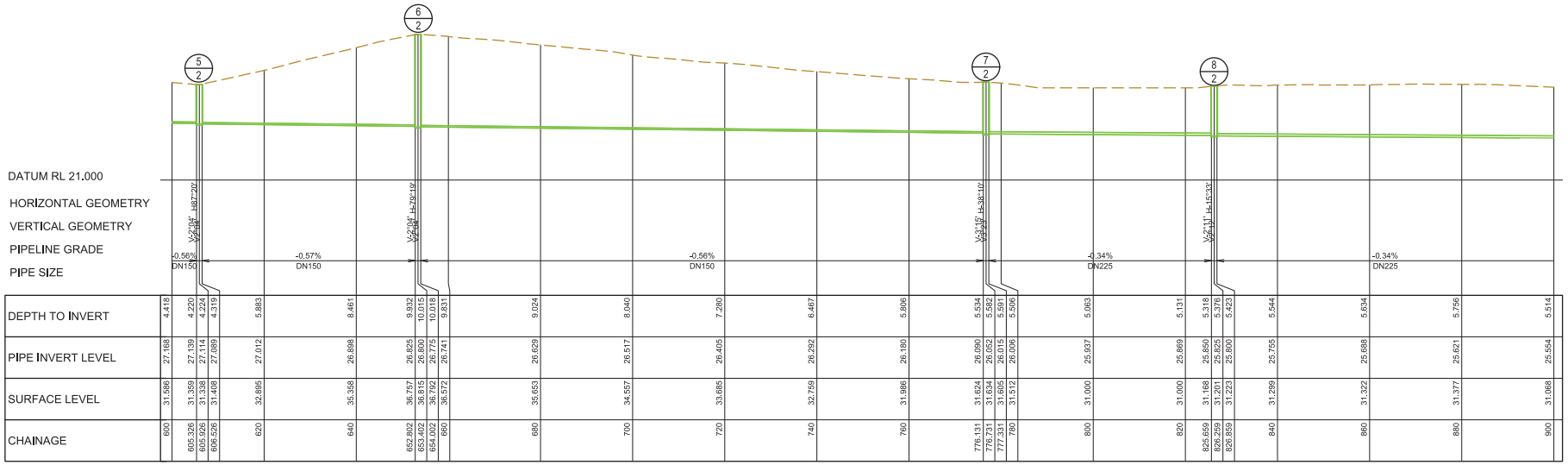
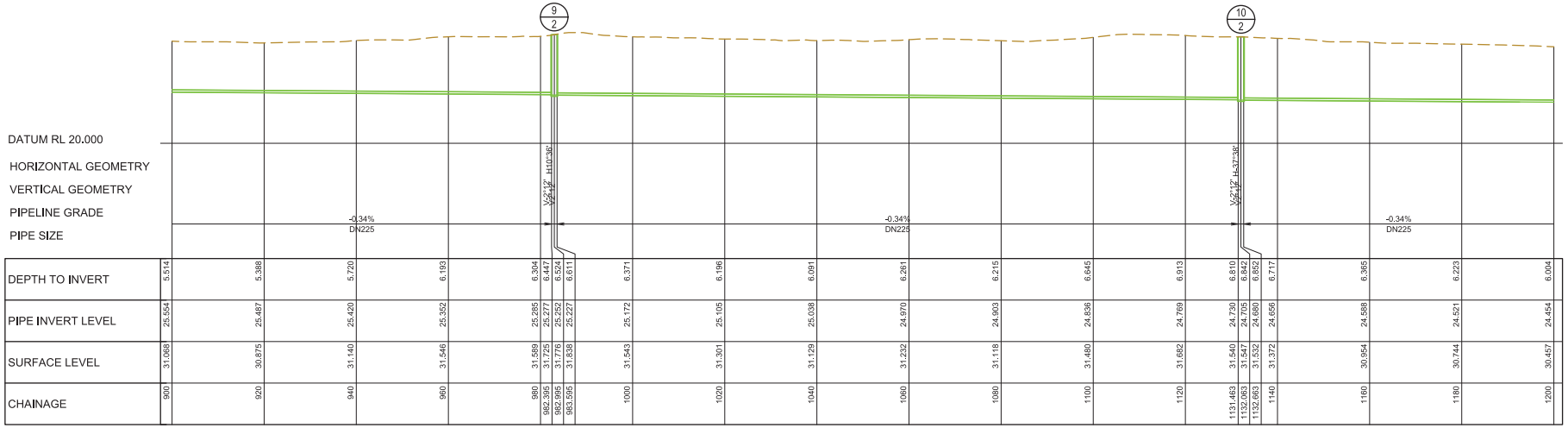
V=2.104 H=23.277
V=2.104 H=24.455
-0.56%
DN150
-0.56%
DN150
-0.59%
DN150

| | | |



W SGM02 - LONGITUDINAL SECTION (3 OF 6)



W SGM02 - LONGITUDINAL SECTION (4 OF 6)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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C				
D				
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DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:4000 A3
HORIZONTAL

1:1000 20 10 0 20 40 A1
1:2000 A3
VERTICAL

CLIENT: Riverside CELESTINO

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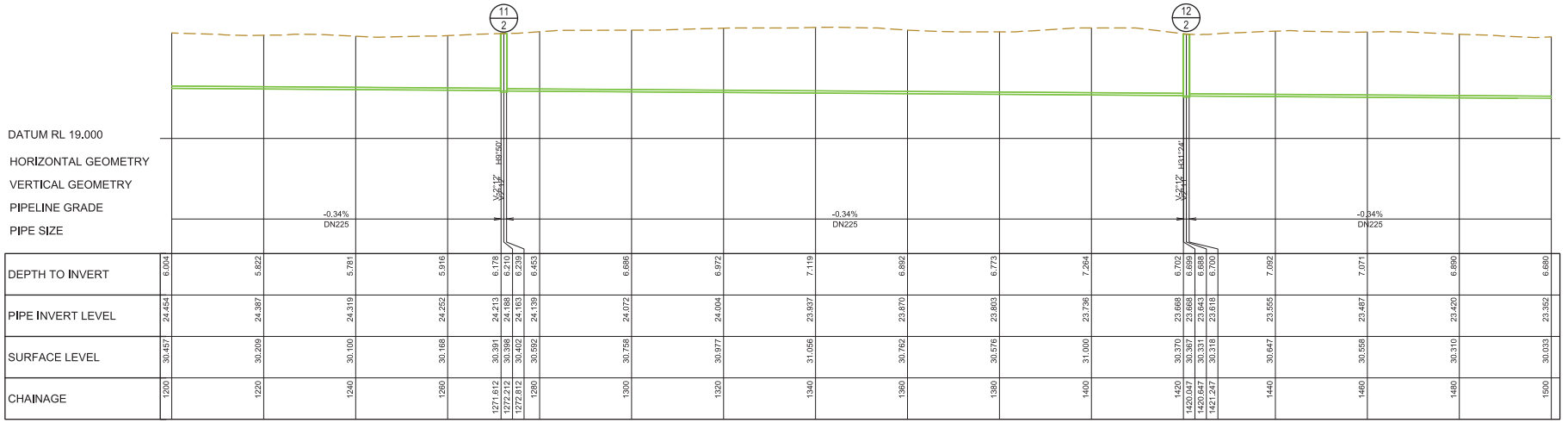
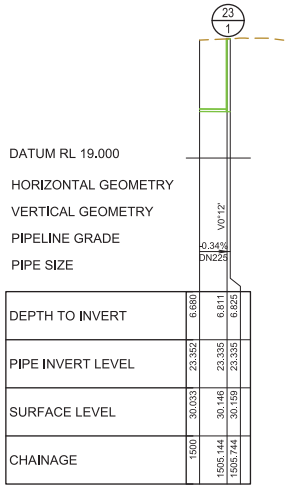
DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 2 LONG SECTION**

PROJECT No.	DRAWING No.	REVISION
16-003350	331	A



W SGM02 - LONGITUDINAL SECTION (5 OF 6)



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H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE
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HORIZONTAL
1:1000 20 10 0 20 40 A1
1:2000 A3
VERTICAL

CLIENT: Riverside CELESTINO

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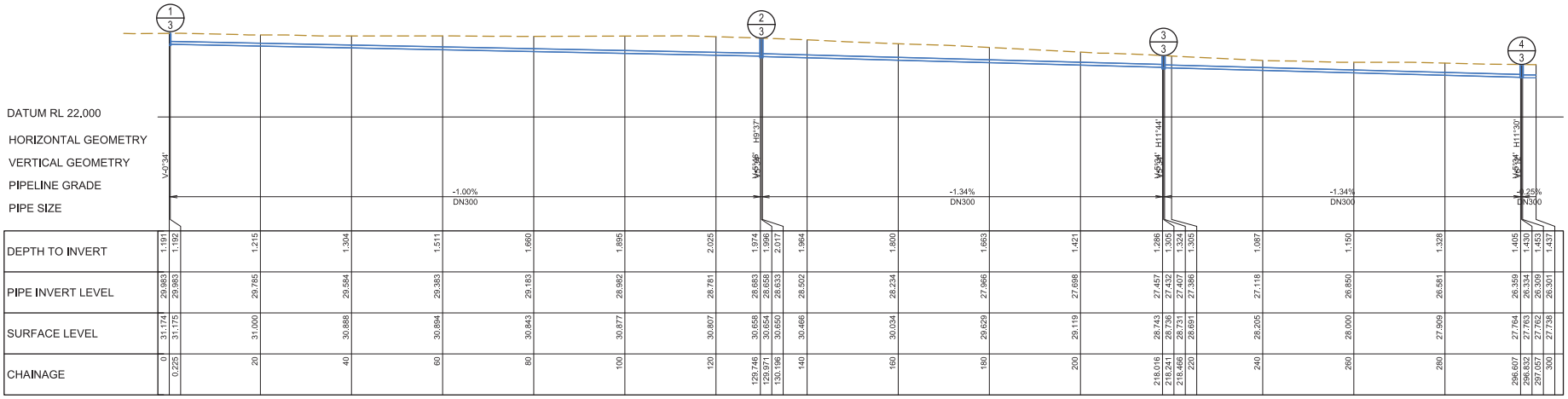
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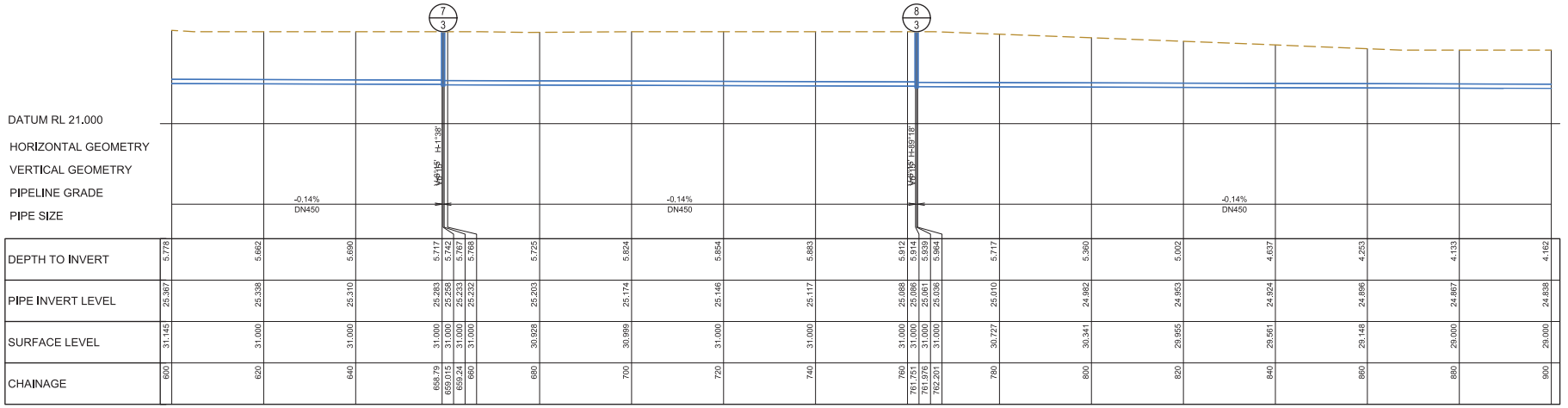
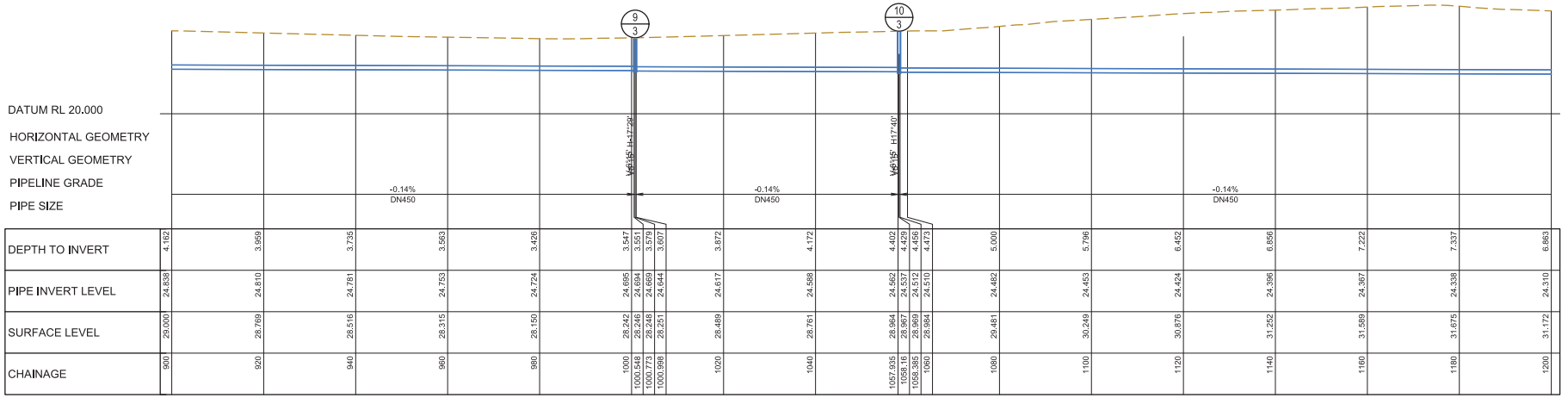
2/3
3/3
4/3
V-0°34'
V=5196
H9°37'
V=5194
H11°44'
V=5194
H11°30'
-1.00%
DN300
-1.34%
DN300
-1.34%
DN300
-1.25%
DN300

| | | |



W SGM03 - LONGITUDINAL SECTION (3 OF 6)



W SGM03 - LONGITUDINAL SECTION (4 OF 6)

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B				
C				
D				
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DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:1000 1:2000 20 10 0 20 40 A1 A3 VERTICAL

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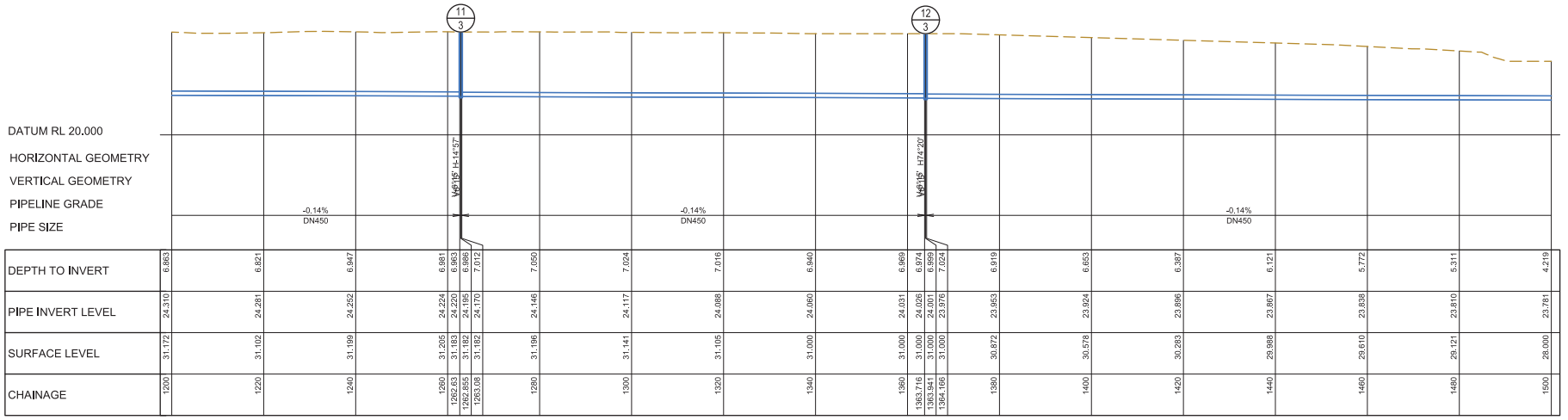
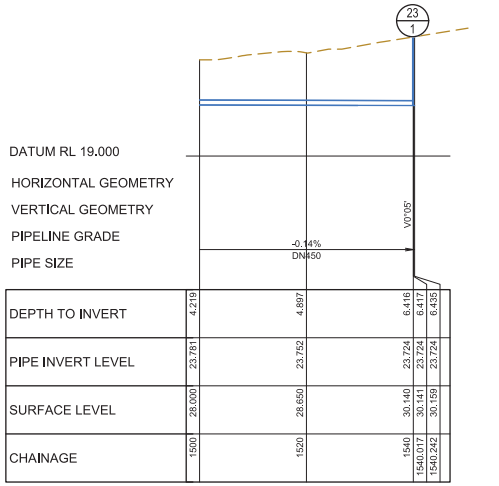
16-003350	341	A



W SGM03 - LONGITUDINAL SECTION (5 OF 6)



W SGM03 - LONGITUDINAL SECTION (6 OF 6)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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B				
C				
D				
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DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:4000 A3
HORIZONTAL

1:1000 20 10 0 20 40 A1
1:2000 A3
VERTICAL

CLIENT: Riverside CELESTINO

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DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 3 LONG SECTION**

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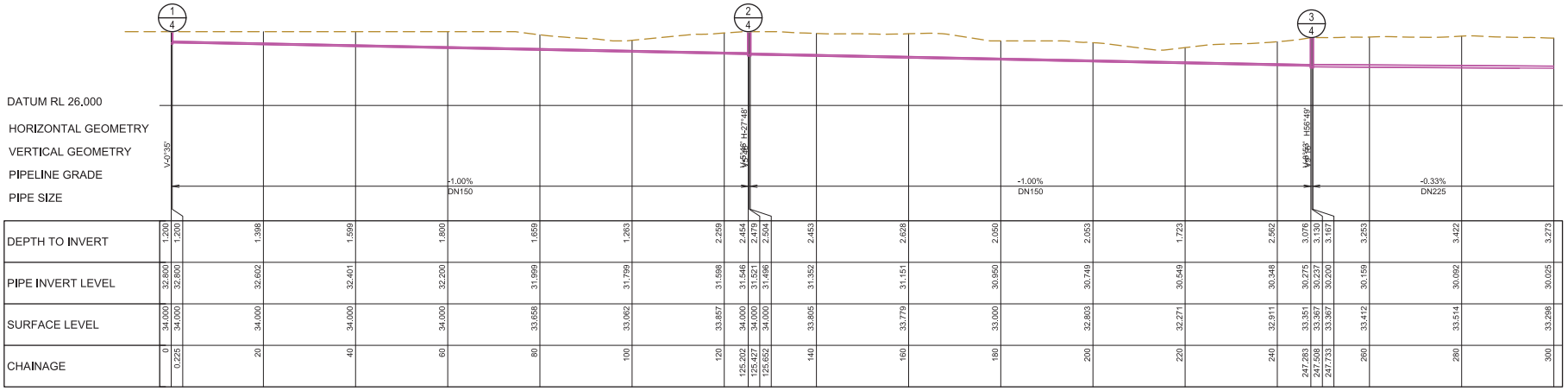
16-003350	342	A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4
2
4
3
4
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VERTICAL GEOMETRY
PIPELINE GRADE
PIPE SIZE
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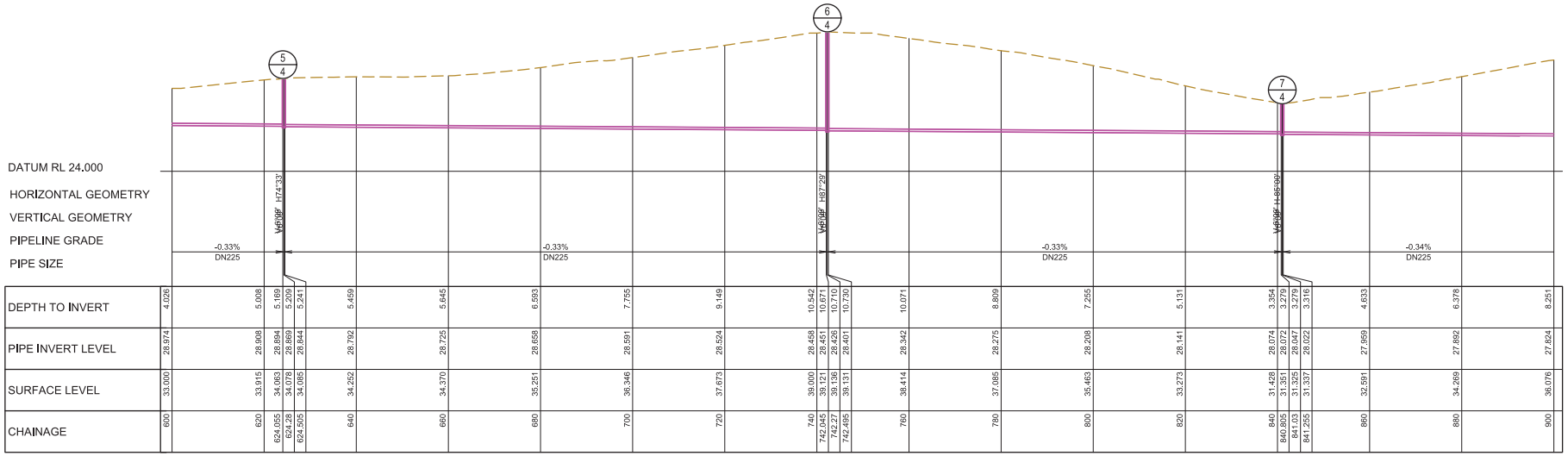
V=5.46 H=27.48°
V=4.15 H=56.49°
-1.00%
DN150
-1.00%
DN150
-0.33%
DN225

| | | |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4
6
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4
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VERTICAL GEOMETRY
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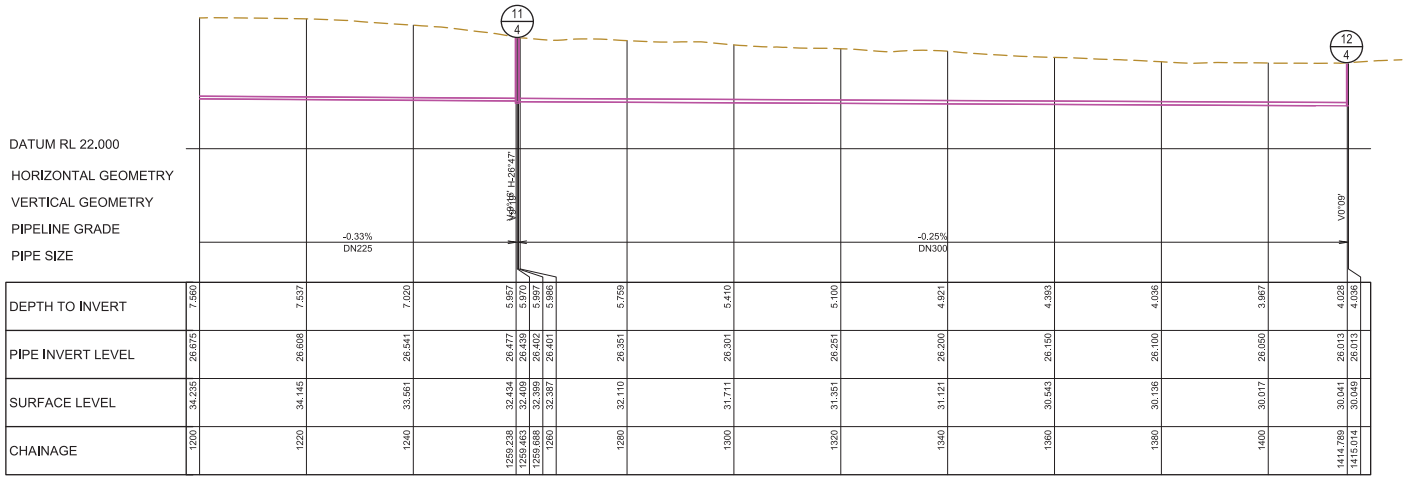
| | | |



W SGM04 - LONGITUDINAL SECTION (5 OF 5)

REVISION	DATE	ISSUE DETAILS	DRAWN	DESIGN
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B				
C				
D				
E				
F				
G				
H				

DRAWN

DESIGN

STATUS: **NOT FOR CONSTRUCTION**

APPROVED

SCALE

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1:4000 A3
HORIZONTAL

1:1000 20 10 0 20 40 A1
1:2000 A3
VERTICAL

CLIENT: Riverside CELESTINO

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DRAWING TITLE: **SEWER GRAVITY MAIN ALIGNMENT 4 LONG SECTION**

PROJECT No.	DRAWING No.	REVISION
16-003350	352	A

ROL 1 JIMBOOMBA CELESTINO

Appendix F LWIA RB1 Planning and Design Report

CELESTINO PTY LTD

LOGAN
WATER INFRASTRUCTURE
ALLIANCE

Proudly

LOGAN
CITY COUNCIL

DETAILED PLANNING & PRELIMINARY DESIGN REPORT

RB1, RB2-2 AND FL-1 WASTEWATER PUMP STATIONS TO CEDAR GROVE WWTP

TASK NUMBER: LS-014

SEPTEMBER 2017

Approval Register

	Date
Submitted by Project Manager	13/9/2017
LoganWIA Internal Review	25/9/2017
Editorial Review	
Program Review	

Controlled Document – Change Register

Revision	Section Changed	Change Description	Initial	Date
A	All	Submission of draft report	DS, BL, LM	13/9/2017
B	All	Preparation of draft report for program review	AH	18/9/2017
C				
D				
E				
1				
2				
3				
4				
5				

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ACRONYMS AND ABBREVIATIONS

ADWF	Average Dry Weather Flow
AHD	Australian Height Datum
ASS	Acid Sulphate Soils
CAPEX	Capital Expenditure
CWP	Capital Works Program
DN	Diameter Nominal
DSS	Desired Standards of Service
EP	Equivalent Population
LoganWIA	Logan Water Infrastructure Alliance
NPC	Net Present Cost
OD	Outside Diameter
OPEX	Operational Expenditure
PDA	Priority Development Area
PDWF	Peak Dry Weather Flow
PWWF	Peak Wet Weather Flow
SEQ	South East Queensland
SMH	Sewerage Maintenance Hole
SPS	Sewerage Pump Station
WWTP	Wastewater Treatment Plant
VSD	Variable Speed Drive

7. DETAILED PLANNING: RB1 PUMPING STATION

This chapter describes the detailed planning for the RB1 pumping station and rising main. Based on Option 1 being the preferred strategy to transfer load from Jimboomba to Cedar Grove, the design basis for the RB1 pump station includes the following items:

- Pump capacity and rising main to be sized for local catchment loads only (i.e. excludes contribution from the Jimboomba WWTP catchment)
- RB1 pumps directly to the inlet works of the Cedar Grove WWTP.

7.1 Approved population forecasts

The approved equivalent population projections for the RB1 catchment are listed in Table 7-1. The values demonstrate that 75% of the expected growth is forecast to occur post 2051.

Table 7-1: RB1 catchment equivalent population projections

Parameter	2021	2026	2031	2041	2051	Ultimate
EP	737	938	1,156	3,397	5,015	20,273
ADWF (L/s)	1.4	1.8	2.2	6.5	9.6	38.7
PWWF (L/s)	9	11	13	39	58	235

7.2 Location options

Four location options for the pumping station have been considered, each of these is illustrated in Figure 7-1. Options 1, 2 and 4 are located on the eastern side of the Bushland Road road reserve (currently an unformed road), close to the proposed first stage of development. Option 3 represents the location which was nominated in the *Logan South Wastewater Servicing Strategy* (Cardno, 2016).

All four locations share the following attributes:

- Located on Lot 800 on SP247625, owned by EJC Riverbend
- Located on the edge of the Q100 flood level.

To determine a preferred location for the pumping station consideration has been given to the potential layout of the municipal sewer network. The future layout of the gravity sewer network was developed based on the natural topography of the catchment. Allowance has been made for two municipal pumping stations in the catchment (PS4 and PS5 shown on Figure 7-1). An indicative layout of the municipal infrastructure for each option is provided in Appendix E.

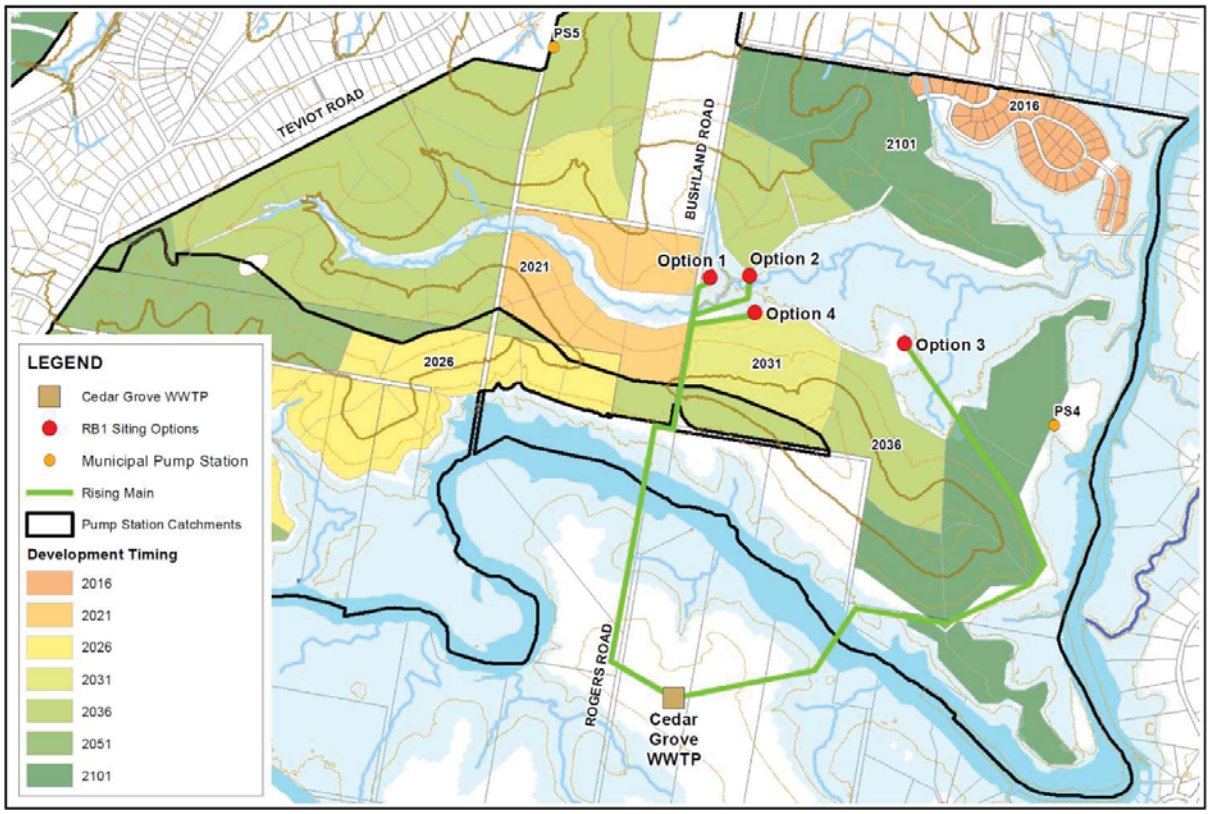


Figure 7-1: RB1 location options

7.2.1 Infrastructure requirements

The pumping station features and rising main details required for each option are listed in Table 7-2. The infrastructure sizing is based on the following assumptions:

- RB1 wet well to be constructed in a single stage sized for ultimate development
- RB1 rising main to be constructed in two stages, with Stage 1 sized for 2051 PWWF and Stage 2 sized for ultimate PWWF.

The staging of the wet well and rising main is discussed further in Section 7.4.

Table 7-2: RB1 pumping station features and rising main details

Asset	Parameter	Option 1	Option 2	Option 3	Option 4
Pumping station	Wet well diameter (m)	4.5	4.5	4.5	4.5
	Depth of incoming gravity sewer (m)	8.0	7.2	7.9	7.6
	Total depth (m)	11.1	10.2	11.0	10.7
	Total installed power for Stage 1 (kW)	51	52	52	52
	Total installed power for Stage 2 (kW)	186	188	212	188
Rising main	Stage 1 diameter	DN200	DN200	DN225	DN200
	Stage 2 diameter	DN375	DN375	DN375	DN375
	Length (m)	1,936	2,081	2,909	1,981

Key points to note are that the depth of the well for all locations will be similar (10 to 11 m deep). The depth is governed by the need for the sewers to traverse a low-lying area in the middle of the catchment.

Location options 1, 2 and 4 require a DN200 rising main to service the catchment up to 2051, a slightly larger rising main is required in Option 3 to reduce the pressure drop in the rising main. Each location option will require a DN375 rising main post 2051.

7.3 Options analysis

This section presents the analysis undertaken to determine a preferred location. The analysis includes a comparison of capital and operational costs, NPC calculation, and risks and opportunities including environment and approvals assessments.

7.3.1 Cost analysis

The capital costs and NPC for each location option are summarised in Table 7-3. For comparative purposes, the costs include the municipal gravity sewers so that a whole of catchment servicing cost is captured. The operating and maintenance costs include the energy and maintenance costs for the pumping station and rising main.

Considering the undeveloped nature of the catchment at present, each location would require a new access road to be constructed if built today. The costs for an access road is not included in this assessment, as it is fair to assume that these would be provided by the developer as part of the normal growth within the catchment.

Table 7-3: Capital ($2017) and NPC comparison

	Option 1	Option 2	Option 3	Option 4
CAPITAL COST				
Municipal gravity mains	$3.26	$3.09	$2.99	$2.70
RB1 rising main	$3.16	$3.37	$4.70	$3.46
RB1 pump station	$2.65	$2.61	$2.76	$2.65
Total	**$9.07**	**$9.08**	**$10.45**	**$8.80**
Ranking	**2**	**3**	**4**	**1**
NPC				
Municipal gravity mains	$1.90	$1.85	$2.38	$1.59
RB1 rising main	$1.36	$1.45	$2.07	$1.49
RB1 pump station	$1.65	$1.60	$1.69	$1.63
Capital NPC	**$4.91**	**$4.91**	**$6.14**	**$4.71**
Operation and maintenance NPC	**$1.10**	**$1.08**	**$1.17**	**$1.10**
Total NPC	**$6.02**	**$5.99**	**$7.31**	**$5.81**
Ranking	**3**	**2**	**4**	**1**
Difference ($)	**$0.20**	**$0.18**	**$1.50**	**-**
Difference (%)	**4%**	**3%**	**26%**	**-**

The cost comparison clearly showed that Option 3 is the most expensive capital and NPC option, this is primarily due to:

- The rising main being approximately 900 metres longer compared to the other locations
- The location is furthest from the first stage of development, and therefore requires bringing forward additional municipal gravity sewers to connect it to the development

For these reasons Option 3 was eliminated from further consideration.

For the other locations there is a minimal difference in NPC; Option 4 is slightly lower due to efficiencies achieved in the layout of the municipal gravity sewer network. There is negligible difference between Options 1, 2 and 4 for the pumping station and rising main costs alone.

7.3.2 Risks and opportunities

This section will describe the risks and opportunities that each location presents.

Access

The indicative location for the future road network within the development is illustrated in Figure 7-2 . At location 2 and 4, the pumping station is adjacent to a future road while for location 1 the pumping station is approximately 170 m south of the future road linking the eastern and western parts of the development. Therefore, siting the pumping station at location 1 is likely to incur additional costs to Council for the provision of an access road.

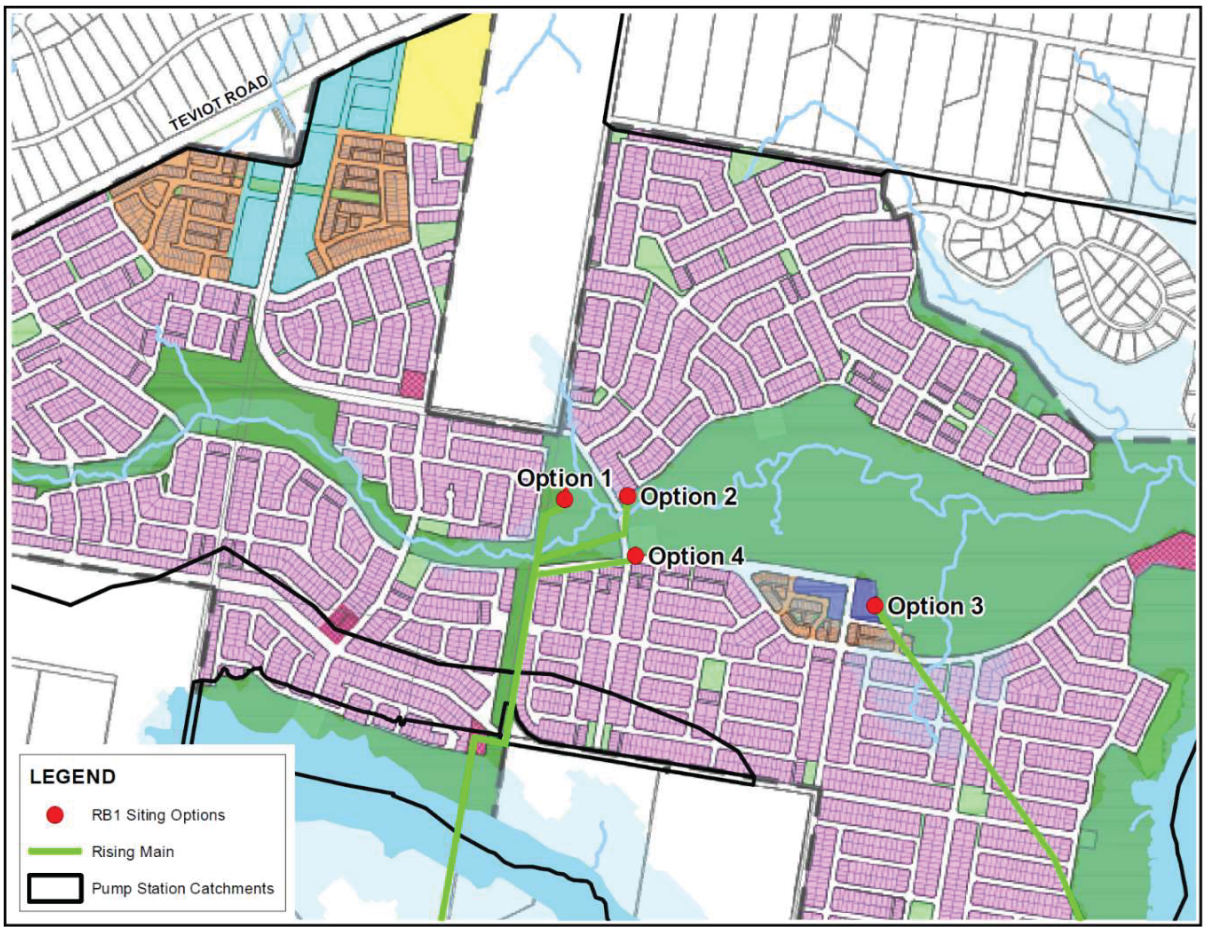


Figure 7-2: Proposed development plan within the RB1 catchment

Property approvals

The properties affected by the proposed rising alignments are shown in Figure 7-3 and described in Table 7-4. The proposed alignment for the RB1 rising main deviates at Lot 104 on SP254145 to avoid impacting property owned by Seqwater on the southern side of the Logan River (Lot 155 on WD3050).

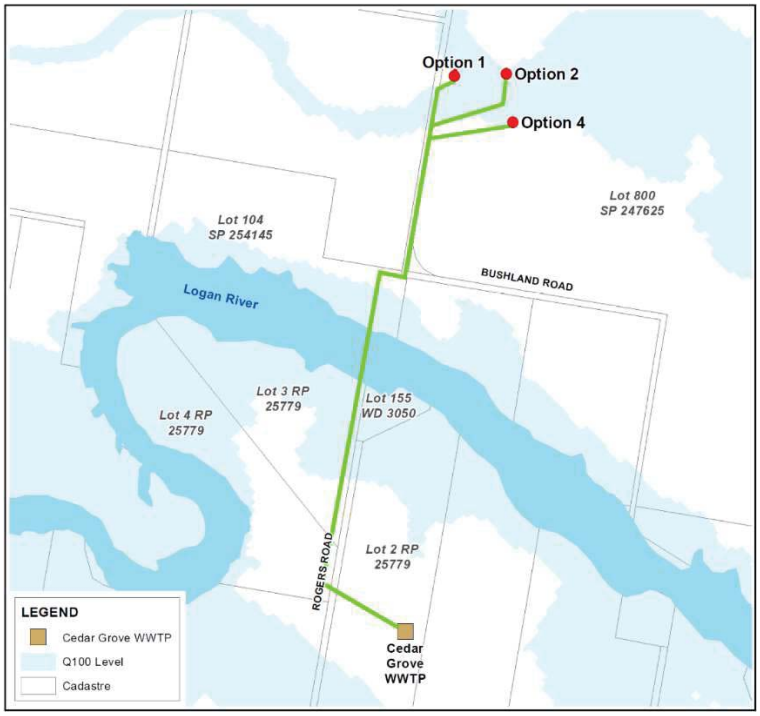


Figure 7-3: Properties affected by RB1 pump station and rising main

Table 7-4: Properties affected by RB1 pumping station and rising main

Tenure	Address	Lot and Plan Number	Owner
Freehold	1342-1464 Teviot Road, Jimboomba	Lot 104 on SP254145	Anthony C Gittins
	72 Riverbend Boulevard, Jimboomba	Lot 800 on SP247625	EJC Riverbend Pty Ltd
	Dennis Road, Cedar Grove	Lot 2 on RP25779	Logan City Council
	Rogers Road, Cedar Grove	Lot 3 on RP25779	Logan City Council
	Rogers Road, Cedar Grove	Lot 4 on RP25779	Logan City Council
Local road	Rogers Road, Cedar Grove		Logan City Council
	Bushland Road, Cedar Grove		Logan City Council
Unallocated State Land	Logan River		DNRM

For Options 1, 2 and 4 the pumping station is located on Lot 800 on SP247625, this land is owned by the developers, EJC Riverbend. The rising main alignment for locations 1, 2 and 3 are all similar; therefore each of these locations have similar town planning and approval requirements. A desktop Approval assessment has been carried out for each location and rising main alignment and this is provided in Appendix F.

All rising alignments will require easements over private freehold land, normally the easement acquisition process occurs over a three to four month period as described below:

- One to two weeks to prepare the report to the Roads and Water Infrastructure (RWI) Committee
- Two to three months to achieve RWI Committee and Council approval and for negotiations with the land owner to resume on voluntary basis.

River crossing

Where the rising main is nominated to cross the Logan River it is classified as riverine (non-tidal). This means that a development application for *Operational Works* relating to tidal works will not be required.

Environmental assessment

The full desktop environmental assessment for the pumping station considering Locations 1, 2 and 4 is provided in Appendix F. As both locations have similar features and alignments the environmental risks are similar. A summary of the key issues identified in the assessment are listed below:

- **Cultural heritage:** The locations fall within Category 4 and Category 5 areas as described under the *Indigenous Cultural Heritage* duty of care guidelines. Consultation with the cultural heritage body will be required prior to any ground disturbance in Category 5 areas but will not be required for Category 4 areas provided the works remain within the road reserve.
- **Ecological:** All locations are located outside of high risk trigger areas defined under the Nature Conservation Act; however, a recent ecological assessment of the Celestino development has recorded *Melaleuca Irbyana* within close proximity to proposed infrastructure. A site assessment will therefore be required to identify any potential impact on the species. Essential Koala habitat areas covers a large part of the alignments for all locations therefore a site investigation will be required under both the *Nature Conservation* Act and *Environmental Protection and Biodiversity Conservation* Act to confirm the absence of threatened species.
- **Non-ecological:** All locations and rising main alignments have similar non-ecological risks, including bushfire, ASS presence and steep slopes. There will be several waterways that need to be crossed by the rising main and these are mapped under the *Vegetation Management* Act, *Fisheries* Act and *Water Act* respectively. Therefore, to minimise associated impacts on these watercourses, it is recommended that trenchless technology is considered.

In summary, the environmental assessment found that Location 4 would have the least impact on vegetation.

Community engagement

The community issues are similar across all of the options. Future community engagement will be required to manage impacts associated with construction and operation of the conveyance infrastructure associated with the RB1 pumping station.

7.3.3 Preferred option

Locating the pumping station at any of locations 1, 2 and 4 will place it adjacent to the early stages of development and future roads. All three locations also have similar environmental and approval requirements and therefore access is the only key differentiator between the locations. Therefore, based on the information available at present the preferred location for the pumping station is Location 4 as it provides the following benefits:

- It is located adjacent to the future access road linking the eastern and western parts of the development
- More optimally situated in relation to the Q100 flood level

- Requires the least amount of vegetation clearing
- Preferred from an environmental perspective.

As the development plans are only in concept stage at the time of writing, further coordination with EDQ and Celestino developers will be required to 'lock' in the final location of the pumping station.

7.4 Infrastructure staging

The infrastructure listed in Section 7.3 is based on the following assumptions:

- RB1 wet-well to be constructed in a single stage (ultimate development)
- The rising main to be constructed in two stages, with Stage 1 sized for 2051 PWWF and Stage 2 sized for ultimate PWWF.

A review of these assumptions is undertaken in the following sections. The optimal arrangement will be selected considering cost (capital and NPC), operability over the asset life, and flexibility to adapt to changes in population projections.

7.4.1 Rising main arrangement

Table 7-5 lists alternative staging configurations considered for the rising main. Scenarios A and B involve constructing the rising main in two stages, while Scenario C involves constructing the rising main in a single stage for ultimate flow. For Scenarios A and B, the DN375 would operate independently, with the first rising main stage providing only contingency or taken out of service.

Table 7-5: Rising main staging scenarios

Scenario	Stage 1		Stage 2	
	Capacity Until	Diameter	Capacity Until	Diameter
A	2036	DN150	Ultimate	DN375
B	2051	DN200	Ultimate	DN375
C	Ultimate	DN375	-	-

Cost analysis

The capital costs and NPC for each scenario are listed in Table 7-6. The costs were calculated using unit rates in $2017.

Table 7-6: Capital costs and NPC ($million) for staging scenarios A-C

	Scenario A	Scenario B	Scenario C
Stage 1 capital cost	$0.72	$0.85	$2.55
Stage 2 capital cost	$2.55	$2.55	-
Total capital cost	**$3.27**	**$3.40**	**$2.55**
NPC	**$1.90**	**$1.47**	**$2.20**

Table 7-6 demonstrates that Scenario C has the lowest overall capital cost but the highest NPC. Conversely, Scenario B has the highest capital cost yet (only marginal compared with A) but the lowest NPC due to the second rising main stage being constructed 15 years later than Scenario A.

Retention times

An important consideration especially in the early years of operation and catchment development is retention time in the rising main. Figure 7-4 is a graph of the rising main retention times for each scenario.

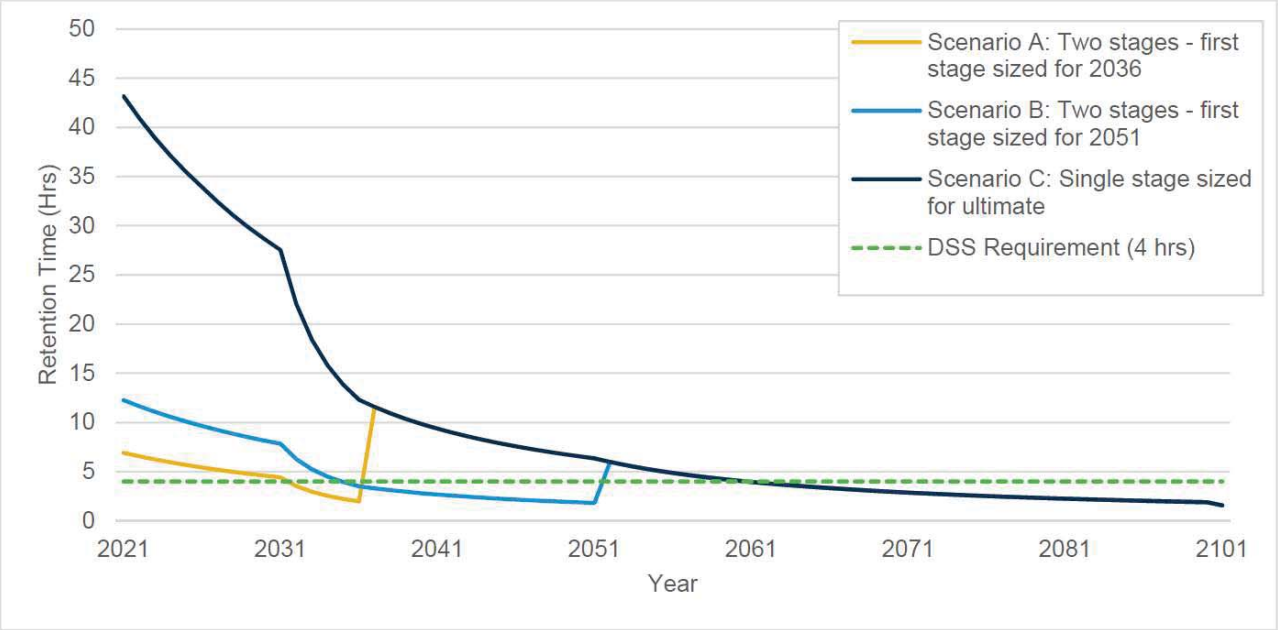


Figure 7-4: Rising main retention times, scenarios A-C

The standards of service specifies that the maximum target retention time for rising mains is four hours. Figure 7-4 demonstrates that sizing the RB1 rising main in a single stage (as per Scenario C) would create an unacceptably high retention time and would not meet the four hour standard until to 2061. Scenario A provides the shortest retention times initially, but results in higher retention times than Scenario B between 2036 and 2051 due to the increase in capacity.

Preferred staging scenario

Considering costs and retention times the optimal staging arrangement is Scenario B (DN200 sized for 2051 PWWF, followed by a DN375 sized for ultimate PWWF), this is justified for the following reasons:

- It minimises upfront capital expenditure and has the lowest NPC
- It minimises retention time over the lifecyle of the main
- Provides Council with greater flexibility to manage changes in the projected growth rates and/or ultimate wastewater loads

7.4.2 Wet well staging

Based on the philosophy adopted for the rising mains, the pumping station is proposed to be staged also, with an initial wet well diameter of 3.0m diameter, followed by an ultimate diameter of 4.5m to be constructed post 2051. It is not proposed to operate both wells in parallel, but rather convert the Stage 1 wet well to a

collector manhole following commissioning of the Stage 2 well. This solution provides Council with maximum flexibility to deal with changes in population forecasts and retention time and the 3.0m diameter well would be operation for at least 50 years.

7.5 Preferred location and infrastructure summary

In summary Locations 1, 2 and 4 are all suitable options for locating the pumping station. After discussions with Council stakeholders and EDQ, all parties agreed to proceed with Location 4 as preferred and to take forward to preliminary design. Should the developer subdivision plans change significantly it would be a simple process to relocate the pumping station without this having a large impact on the design.

A first principles cost estimate of the pumping station and rising main has been undertaken and is summarized in Table 7-7, as well as the infrastructure requirements for this location. The detailed cost schedule is provided in Appendix G for reference.

Table 7-7: Cost estimate for RB1 pumping station and rising main

Asset	Sizing Details	Capital Cost ($2017)	NPC
Stage 1 (2021)			
RB1 rising main	2,130 m of DN200	$2,258,100	$1,950,800
RB1 pump station	3.0m diameter x 10 m deep wet well 25 kW total installed power	$2,083,700	$1,800,100
Total cost for Stage 1		**$4,341,800**	**$3,750,900**
Stage 2 (2051)			
RB1 rising main	2,130 m of DN375	$2,897,300	$835,400
RB1 pump station	4.5m diameter x 10m deep wet well 188 kW total installed power	$2,968,800	$856,000
Total cost for Stage 2		**$5,866,100**	**$1,691,400**
Total cost		**$10,208,000**	**$5,442,300**

8. DETAILED PLANNING: RB2-2 PUMP STATION

This chapter presents the detailed planning for the RB2-2 pump station and rising main. In accordance with the preferred conveyance strategy identified in Section 6, the sizing basis for the RB-2 pump station includes the following:

- The RB2-2 pump station and rising main is sized for local catchment loads only (i.e. excludes contribution from the RB1 catchment)
- The RB2-2 pump station will pump into the RB2-1 rising main.

8.1 Approved population forecasts

The forecast population projections for the RB2-1 catchment are listed in Table 8-1, as the table shows no growth is expected until 2026 at the earliest.

Table 8-1: RB2-2 catchment - forecast population projections

Parameter	2021	2026	2031	2041	2051	Ultimate
EP	0	212	785	1,811	2,163	9,087
ADWF (L/s)	0	0.4	1.5	3.5	4.1	17.4
PWWF (L/s)	0	2	9	21	25	105

8.2 Options development

8.2.1 Location

The proposed location for the RB2-2 pump station is shown on Figure 8-1. This site was selected as it can command the whole catchment and given that there are no immediate development plans for the catchment (as demonstrated by the population forecasts), there is no basis for assessing alternative locations.

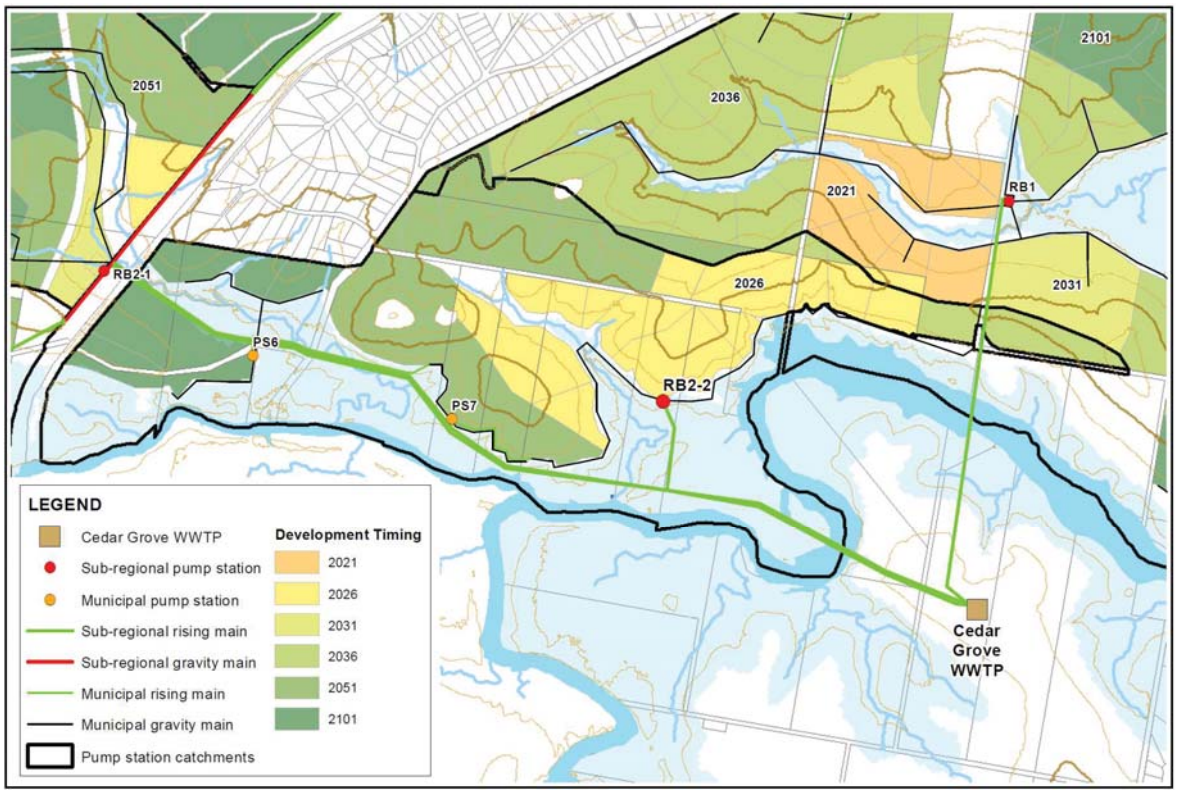


Figure 8-1: Proposed location of the RB2-2 pump station

8.2.2 Infrastructure staging

Staging and non-staging options have been considered for the pumping station and rising main and these options are listed below:

- Option 1: Construct rising main for ultimate flows
- Option 2: Construct rising main in two stages, first stage sized for 2041 flow
- Option 3: Construct rising main in two stages, first stage sized for 2056 flow.

The infrastructure capacity requirements for each option are detailed in Table 8-2, the rising main alignment is the same for all three options and is approximately 400 metres long.

Table 8-2: Infrastructure requirements for RB2-2 options

Asset	Parameter	Option 1	Option 2	Option 3
Pump station	Diameter (m)	3.0	3.0	3.0
	Depth of incoming gravity main (m)	7.2	7.2	7.2
	Total depth (m)	9.7	9.7	9.7
	Total installed power for Stage 1 (kW)	24	16	12
	Total installed power for Stage 2 (kW)	68	68	68
Rising main	Stage 1 diameter	DN225	DN100	DN150
	Stage 2 diameter	-	DN225	DN225

For Option 1 the pumps are oversized to maintain adequate velocity in the rising main. For all three options a DN225 rising main is proposed to service the ultimate load. For Options 2 and 3 it is proposed that the DN225 would service all flows, and the first stage mains would provide contingency only or be taken out of service. The proposed wet well diameter is 3.0 metres.

8.3 Options analysis

To select a preferred arrangement analysis of capital and operational costs, NPC and risks and opportunities has been undertaken and is detailed in the following sections.

8.3.1 Cost estimate

The capital costs and NPC for each option is summarised in Table 8-3. The capital costs are calculated from unit rates and are presented in $2017. The operating and maintenance costs include the energy costs for the pumping station and maintenance costs for the pumping station and rising main.

Table 8-3: Capital ($2017) and NPC comparison

	Option 1	Option 2	Option 3
CAPITAL COST			
RB2-2 rising main	$0.19	$0.25	$0.27
RB2-2 pump station	$1.55	$1.43	$1.39
Total	**$1.74**	**$1.68**	**$1.67**
Ranking	**3**	**2**	**1**
NPC			
RB2-2 rising main	$0.14	$0.13	$0.12
RB2-2 pump station	$0.75	$0.69	$0.67
Capital NPC	**$0.89**	**$0.82**	**$0.79**
Operation and maintenance NPC	**$0.47**	**$0.43**	**$0.42**
Total NPC	**$1.36**	**$1.25**	**$1.21**
Ranking	**3**	**2**	**1**
Difference ($)	**$0.15**	**$0.04**	**-**
Difference (%)	**12%**	**4%**	**-**

The costs show that Option 1 has the highest NPC this is because it proposes to construct the rising main in a single stage, furthermore Option 1 oversizes the pumps to avoid low velocities in a rising main sized for post 2051 flows.

Option 3 has a lower NPC than Option 2 due to the savings realized by delaying the construction of the Stage 2 rising main by 15 years which outweighs the additional cost of installing a DN150 over a DN100 in the first stage.

8.3.2 Retention times

Figure 8-2 graphs the retention times for each rising main option (only between the pumping station and the connection to the RB2-1 rising main).

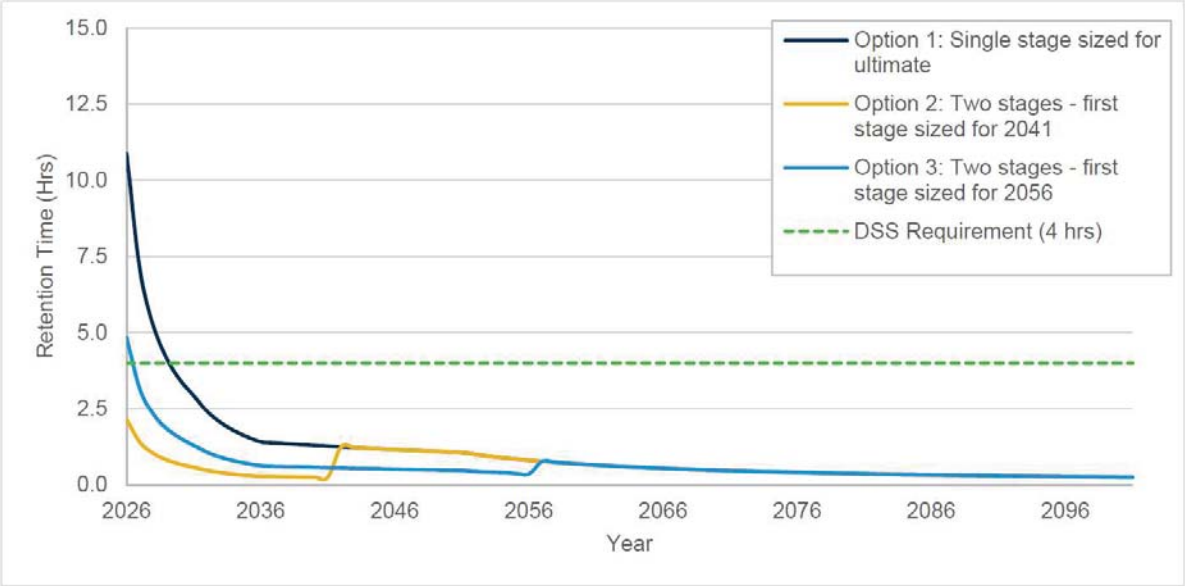


Figure 8-2: Retention times for RB2-2 rising main options

As mentioned in the previous chapter the level of service standards specifies a target maximum retention time of four hours, as Figure 8-2 demonstrates this can be achieved initially only by constructing the rising main over two stages but by approximately 2030 (four years after operation) all options will achieve the target.

8.3.3 Risks and opportunities

This section describes the risks and opportunities associated with the options for the pumping station and rising main.

Property approvals

The properties affected by the pumping station and rising main alignment are listed in Table 8-4 and shown in Figure 8-3. Easements will be required over these properties for the rising main. For the pumping station site, Council has a preference to purchase the required land rather than obtain a separate easement. However, as development within the area will likely require a portion of land to be dedicated to Council or the State for community greenspace the purchasing aspect for the site will need to be assessed once development plans are submitted.

Table 8-4: List of properties affected by RB2-2 pump station ang rising main alignment

Tenure	Address	Lot and Plan Number	Owner
Freehold	Gittins Road, Jimboomba	Lot 156 on SL11068	Anthony C Gittins
	Gittins Road, Jimboomba	Lot 157 on SL11068	Anthony C Gittins

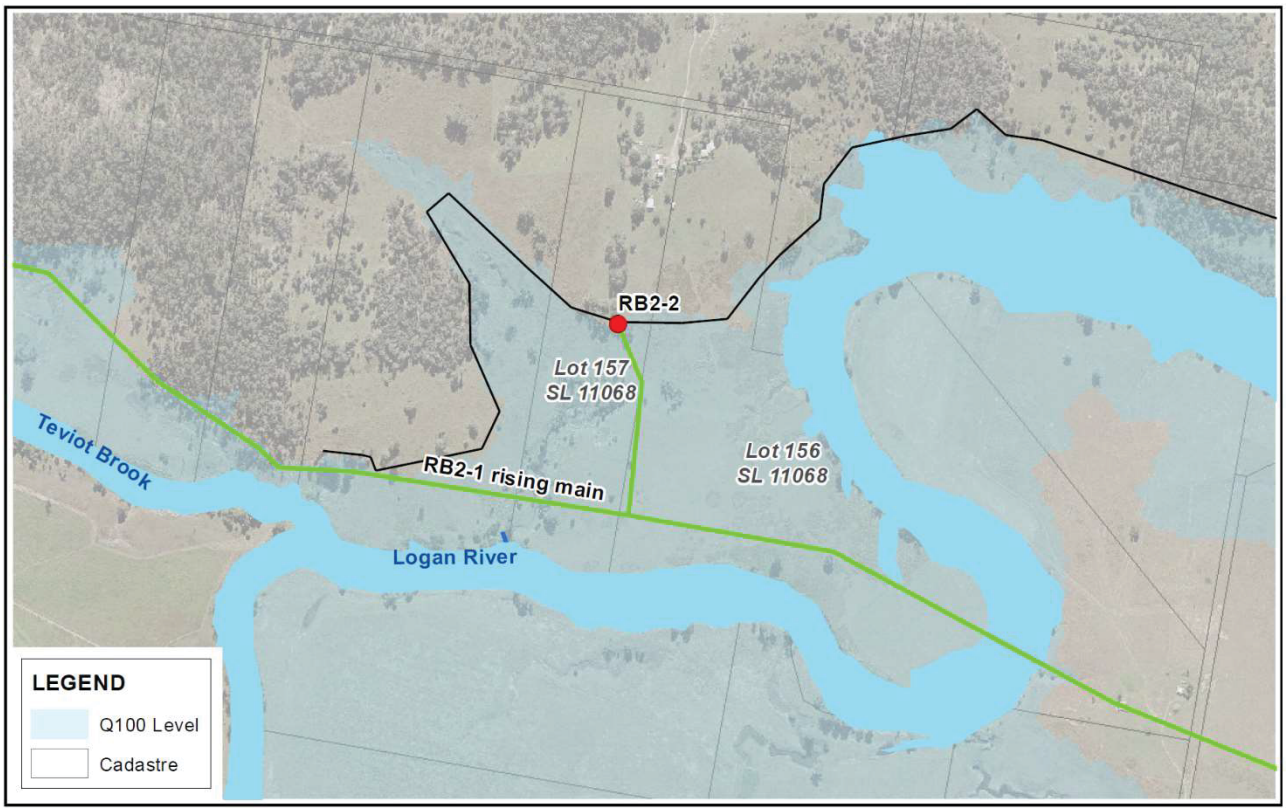


Figure 8-3: Properties affected by the pumping station and rising main alignment

Environmental assessment

A desktop environmental assessment for the pumping station and rising main has been carried out and is provide in Appendix F for reference. A summary of the key issues identified by the assessment are listed below:

- **Cultural heritage:** The site is located within both a Category 4 and Category 5 area under the *Indigenous Cultural Heritage* duty of care guidelines. Consultation with the cultural heritage body will be required prior to any ground disturbance.
- **Ecological:** The proposed rising main alignment is located entirely within Category X mapped vegetation. The alignment does not traverse any watercourse, however is close to the wetland associated with the Logan River. The alignment is located outside of the high risk trigger area under the *Nature Conservation* Act.
- **Non-ecological:** The RB2-2 alignment crosses through a lot listed on the Environmental Management Register for Contaminated Land. The vast majority of the alignment is mapped in a Flood Hazard area. There are a number of common environmental risks that will need to be managed during construction, including: groundwater intrusion, bushfire, flood and fire ants.

Community engagement

The community issues are similar across all of the options. Future community engagement will be required to manage impacts associated with construction and operation of the conveyance infrastructure associated with the RB2-2 pumping station.

8.4 Preferred option summary

Based on cost and non-cost considerations and analysis, the recommended option is Option 3. This option involves constructing a DN150 rising main in Stage 1 (2026), followed by a DN225 rising main in Stage 2 (2056). This option was selected as it provides the following benefits:

- Flexibility to cater for changes in forecast growth[1]
- Marginally reduced upfront capital expenditure
- Avoids having to oversize the pumps to achieve adequate velocities in the rising main.

Table 8-5 summarises the first principles cost estimate of the infrastructure required for the RB2-2 pump station. The detailed cost schedule is provided in Appendix G.

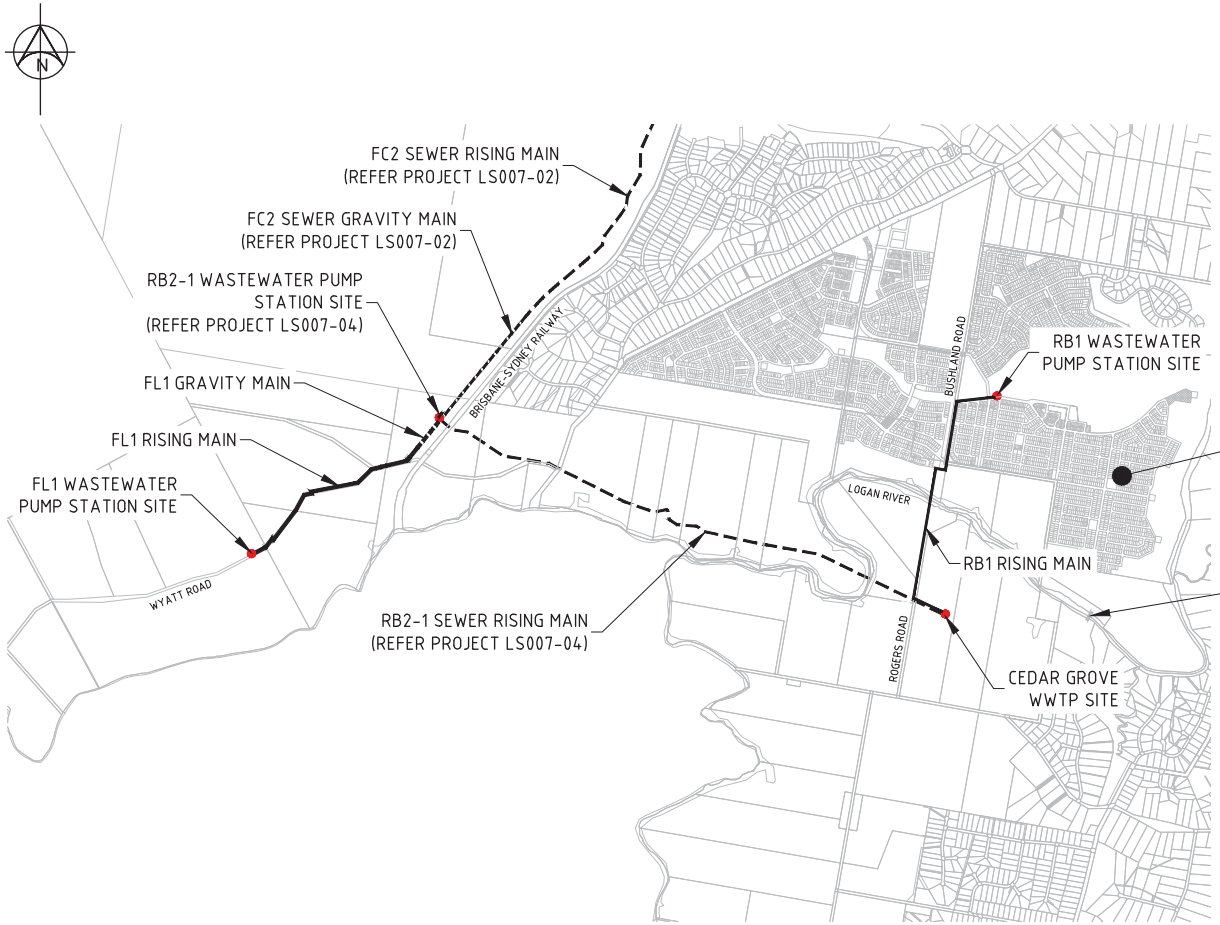
Table 8-5: Unit rate cost estimate for RB2-2 pump station and rising main

Asset Description	Sizing Details	Capital Cost ($2017)	NPC
Stage 1 (2026)			
RB2-2 rising main	385 m of DN150	$367,100	$264,100
RB2-2 pump station	3 m diameter x 9.7 m deep wet well 12 kW total installed power	$1,698,300	$1,221,900
Total cost for Stage 1		**$2,065,400**	**$1,486,00**
Stage 2 (2056)			
RB2-2 rising main	385 m of DN225	$426,100	$102,300
RB2-2 pump station	12 kW total installed power	$143,500	$34,500
Total cost for Stage 2		**$569,600**	**$136,800**
Total cost		**$2,634,929**	**$1,622,800**

[1] There is a risk with constructing a smaller main that it will have insufficient capacity if growth proceeds faster than currently forecast. Conversely, constructing a larger diameter main may result in the infrastructure being oversized if growth projections are not realized

FL1, RB1 AND RB2-2 WW PUMP STATIONS

GENERAL DRAWING SET



INDICATIVE MASTER PLAN, REFER TO DEVELOPERS DRAWING No 30697-GP09 - ILLUSTRATIVE MASTER PLAN (INDICATIVE LOTS AND PROPOSED ROADS SHOWN THROUGHOUT LS014-02 DRAWING SET)

CEDAR GROVE WEIR

LOCALITY PLAN
SCALE 1:20000

LoganWIA - DRAWINGS

DRAWING No	TITLE
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LS014-00-S-DWG-ND-0030	LEGEND
LS014-00-S-DWG-ND-0100	KEY PLAN,

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A	FOR 30% DOAR WORKSHOP	PS	LM	DS	14/07/2017

DESIGNED BY: L. MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F.JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No.: LS014-00
FILE REFERENCE:
SHEET SIZE: A1
DATUM : AHD
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LOGAN WATER INFRASTRUCTURE ALLIANCE

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PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS
DRAWING TITLE: GENERAL DRAWING SET COVER SHEET, LOCALITY PLAN AND DRAWING LIST
DRAWING No.: LS014-00-S-DWG-ND-0000
DESIGN COMPANY.: LoganWIA
REV.: B
TECH SERVICES DRG No.

CAD FILE: T:\LS014 FLINDERS\01 - General Set\02I Acad\LS014-00-S-DWG-ND-0000.dwg
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LEGEND

PROPOSED

- PROPOSED SEWER RISING MAIN
- STRUCTURE NUMBER / LINE NUMBER (X / Y)
- PROPOSED GRAVITY SEWER AND MAINTENANCE HOLE
- FLOW ARROW
- PROPOSED STORMWATER PIPE (D—D—D—D—D)
- PROPOSED WATERMAIN (W—W—W—W—W)
- PROPOSED EASEMENT
- CONSTRUCTION RIGHT OF WAY
- LAYDOWN AREA
- EX-GRAVITY SEWER TO BE ABANDONED (X—X—X—X—X)
- EX-RISING MAIN SEWER TO BE ABANDONED (XX—XX—XX—XX)
- PROPOSED EARTH FILL

EXISTING

- WATER MAIN (TO AS5488 QUALITY LEVEL A) (W(A))
- WATER MAIN (TO AS5488 QUALITY LEVEL B) (W(B))
- WATER MAIN (TO AS5488 QUALITY LEVEL C) (W(C))
- WATER MAIN (TO AS5488 QUALITY LEVEL D) (W(D))
- FIRE HYDRANT
- SLUICE VALVE
- WATER METER
- STORMWATER PIPE (TO AS5488 QUALITY LEVEL A) (D(A))
- STORMWATER PIPE (TO AS5488 QUALITY LEVEL B) (D(B))
- STORMWATER PIPE (TO AS5488 QUALITY LEVEL C) (D(C))
- STORMWATER PIPE (TO AS5488 QUALITY LEVEL D) (D(D))
- STORMWATER MAINTENANCE HOLE
- STORMWATER GRATE
- ELECTRICAL OH (V V V V V)
- PP — POWER POLE
- IB EL — INSPECTION BOX / PIT - ELECTRICITY
- STREET LIGHT
- ELECTRICAL UG (TO AS5488 QUALITY LEVEL A) (E(A))
- ELECTRICAL UG (TO AS5488 QUALITY LEVEL B) (E(B))
- ELECTRICAL UG (TO AS5488 QUALITY LEVEL C) (E(C))
- ELECTRICAL UG (TO AS5488 QUALITY LEVEL D) (E(D))
- TELECOMMUNICATION UG (TO AS5488 QUALITY LEVEL A) (T(A))
- TELECOMMUNICATION UG (TO AS5488 QUALITY LEVEL B) (T(B))
- TELECOMMUNICATION UG (TO AS5488 QUALITY LEVEL C) (T(C))
- TELECOMMUNICATION UG (TO AS5488 QUALITY LEVEL D) (T(D))
- IB TE — TELECOMMUNICATION PIT
- TE — TELECOMMUNICATION

- OPTIC FIBRE (TO AS5488 QUALITY LEVEL A) (OF(A))
- OPTIC FIBRE (TO AS5488 QUALITY LEVEL B) (OF(B))
- OPTIC FIBRE (TO AS5488 QUALITY LEVEL C) (OF(C))
- OPTIC FIBRE (TO AS5488 QUALITY LEVEL D) (OF(D))
- GAS (TO AS5488 QUALITY LEVEL A) (G(A))
- GAS (TO AS5488 QUALITY LEVEL B) (G(B))
- GAS (TO AS5488 QUALITY LEVEL C) (G(C))
- GAS (TO AS5488 QUALITY LEVEL D) (G(D))
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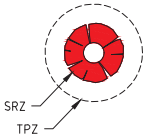
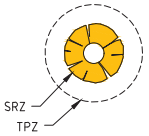
- SEWER RISING MAIN (TO AS5488 QUALITY LEVEL B) (RM(B))
- SEWER RISING MAIN (TO AS5488 QUALITY LEVEL C) (RM(C))
- SEWER RISING MAIN (TO AS5488 QUALITY LEVEL D) (RM(D))
- SE — VALVE - SEWER
- SEWER GRAVITY MAIN (TO AS5488 QUALITY LEVEL A) (S(A))
- SEWER GRAVITY MAIN (TO AS5488 QUALITY LEVEL B) (S(B))
- SEWER GRAVITY MAIN (TO AS5488 QUALITY LEVEL C) (S(C))
- SEWER GRAVITY MAIN (TO AS5488 QUALITY LEVEL D) (S(D))
- SEWER MAINTENANCE HOLE
- EXISTING EASEMENT
- BELOW 1 IN 100 YEAR ARI FLOOD LINE — 1 IN 100 YEAR ARI FLOOD LINE (LCC PLANNING SCHEME GIS)
- TRAFFIC / ROAD SIGN
- PSM — PERMANENT SURVEY MARK
- STN — SURVEY STATION
- RP PEG
- PH1 — POTHOLE
- BH1 — BOREHOLE

- INDICATES A RESIDUAL RISK FOR INFORMATION.
- INDICATES A RESIDUAL RISK AS A WARNING.
- INDICATES A RESIDUAL RISK REQUIRING A PROHIBITIVE ACTION
- INDICATES A RESIDUAL RISK REQUIRING A COMPULSORY ACTION



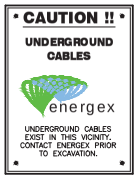
TREE TO BE RETAINED. SUBCONTRACTOR SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:
1. ENSURE ARBORIST IS ON SITE AT ALL TIMES WHEN CONSTRUCTION IS OCCURRING WITHIN THE TPZ.
2. TRENCHING WITHIN THE TPZ AND OUTSIDE THE SRZ IS ACCEPTABLE ONLY UNDER THE SUPERVISION OF THE ARBORIST. TRENCHING WITHIN THE SRZ SHALL NOT BE ASSUMED BUT MAY BE APPROVED BY THE ARBORIST AND LoganWIA PROJECT ENGINEER ON SITE DURING CONSTRUCTION. ARBORIST TO VERIFY ROOT SYSTEM AND EXTENT OF TRENCHING, REMAINDER SHALL BE TRENCHLESS.
3. TRENCHLESS CONSTRUCTION METHODOLOGY MAY INCLUDE PIPE PULLING, HDD OR AIR SHOVEL. SUBCONTRACTOR TO PROVIDE DETAILS TO LoganWIA PROJECT MANAGER FOR APPROVAL.
4. ANY TRIMMING REQUIRED IS TO BE UNDERTAKEN BY THE ARBORIST.

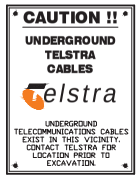


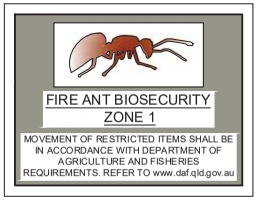
ATTEMPT TO RETAIN TREE. SUBCONTRACTOR SHALL ATTEMPT TO RETAIN TREE BUT CAN REMOVE TREE WITH PRIOR APPROVAL FROM LoganWIA PROJECT ENGINEER OR ARBORIST. REQUIREMENTS FOR RETAINING TREE SHALL BE THE SAME AS "TREE TO BE RETAINED".

EXISTING TREE TO BE REMOVED PRIOR TO SUBCONTRACTOR MOBILISING TO SITE.

DIAL 1100 BEFORE YOU DIG







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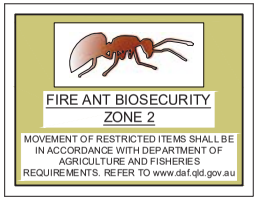

WARNING

NOTE
1. FOR GENERAL NOTES REFER TO DWG ND-0020 AND ND-0021.

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DESIGNED BY: L. MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F.JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No.: LS014-00
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LOGAN WATER INFRASTRUCTURE ALLIANCE

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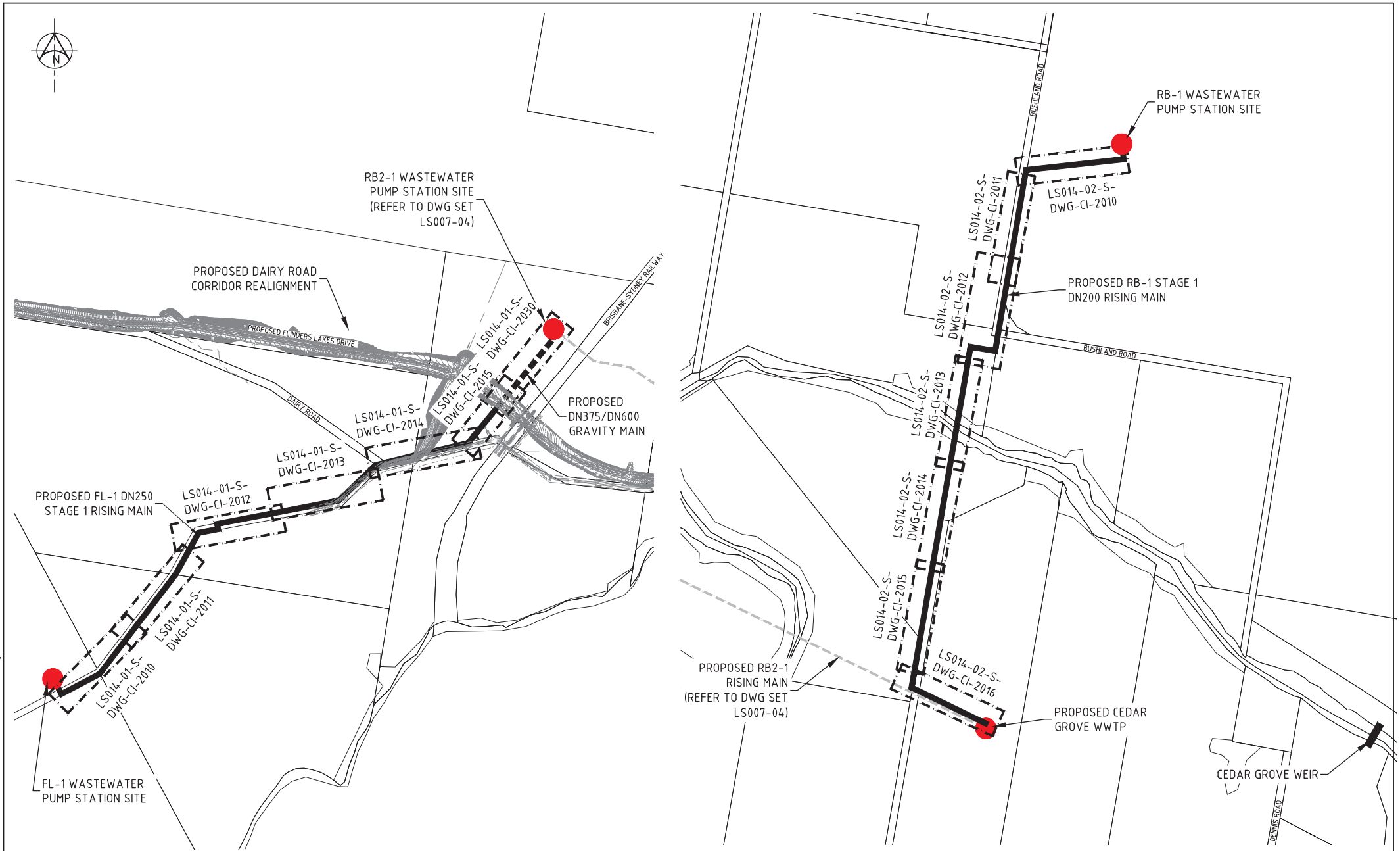

PLAN
SCALE 1:5000

PLAN
SCALE 1:5000

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DRAFTING CHECK BY: F.JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

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PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS

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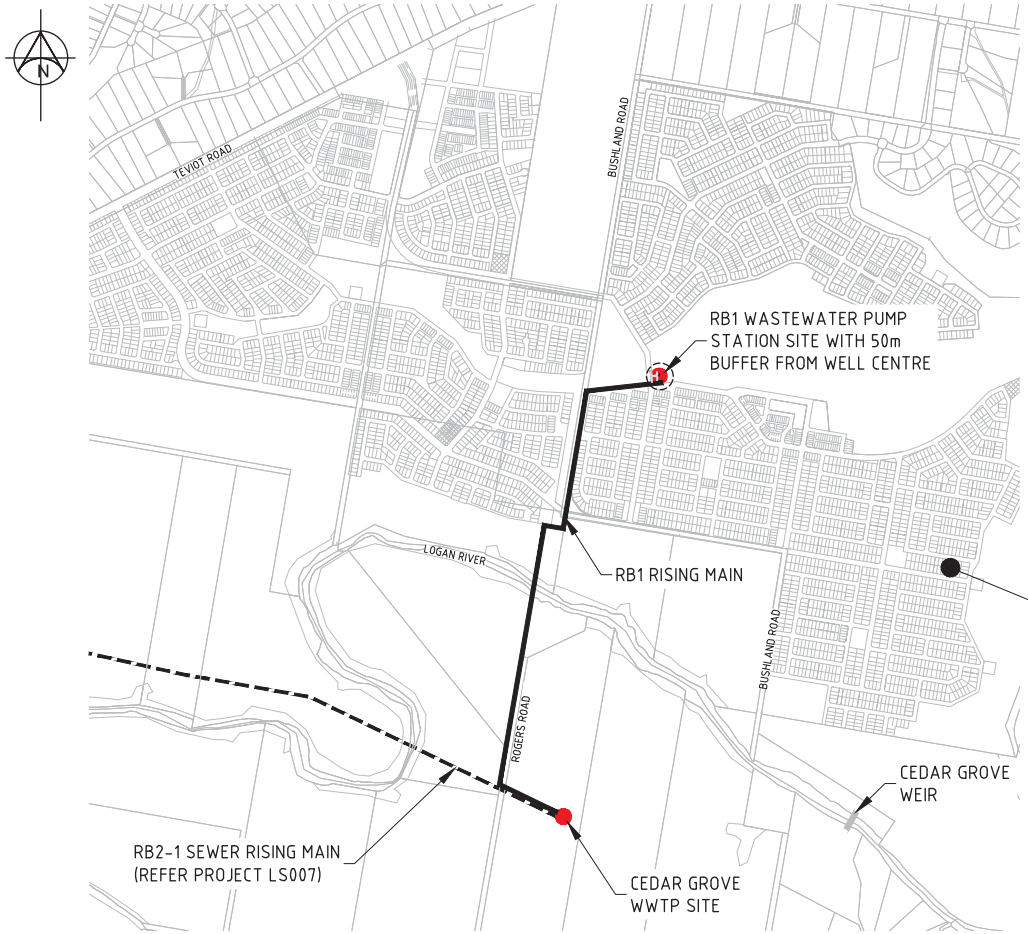
FL1, RB1 AND RB2-2 WW PUMP STATIONS

RB1 WASTEWATER PUMP STATION AND SEWER RISING MAIN



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LoganWIA - DRAWINGS

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LS014-02-S-DWG-CI-2010	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 1 OF 7
LS014-02-S-DWG-CI-2011	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 2 OF 7
LS014-02-S-DWG-CI-2012	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 3 OF 7
LS014-02-S-DWG-CI-2013	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 4 OF 7
LS014-02-S-DWG-CI-2014	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 5 OF 7
LS014-02-S-DWG-CI-2015	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 6 OF 7
LS014-02-S-DWG-CI-2016	RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN, PLAN AND LONGITUDINAL SECTION, SHEET 7 OF 7
LS014-02-S-DWG-CI-2100	RB1 PUMP STATION, SITE LAYOUT
LS014-02-S-DWG-CI-2110	RB1 PUMP STATION, SITE GENERAL ARRANGEMENT
LS014-02-S-DWG-CI-2112	RB1 PUMP STATION, SECTIONS AND DETAILS
LS014-02-S-DWG-CI-2120	RB1 PUMP STATION, WET WELL GENERAL ARRANGEMENT

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DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

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LOGAN WATER INFRASTRUCTURE ALLIANCE

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DRAWING TITLE: RB1 WW PUMP STATION AND SEWER RISING MAIN COVER SHEET, LOCALITY PLAN AND DRAWING LIST
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DESIGN COMPANY: LoganWIA
REV.: B
TECH SERVICES DRG No.:

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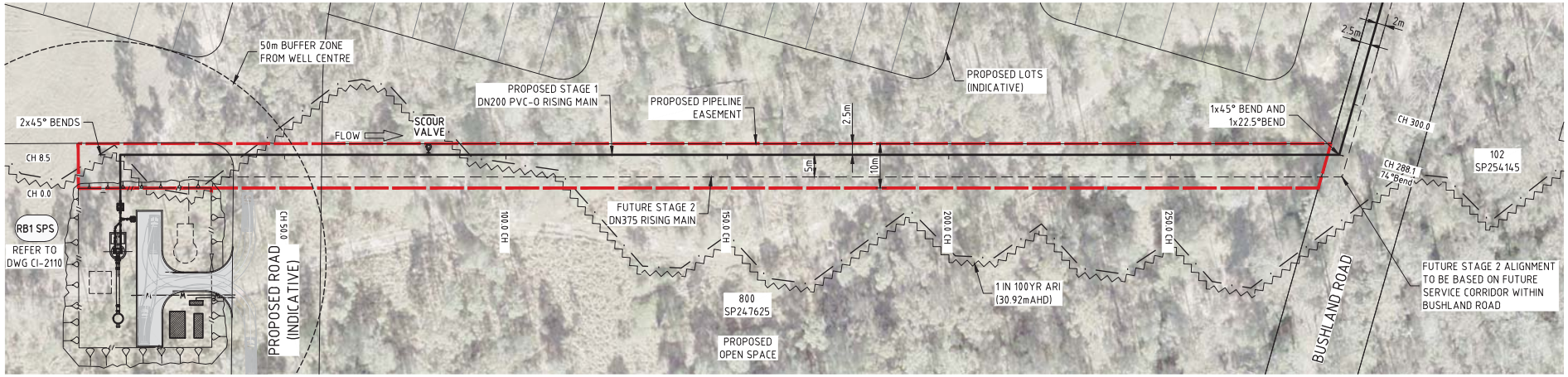
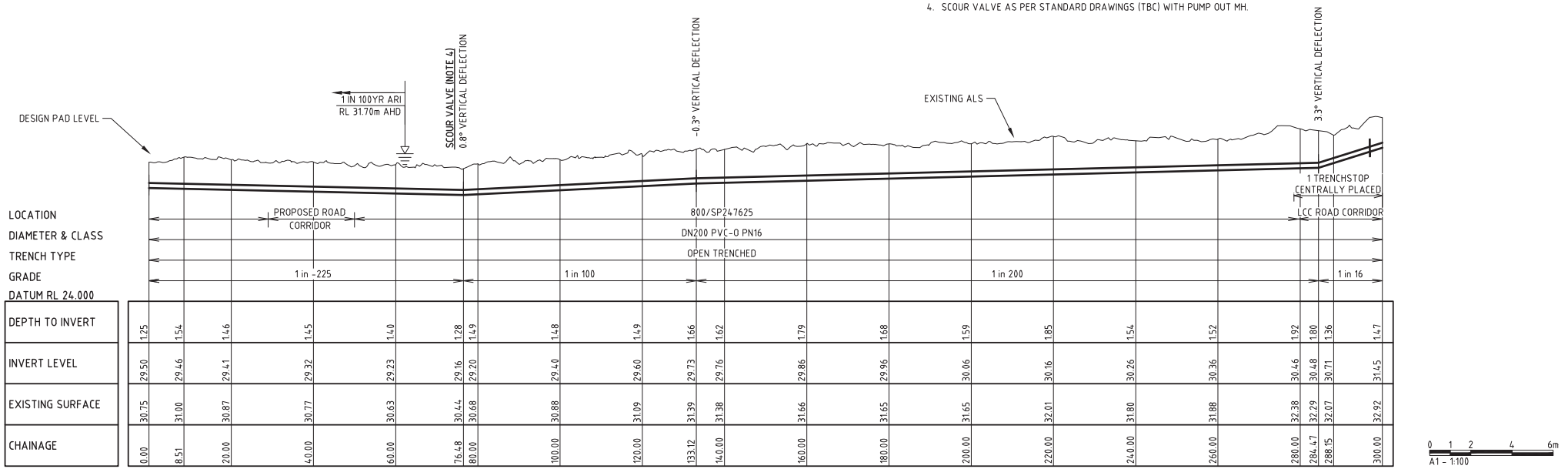


PLAN
SCALE 1:500

NOTE
1. Q100 FLOOD EXTENTS ARE EXTRACTED FROM THE LOGAN CITY COUNCIL PLANNING SCHEME MAPS. WHERE THE Q100 LEVEL CONTRADICTS ALS SURFACE LEVEL DATA, THE Q100 LEVEL HAS BEEN ADJUSTED TO MATCH THE ALS SURFACE DATA FOR THAT LEVEL AS INDICATED ON THE RELEVANT DRAWING.
2. RISING MAIN ALIGNMENT TO BE CONFIRMED AGAINST DEVELOPERS PLANS AND STAGING.
3. DN375 STAGE 2 RISING MAIN EXPECTED TO REPLACE STAGE 1 RISING MAIN BEYOND 2051. STAGE 1 RISING MAIN EXPECTED TO BE USED AS CONTINGENCY BEYOND 2051.
4. SCOUR VALVE AS PER STANDARD DRAWINGS (TBC) WITH PUMP OUT MH.



LONGITUDINAL SECTION
SCALE HOR 1:500, VERT 1:100

0 1 2 4 6m
A1 - 1:100
0 5 10 20 30m
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DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

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PROJECT No. LS014-02
FILE REFERENCE:
SHEET SIZE A1
DATUM : AHD GDA 94 ZONE 56.

LOGAN WATER INFRASTRUCTURE ALLIANCE
Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS
DRAWING TITLE: RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN
PLAN AND LONGITUDINAL SECTION
SHEET 1 OF 7
DRAWING No. LS014-02-S-DWG-CI-2010
DESIGN COMPANY: LoganWIA
REV. B
TECH SERVICES DRG No.

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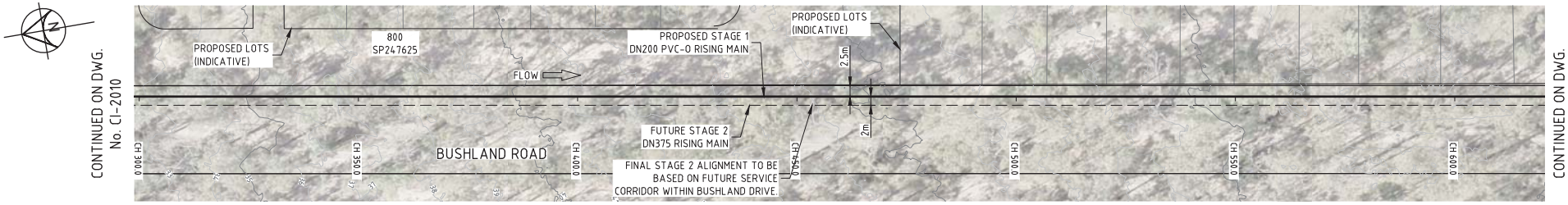
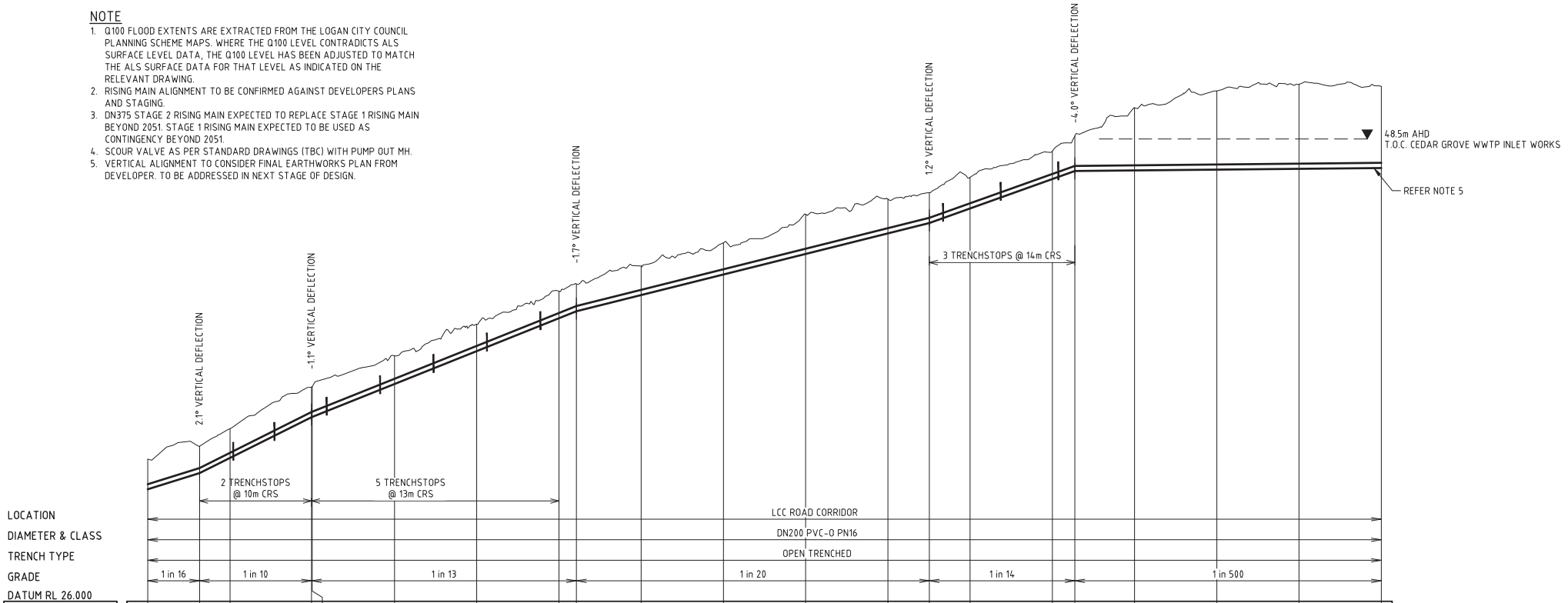


PLAN
SCALE 1:500

NOTE
1. Q100 FLOOD EXTENTS ARE EXTRACTED FROM THE LOGAN CITY COUNCIL PLANNING SCHEME MAPS. WHERE THE Q100 LEVEL CONTRADICTS ALS SURFACE LEVEL DATA, THE Q100 LEVEL HAS BEEN ADJUSTED TO MATCH THE ALS SURFACE DATA FOR THAT LEVEL AS INDICATED ON THE RELEVANT DRAWING.
2. RISING MAIN ALIGNMENT TO BE CONFIRMED AGAINST DEVELOPERS PLANS AND STAGING.
3. DN375 STAGE 2 RISING MAIN EXPECTED TO REPLACE STAGE 1 RISING MAIN BEYOND 2051. STAGE 1 RISING MAIN EXPECTED TO BE USED AS CONTINGENCY BEYOND 2051.
4. SCOUR VALVE AS PER STANDARD DRAWINGS (TBC) WITH PUMP OUT MH.
5. VERTICAL ALIGNMENT TO CONSIDER FINAL EARTHWORKS PLAN FROM DEVELOPER. TO BE ADDRESSED IN NEXT STAGE OF DESIGN.



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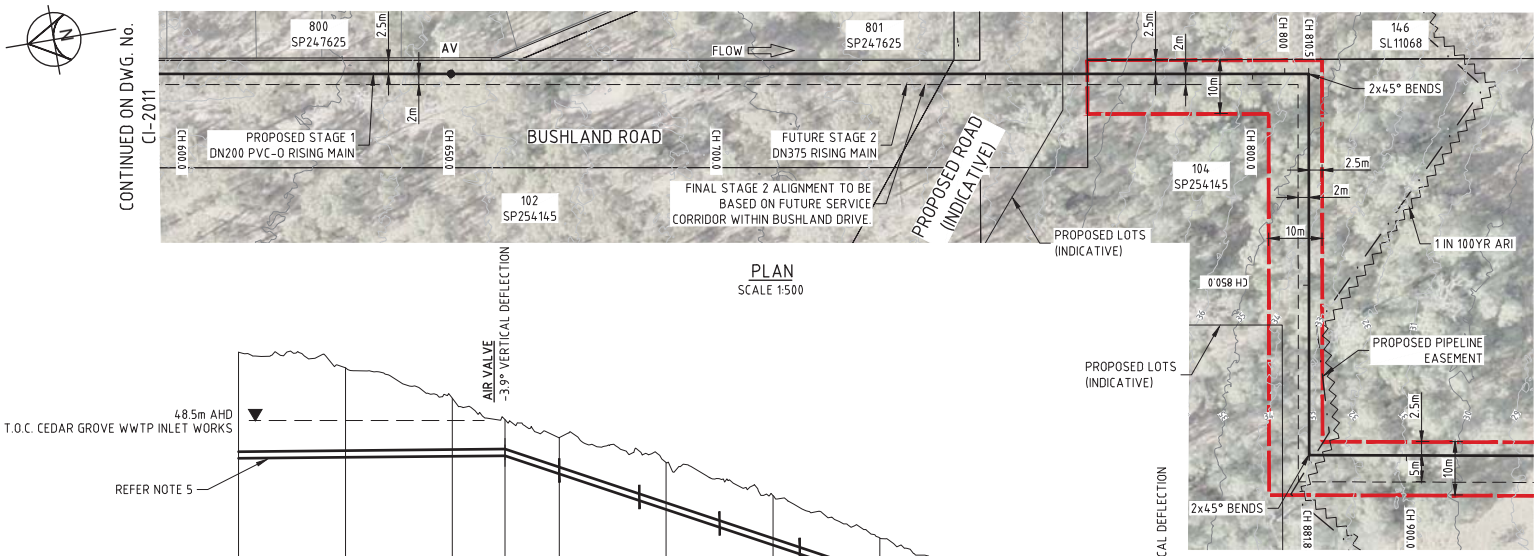

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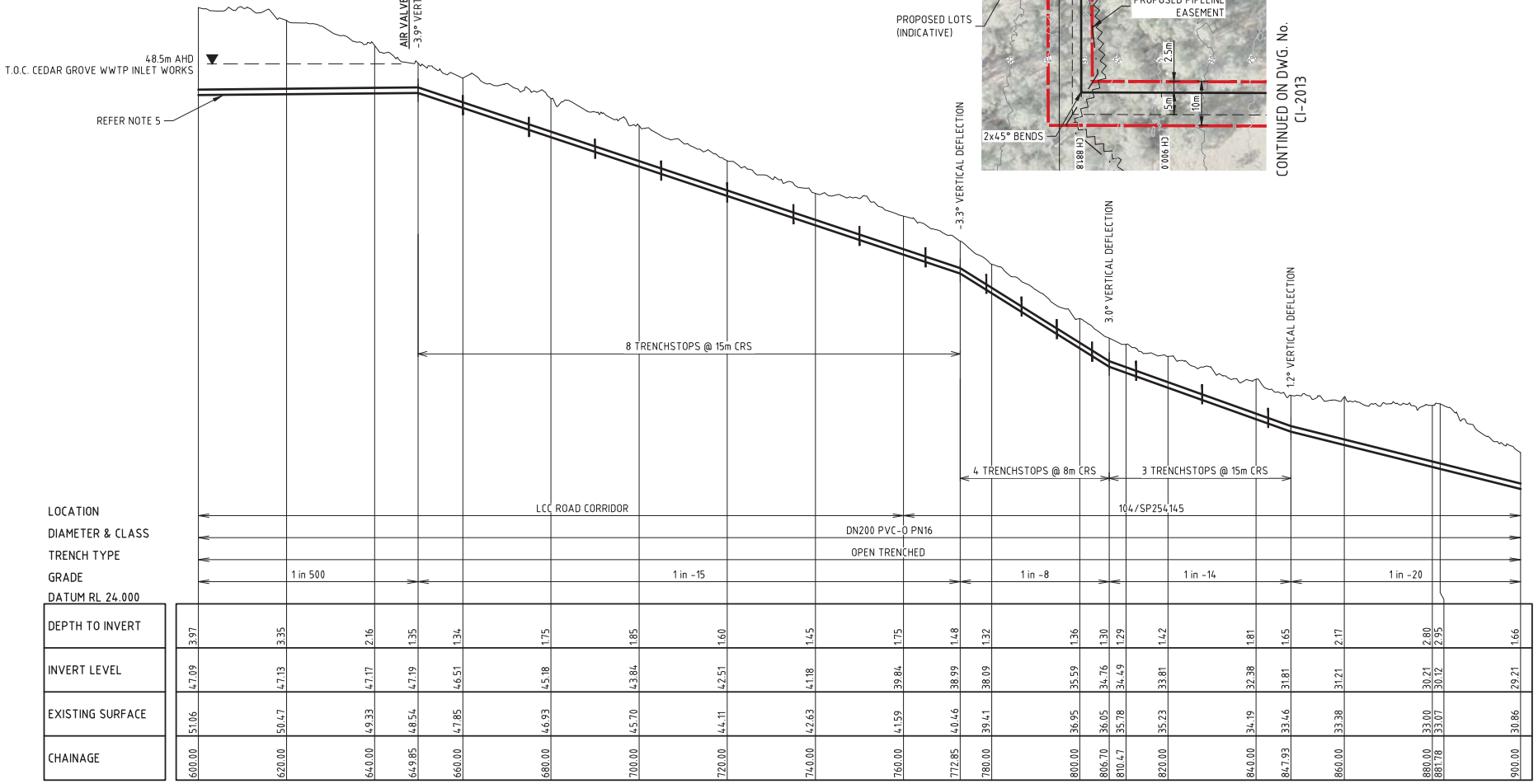

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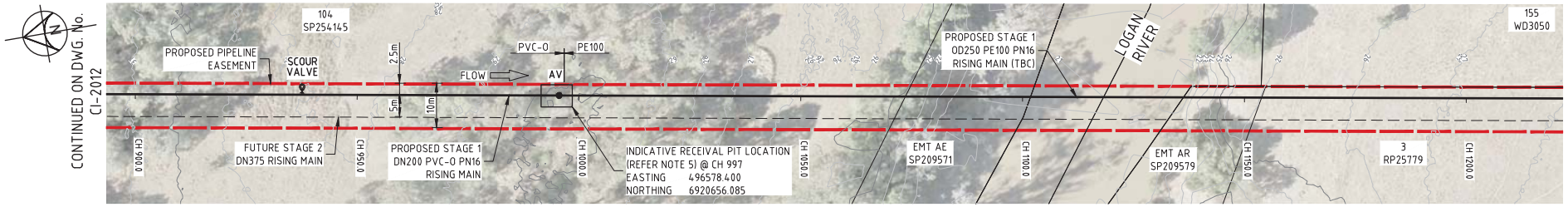

PLAN
SCALE 1:500

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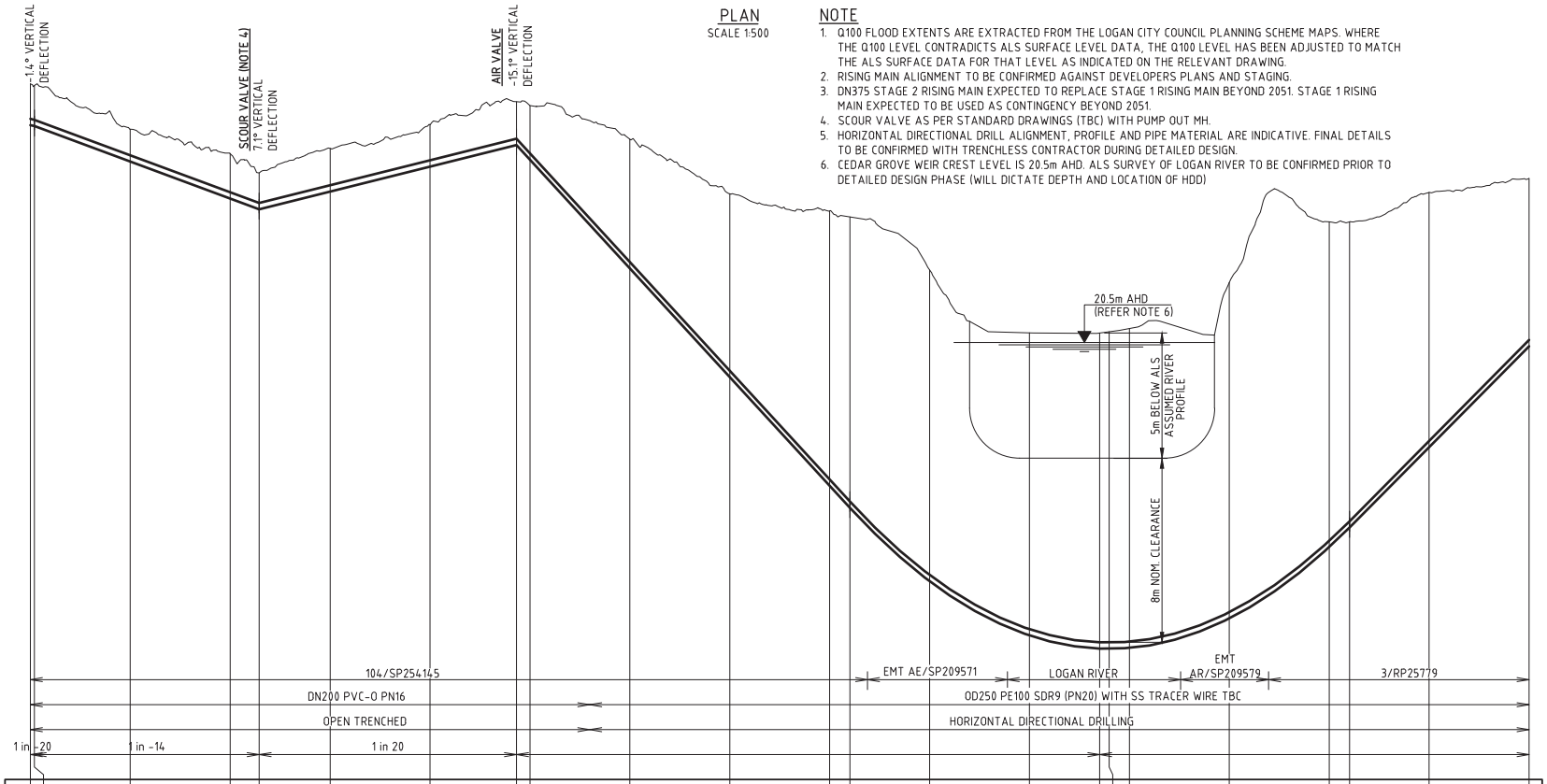
4. SCOUR VALVE AS PER STANDARD DRAWINGS (TBC) WITH PUMP OUT MH.
5. HORIZONTAL DIRECTIONAL DRILL ALIGNMENT, PROFILE AND PIPE MATERIAL ARE INDICATIVE. FINAL DETAILS TO BE CONFIRMED WITH TRENCHLESS CONTRACTOR DURING DETAILED DESIGN.
6. CEDAR GROVE WEIR CREST LEVEL IS 20.5m AHD. ALS SURVEY OF LOGAN RIVER TO BE CONFIRMED PRIOR TO DETAILED DESIGN PHASE (WILL DICTATE DEPTH AND LOCATION OF HDD)



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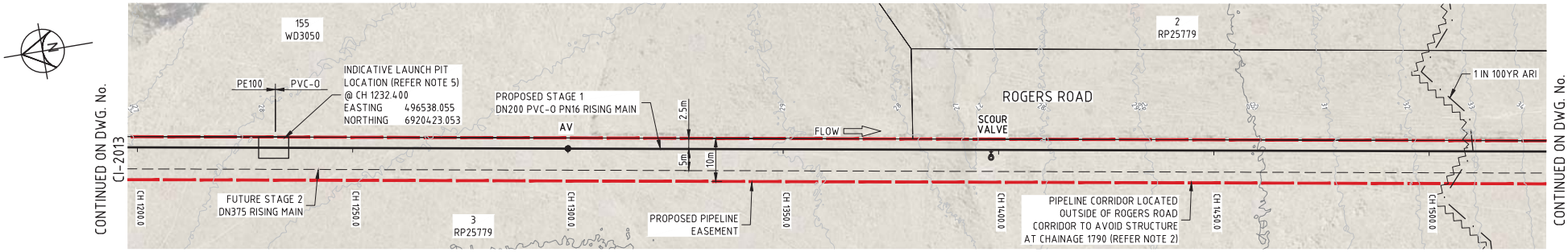

PLAN
SCALE 1:500

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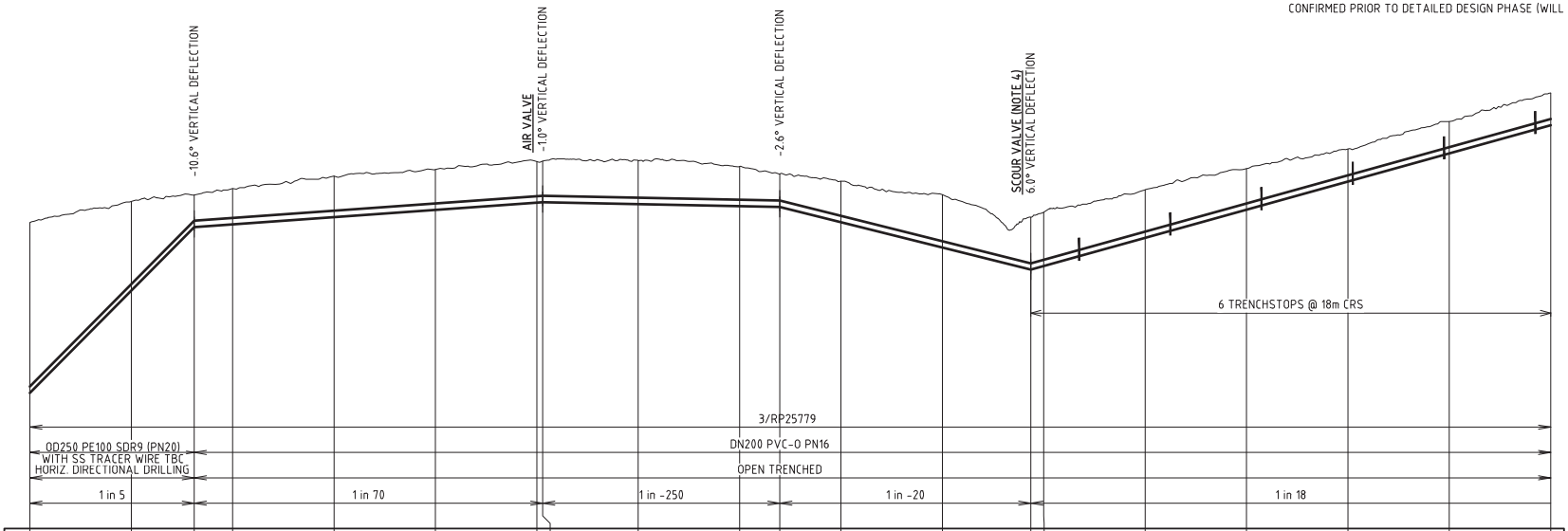

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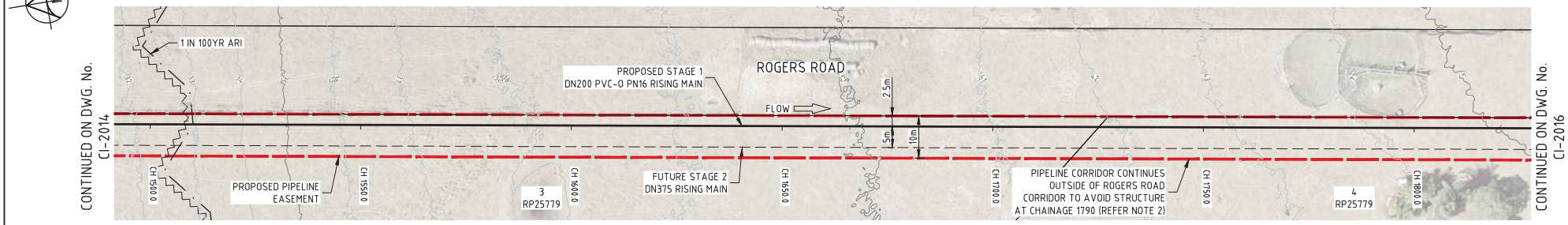

PLAN
SCALE 1:500

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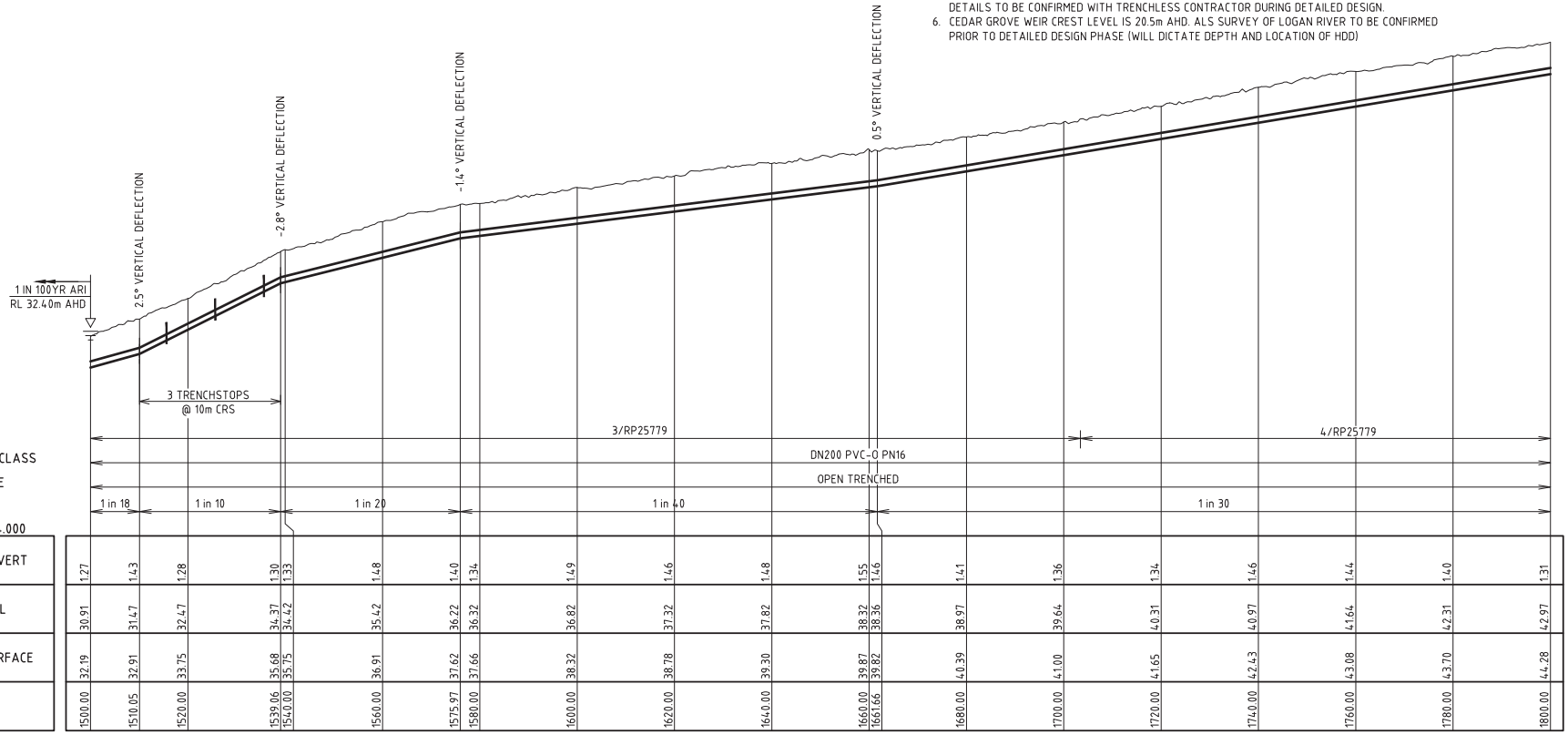

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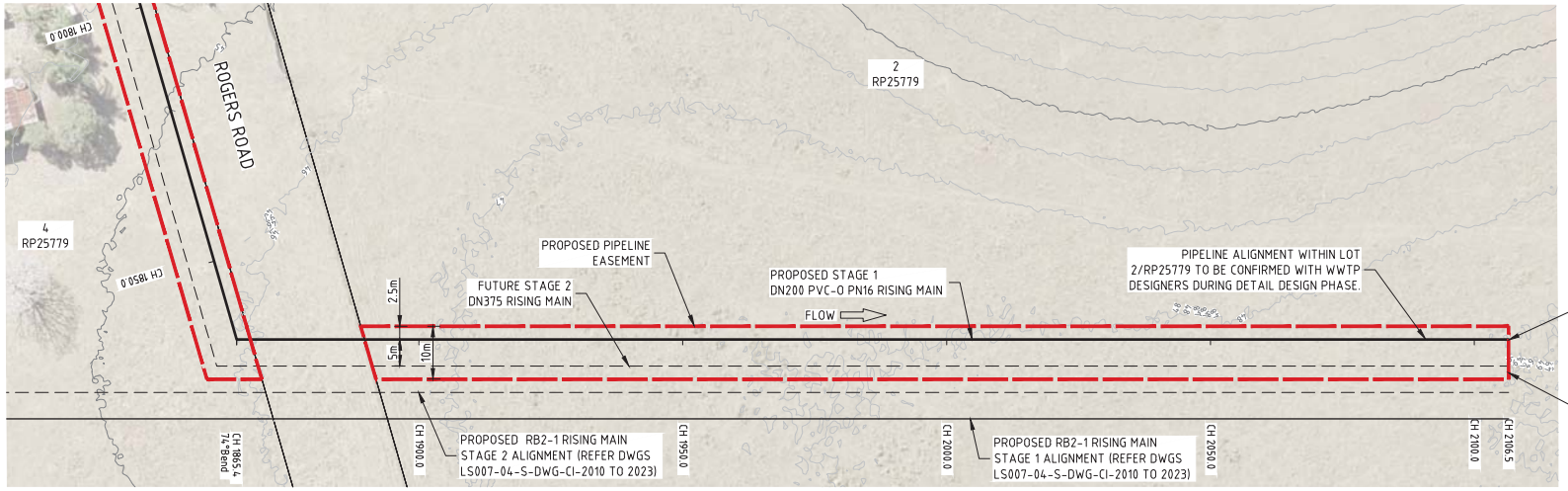
CONTINUED ON DWG. No. CI-2015



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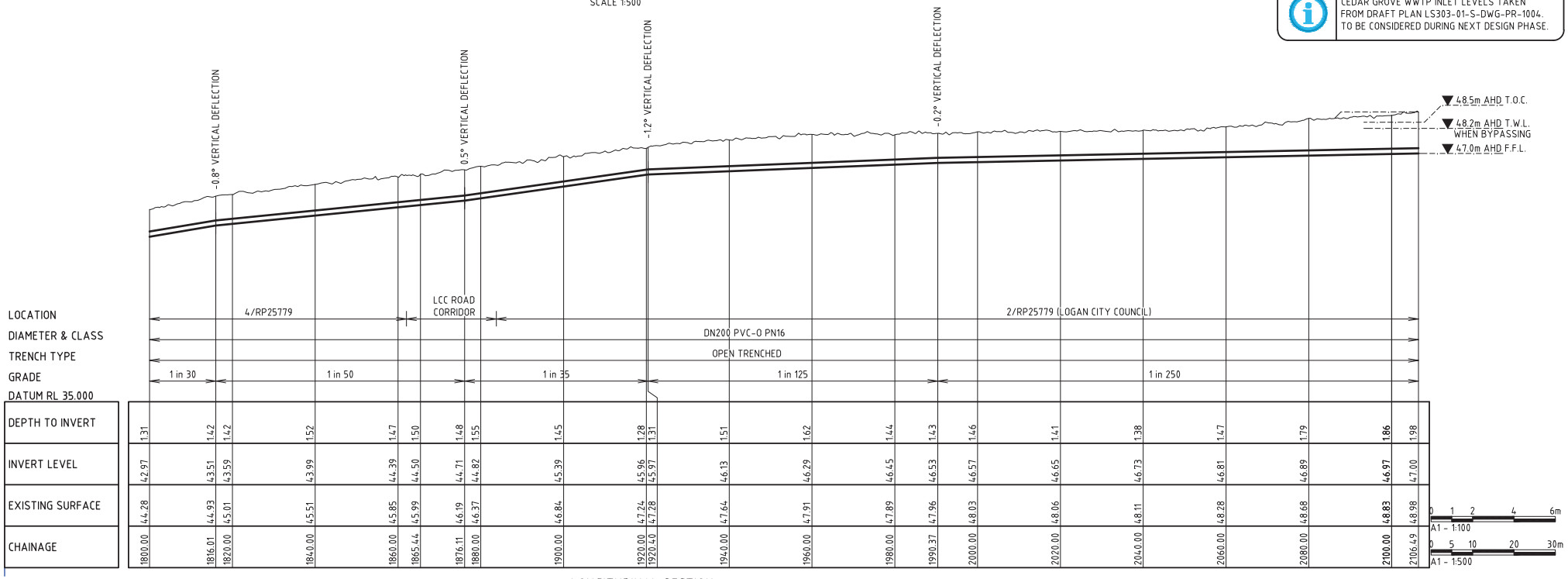
4. DEPENDING ON WWTP INLET DESIGN, IT MAY BE ADVANTAGEOUS TO RUN BOTH STAGE 1 PIPELINES FROM RB-1 AND RB2-1 NEXT TO EACH OTHER, WITH ALL FUTURE PIPELINES ON THE OUTSIDE OF STAGE 1 PIPELINES (TO BE CONFIRMED AT DETAILED DESIGN PHASE).

PLAN

SCALE 1:500

CEDAR GROVE WWTP INLET LEVELS TAKEN FROM DRAFT PLAN LS303-01-S-DWG-PR-1004. TO BE CONSIDERED DURING NEXT DESIGN PHASE.



LONGITUDINAL SECTION

SCALE HOR 1:500, VERT 1:100

PRELIMINARY

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REV. No.	DESCRIPTION	DRAWN	DESIGNED	APPR.	DATE
B	FOR LS014 PRELIMINARY DESIGN REPORT	PS	LM	DS	04/10/2017
A	FOR 30% DOAR WORKSHOP	PS	LM	DS	14/07/2017

DESIGNED BY: L.MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No. LS014-02
FILE REFERENCE:
SHEET SIZE A1
DATUM : AHD GDA 94 ZONE 56.

LOGAN WATER INFRASTRUCTURE ALLIANCE

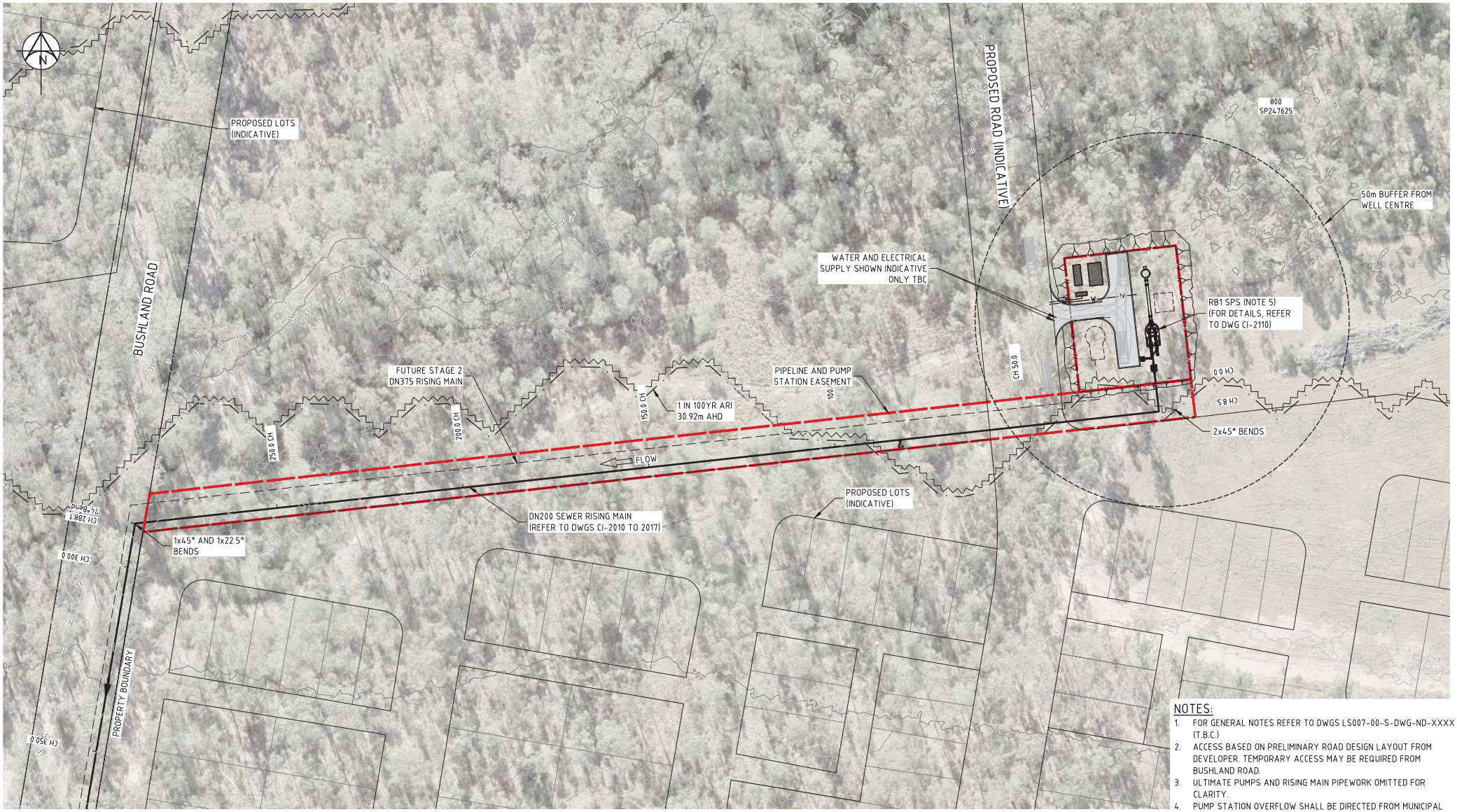
Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS

DRAWING TITLE: RB1 TO CEDAR GROVE WWTP SEWER RISING MAIN
PLAN AND LONGITUDINAL SECTION
SHEET 7 OF 7

DRAWING No. LS014-02-S-DWG-CI-2016
DESIGN COMPANY: LoganWIA
REV. B
TECH SERVICES DRG No.

CAD FILE: I:\LS014 FLINDERS\02 - RIVERBANK EAST\020 Acad\LS014-02-S-DWG-CI-2016.dwg
DATE PLOTTED: 06 October, 2017 - 12:45pm



OVERALL LAYOUT
SCALE 1:500

PUMP STATION AND RISING MAIN ALIGNMENT TO BE CONFIRMED AGAINST DEVELOPERS PLANS AND STAGING.

NOTES:

1. FOR GENERAL NOTES REFER TO DWGS LS007-00-S-DWG-ND-XXXX (T.B.C.)
2. ACCESS BASED ON PRELIMINARY ROAD DESIGN LAYOUT FROM DEVELOPER. TEMPORARY ACCESS MAY BE REQUIRED FROM BUSHLAND ROAD.
3. ULTIMATE PUMPS AND RISING MAIN PIPEWORK OMITTED FOR CLARITY.
4. PUMP STATION OVERFLOW SHALL BE DIRECTED FROM MUNICIPAL INFRASTRUCTURE TO NEARBY STREAM TO THE NORTH OF THE PUMP STATION (TO BE CONFIRMED DURING DETAILED DESIGN PHASE).
5. GIVEN UNCERTAINTY OF DEVELOPEMENT PLANS AVAILABLE, FINAL LOCATION TO BE CONFIRMED PRIOR TO DETAILED DESIGN.

0 5 10 20 30m
A1 - 1:500

PRELIMINARY

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A	FOR 30% DOAR WORKSHOP	PS	LM	DS	14/07/2017

DESIGNED BY: L.MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No.: LS014-02
FILE REFERENCE:
SHEET SIZE: A1
DATUM : AHD GDA 94 ZONE 56.

LOGAN WATER INFRASTRUCTURE ALLIANCE

Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS

DRAWING TITLE: RB1 PUMP STATION SITE LAYOUT

DRAWING No.: LS014-02-S-DWG-CI-2100

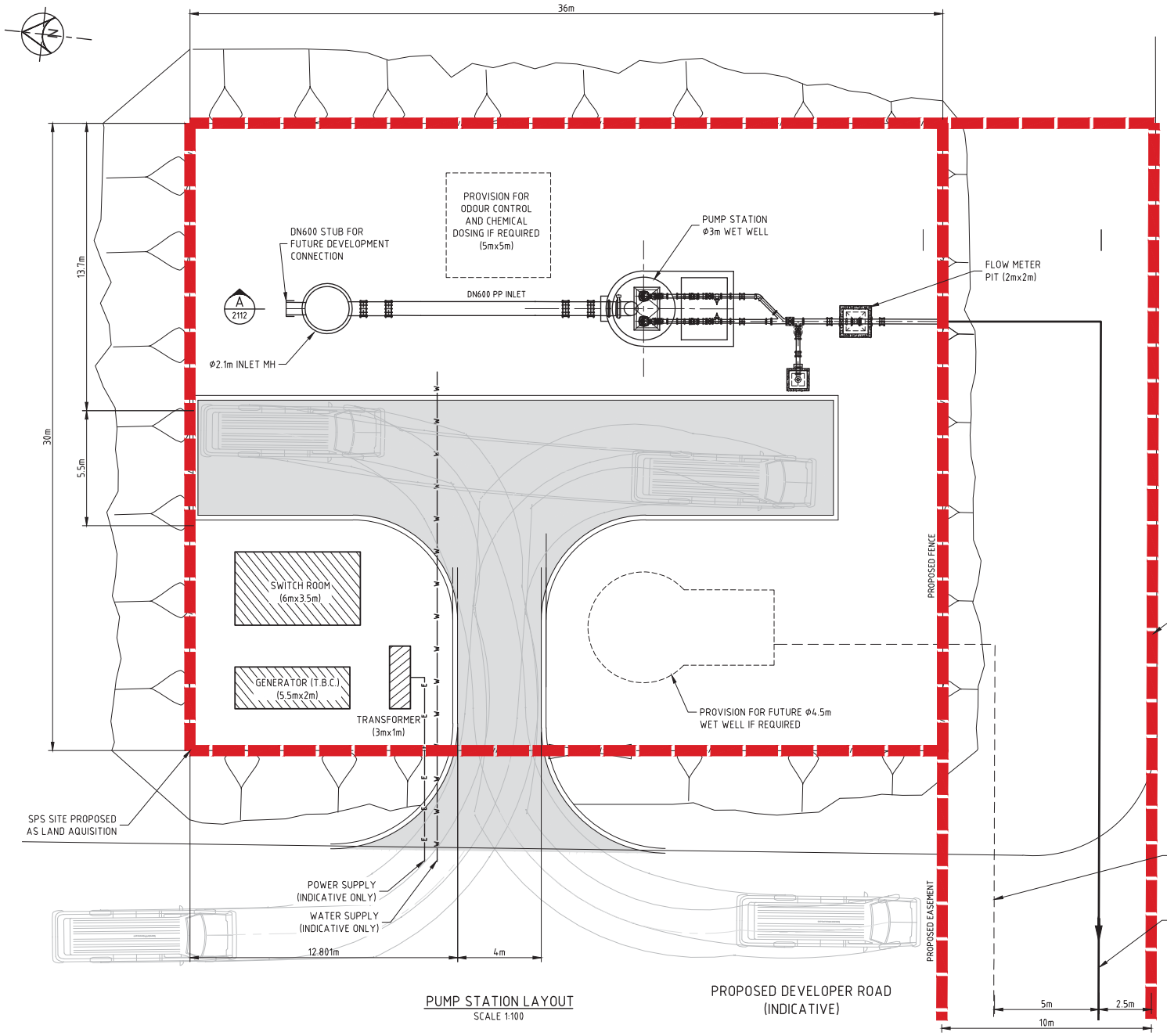
DESIGN COMPANY: LoganWIA

REV.: B

TECH SERVICES DRG No.:

CAD FILE: T:\LS014 FLINDERS\02 - RIVERBANK EAST\020 Acad\LS014-02-S-DWG-CI-2100.dwg

DATE PLOTTED: 06 October, 2017 - 12:43pm



PUMP STATION LAYOUT
SCALE 1:100

NOTES:

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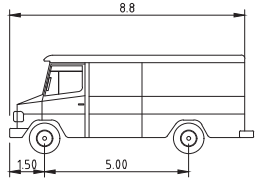
2. ACCESS BASED ON PRELIMINARY ROAD DESIGN LAYOUT FROM DEVELOPER. TEMPORARY ACCESS MAY BE REQUIRED FROM BUSHLAND ROAD.
3. ULTIMATE PUMPS AND RISING MAIN PIPEWORK OMITTED FOR CLARITY.
4. PUMP STATION OVERFLOW SHALL BE DIRECTED FROM MUNICIPAL INFRASTRUCTURE TO NEARBY STREAM TO THE NORTH OF THE PUMP STATION (TO BE CONFIRMED DURING DETAILED DESIGN PHASE).



SERVICE VEHICLE PROFILE (8.80m LONG)
(AS PER SEQ-SPS-1102-2)

	METERS
WIDTH	2.50
TRACK	2.50
LOCK TO LOCK TIME	6s
STEERING ANGLE	38.7°

0 1 2 4 6m
A1 - 1:100

PRELIMINARY

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REV. No.	DESCRIPTION	DRAWN	DESIGNED	APPR.	DATE
B	FOR LS014 PRELIMINARY DESIGN REPORT	PS	LM	DS	04/10/2017
A	FOR 30% DOAR WORKSHOP	PS	LM	DS	14/07/2107

DESIGNED BY: L.MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No. LS014-02
FILE REFERENCE:
SHEET SIZE A1
DATUM : AHD GDA 94 ZONE 56.

LOGAN WATER INFRASTRUCTURE ALLIANCE

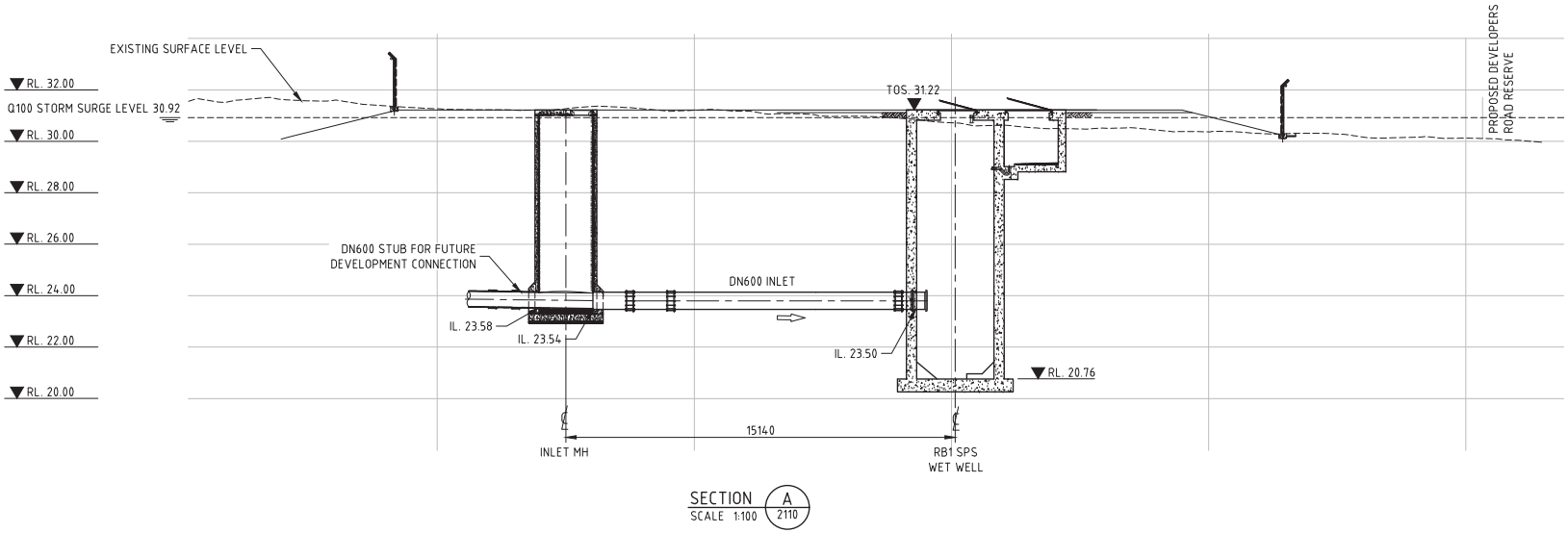
Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS

DRAWING TITLE:
RB1 PUMP STATION
SITE GENERAL ARRANGEMENT

DRAWING No. LS014-02-S-DWG-CI-2110
DESIGN COMPANY: LoganWIA
REV. B
TECH SERVICES DRG No.

CAD FILE: T:\LS014 FLINDERS\02 - RIVERBANK EAST\020 Acad\LS014-02-S-DWG-CI-2110.dwg
DATE PLOTTED: 06 October, 2017 - 12:43pm



SECTION A 2110
SCALE 1:100

NOTES:

1. FOR GENERAL NOTES REFER TO DWGS LS014-00-S-DWG-ND-XXXX (T.B.C.).
2. WET WELL DIAMETER SIZED FOR ULTIMATE PWWF.
3. INLET LEVEL BASED ON NETWORK MODELLING WITH ASSUMED MUNICIPAL INFRASTRUCTURE WITH NO DEVELOPER INPUT.

0 1 2 4 6m
A1 - 1:100

PRELIMINARY

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B	FOR LS014 PRELIMINARY DESIGN REPORT	PS	LM	DS	04/10/2017
A	FOR DOAR WORKSHOP	PS	LM	DS	14/07/2107

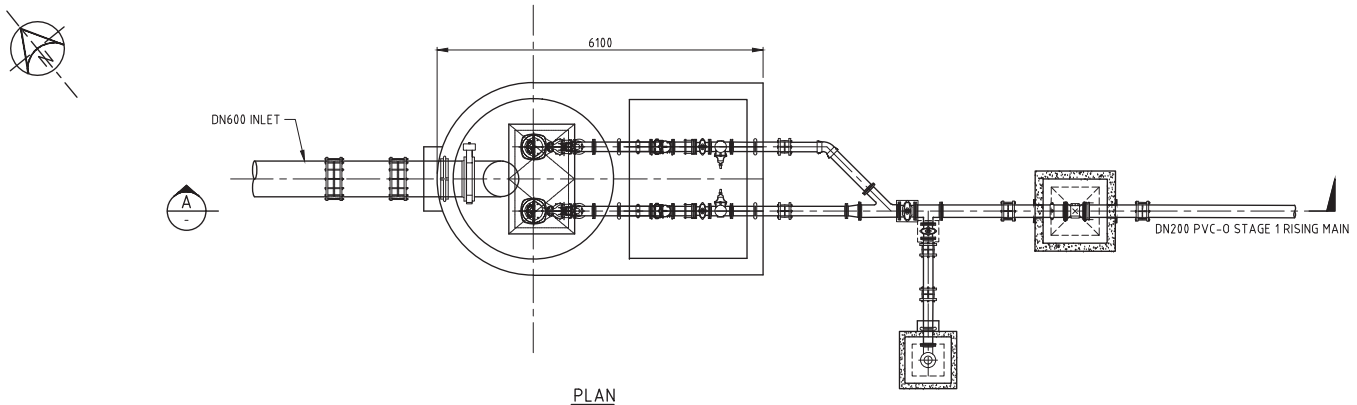
DESIGNED BY: L.MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No.: LS014-02
FILE REFERENCE:
SHEET SIZE: A1
DATUM : AHD
GDA 94 ZONE 56.

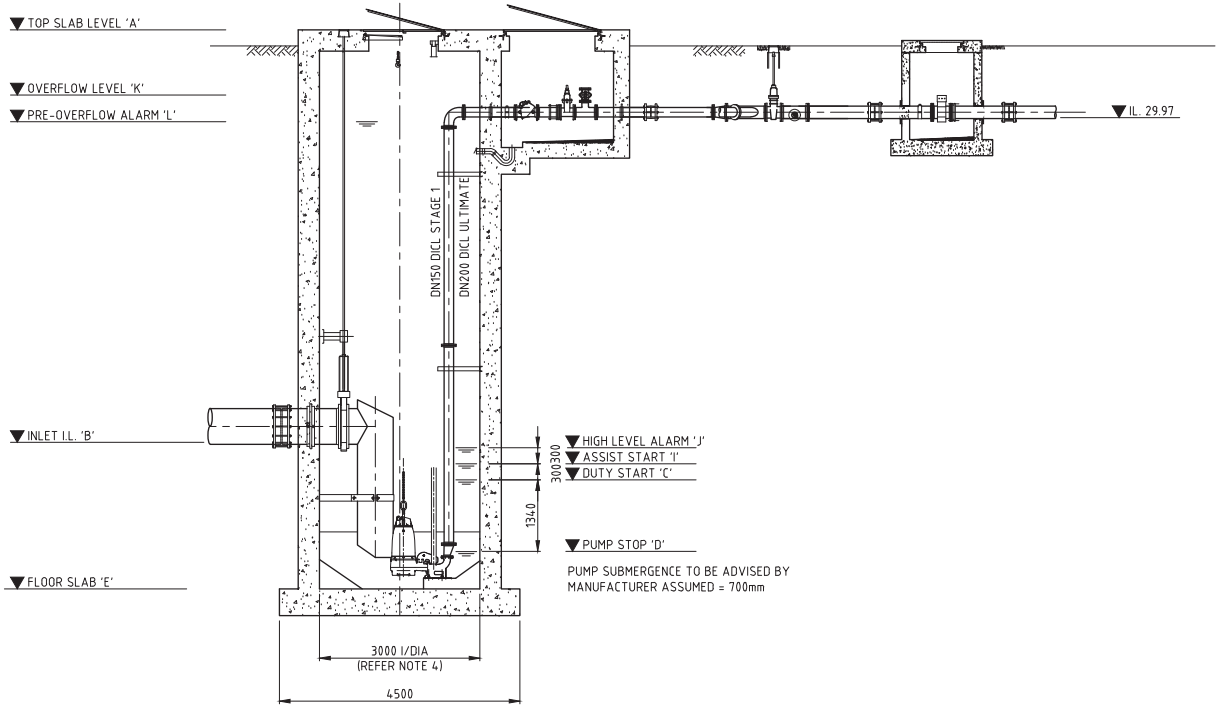
LOGAN WATER INFRASTRUCTURE ALLIANCE
Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS
DRAWING TITLE: RB1 PUMP STATION SECTIONS AND DETAILS
DRAWING No.: LS014-02-S-DWG-CI-2112
DESIGN COMPANY.: LoganWIA
REV.: B
TECH SERVICES DRG No.:

CAD FILE: T:\LS014 FLINDERS\02 - RIVERBANK EAST\03 Acad\LS014-02-S-DWG-CI-2112.dwg
DATE PLOTTED: 04 October, 2017 - 1:19pm



PLAN
SCALE 1:50



SECTION A
SCALE 1:50

PUMP STATION CONTROL LEVELS - (ULTIMATE)			
ITEM	DESCRIPTION	LEVEL (m) (A.H.D.)	HEIGHT FROM BASE (m)
A	TOP OF SLAB	31.22	10.46
B	INLET I.L.	23.50	2.74
C	DUTY START	22.80	2.04
D	PUMP STOP^	21.46	0.70
E	FLOOR SLAB	20.76	
F	PUMP STATION DEPTH		10.46
H	EXISTING GROUND LEVEL	30.92	10.16
I	ASSIST START	23.10	2.34
J	HIGH LEVEL ALARM	23.40	2.64
K	OVERFLOW LEVEL	30.00*	9.24
L	PRE-OVERFLOW ALARM	29.50	8.74
M	Q100 STORM SURGE LEVEL	30.92	10.16
PUMP OPERATING LEVELS AND DEFAULT SETTINGS SHALL BE AS PER THE SEQ-SP FUNCTIONAL DESCRIPTION			

^ PUMP STOP 2051 DESIGN HORIZON, STAGE 1, RL 22.47m AHD (BASED ON MINIMUM OPERATING VOLUME AND 8 PUMP STARTS PER HOUR WITH SINGLE PUMP DUTY FLOW)

OVERFLOW	
OVERFLOW LEVEL AT WELL	30.00m AHD*
LOWEST HOUSE FLOOR LEVEL OR MANHOLE LID	30.42m AHD*

* ASSUMED - NO DEVELOPERS PLANS AVAILABLE AT TIME OF PRELIMINARY DESIGN

DESIGN CRITERIA		
	SINGLE PUMP DUTY FLOW	PWWF
STAGE 1 (DUTY/ASSIST) - (2051)	46 L/s	58 L/s
STAGE 2 (DUTY/ASSIST/ASSIST) - (ULTIMATE)	190 L/s	235 L/s

PUMP (OR EQUIVALENT)		
INFORMATION	2051 FLOWS	ULTIMATE FLOWS
PUMP MAUNFACTURER	FLYGT	FLYGT
PUMP MODEL No	NP3202 SH 3 ~ 272	NP3231/665 3 ~ 480
IMPELLER DIAMETER	244mm	400mm
MOTOR SIZE	47 kW	105 kW
DUTY POINT (FLOW @ HEAD)	46.6 L/s @ 51m	190 L/s @ 43.5m

NOTES:
1. FOR GENERAL NOTES REFER TO DWGS LS014-00-S-DWG-ND-XXXX (T.B.C.).
2. PUMP STATION TO INCLUDE VSD AND SCADA CONTROL TO LIMIT INSTANTANEOUS FLOW PEAKS TO TREATMENT PLANT.
3. INLET LEVEL BASED ON NETWORK MODELLING WITH ASSUMED MUNICIPAL INFRASTRUCTURE WITH NO DEVELOPER INPUT.
4. WET WELL DIAMETER SIZED FOR 2051 PWWF.
5. ULTIMATE PUMPS AND PIPEWORK OMITTED FOR CLARITY.
6. PUMP STATION OVERFLOW TO BE DIRECTED FROM MUNICIPAL INFRASTRUCTURE TO NEARBY EPHEMERAL STREAM (TO BE CONFIRMED DURING DETAILED DESIGN).

0 0.5 1.0 2.0 3.0m
A1 - 1:50

PRELIMINARY

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A	FOR 30% DOAR WORKSHOP	PS	LM	DS	14/07/2017

DESIGNED BY: L.MARANO
DRAFTED BY: P. SPEAR
DRAFTING CHECK BY: F. JESPERSEN
DESIGN CHECKED BY: RPEQ
APPROVED BY: RPEQ

REFERENCES
PROJECT No.: LS014-02
FILE REFERENCE:
SHEET SIZE: A1
DATUM : AHD GDA 94 ZONE 56.

LOGAN WATER INFRASTRUCTURE ALLIANCE
Proudly LOGAN CITY COUNCIL

PROJECT: FL1, RB1 AND RB2-2 WW PUMP STATIONS
DRAWING TITLE: RB1 PUMP STATION
WET WELL GENERAL ARRANGEMENT
DRAWING No.: LS014-02-S-DWG-CI-2120
DESIGN COMPANY.: LoganWIA
REV.: B
TECH SERVICES DRG No.

CAD FILE: T:\LS014 FLINDERS\02 - RIVERBANK EAST\023 Acad\LS014-02-S-DWG-CI-2120.dwg
DATE PLOTTED: 16 October, 2017 - 1:59pm