Waterfall Park Developments Ltd

Ayrburn Rezoning

Water, Wastewater and Stormwater Infrastructure Assessment

June 2018

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Waterfall Park Developments Ltd

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Water, Wastewater and Stormwater Infrastructure Assessment

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Waterfall Park Developments Ltd

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Water, Wastewater and Stormwater Infrastructure Assessment

1.0 Introduction

1.1 General

Fluent Infrastructure Solutions Limited (FS) has been engaged by Waterfall Park Developments Ltd to undertake a water, wastewater and stormwater infrastructure assessment for the proposed rezoning of Ayrburn Farm.

1.2 The Site

The proposed Ayrburn Farm development area is located to the north of Lake Hayes and approximately 3km southwest of Arrowtown, as shown in Figure 1.1 below. The eastern and western parts of Ayrburn Farm are flat and rolling land respectively located on river terraces above Mill Creek. Between the terraces is the Mill Creek river channel and an adjacent flood plain that follows the gentle gradient of Mill Creek as it flows south to Lake Hayes. Refer to Figure 1.1 below for the locality of the proposed Ayrburn Farm development area.

1.3 The Proposed Development Plan

For the purposes of this assessment, an assumed worst-case scenario of 200 residential dwelling units has been used. This has then been assumed to consist of approximately 100 small lots (less than 300m^2) and approximately 100 larger lots (greater than 300m^2). Other scenarios would consist of a smaller total number of lots and would likely consist of larger lots. This infrastructure assessment has been undertaken on the proposed maximum development scenario.

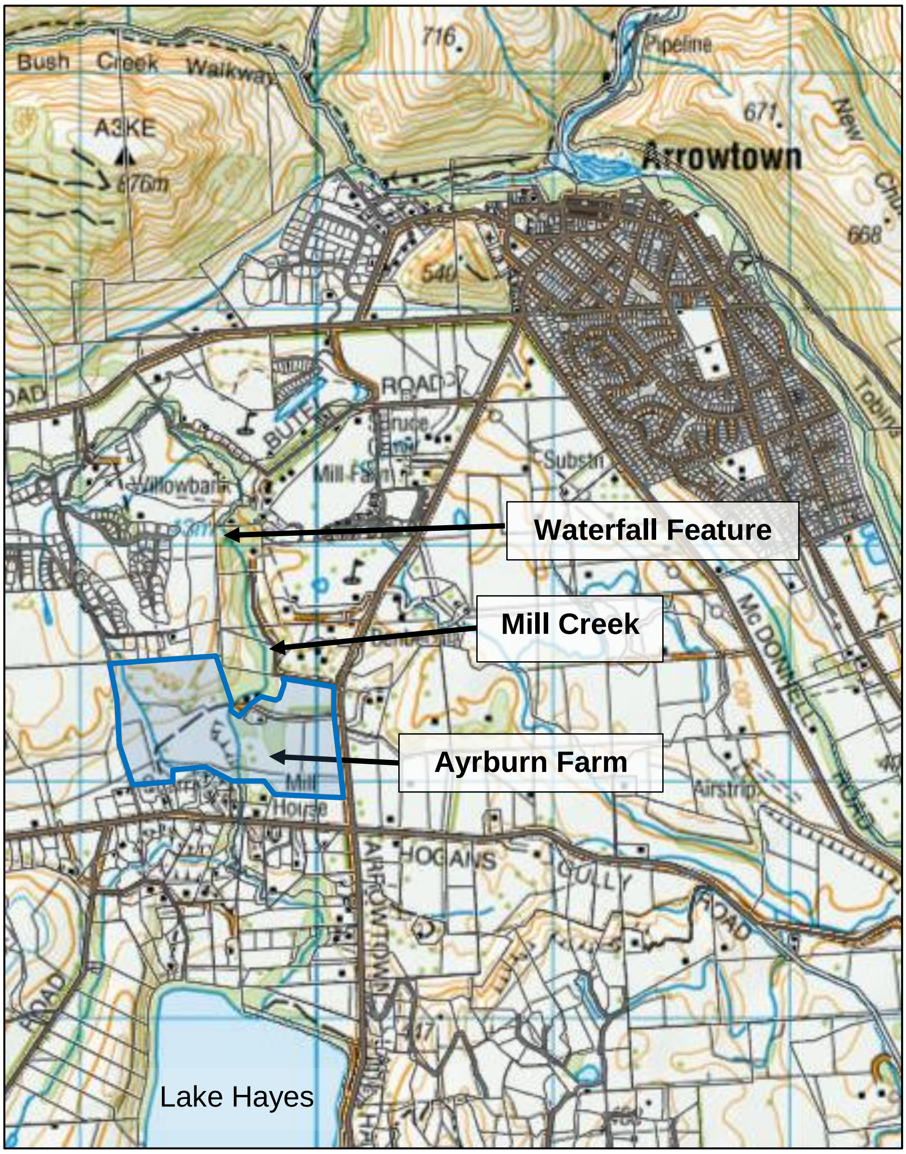


Figure 1.1: Site Location

2.0 Wastewater

2.1 Wastewater Collection and Conveyance System Design

The design, sizing and layout of the wastewater collection and conveyance network to service the proposed Ayrburn Farm development depends on the population served, the facilities to be provided and the capacity of the existing QLDC wastewater network. The following aspects have been investigated to assess wastewater collection and conveyance requirements:

- Population (i.e. the number of residential dwelling units)
- Wastewater production - both peak wet weather and peak dry weather
- Capacity of the existing QLDC infrastructure to convey the wastewater loads
- Wastewater pumping requirements

2.2 Wastewater Flows

The following design flows have been estimated for the proposed Ayrburn rezoning based on the maximum scenario of 200 residential dwelling units.

Table 3.1: Wastewater Design Flows

No. Dwellings	Average No. People / Dwelling	Average per Capita Daily Wastewater Production	Average Daily Wastewater Production (m^3/day)	Dry Weather Diurnal Peaking Factor	Peak Dry Weather Flow (L/s)	Dilution / Infiltration Peaking Factor for Wet Weather	Peak Wet Weather Flow
200	3	250	150	2.5	4.3	2.0	8.7

The wastewater design criteria for the development has been established based on Queenstown Lakes District Council's 2015 Land Development and Subdivision Code of Practice. Average dry weather design flows are based on 250L/p/d, with a peaking factor of 2.5 for the dry weather diurnal and a dilution/infiltration factor of 2 for wet weather, as for residential properties. The estimates are considered to be conservative.

The assessment identifies a peak wastewater design flowrate requirement of 8.7L/s. Allowing for the fact that the actual pumping rate can be higher than the design requirement (once a pump is selected in detailed design) it would be prudent to assume a discharge rate into the Queenstown system of, say, 9-10L/s.

2.3 Existing QLDC Infrastructure

Wastewater from Arrowtown is currently pumped to a manhole located east of the Arrowtown-Lake Hayes Road (manhole ID SM11784, refer Figure 3.1). This manhole also receives wastewater from Millbrook. Wastewater is conveyed from this manhole via a 300mm uPVC trunk main that runs along the Arrowtown-Lake Hayes Rd to the Bendemeer Wastewater Pump Station, located East of Lake Hayes. Although this main is classified as a rising main it is understood that the wastewater is conveyed by gravity from manhole SM11784 to the Bendemeer Pump Station.

Existing 150mm mPVC sewer mains drain wastewater from the properties south of the proposed Ayrburn residential development area and north of Lake Hayes to the Lake Hayes Sewer Pump Station #1 (located north of Lake Hayes), from where the wastewater is pumped to a 150mm mPVC gravity sewer main on the Arrowtown Lake Hayes Road. This gravity main also collects wastewater from properties east of Lake Hayes and drains to the Lakes Hayes Sewer Pump Station #2 which then pumps directly to the Bendemeer Pump Station.

Figure 3.1 presents an overview of the main existing sewer infrastructure in the vicinity of the Ayrburn residential Development area.

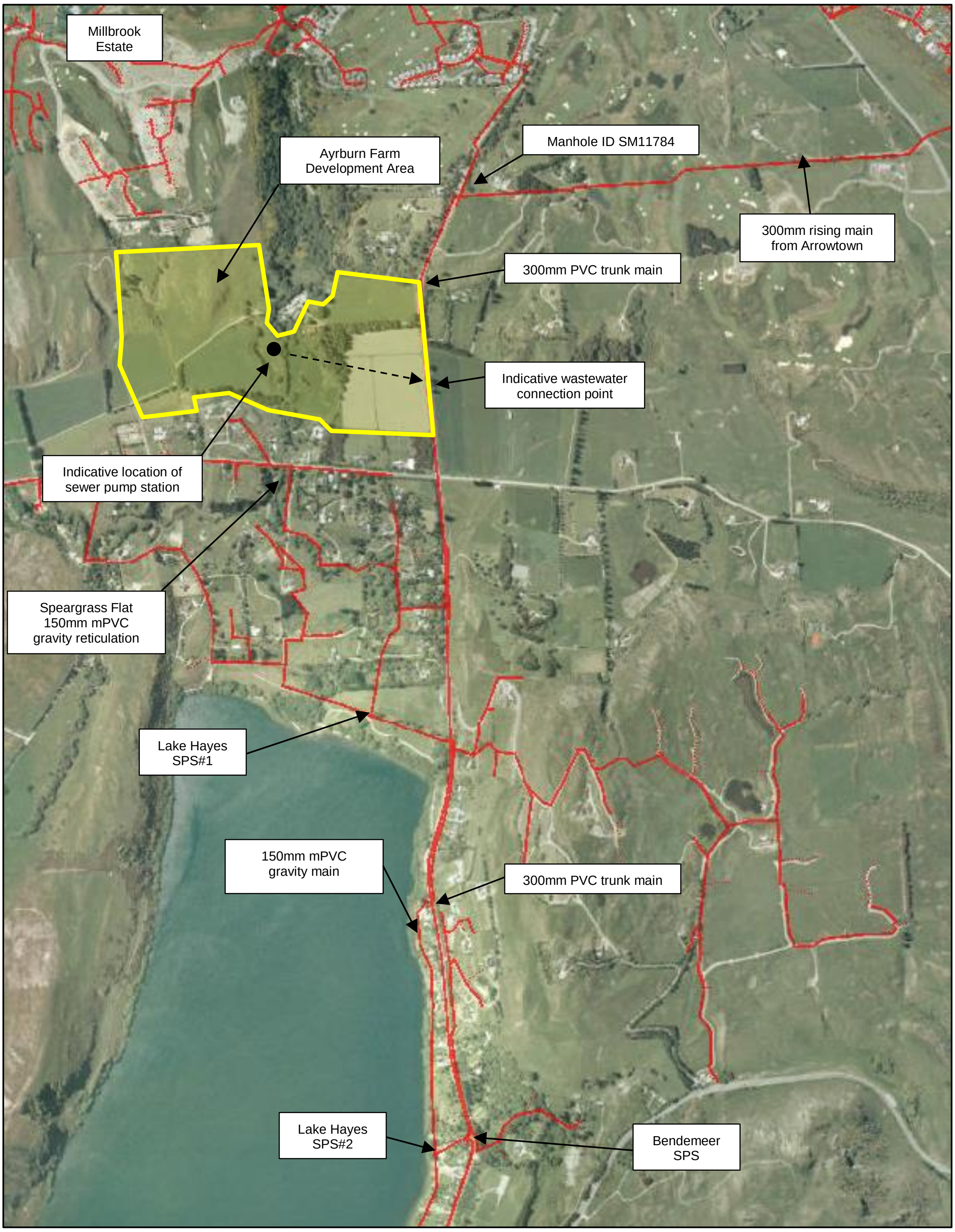


Figure 3.1: Schematic overview of existing sewer services in the vicinity of the Ayrburn residential development with the potential location of new sewer pump station indicated. The location of the pump station is subject to confirmation during detailed design

2.3.1 Capacity of existing Infrastructure

The capacity of the existing infrastructure to carry the loads from a residential development at Ayrburn Farm was modelled by QLDC's modelling consultants, BECA, during February 2018. Their report is provided in Appendix A. Flows modelled from the residential development were slightly higher than those assessed in Section 2.2 above due to differences in the maximum development scenario at the time of modelling. The results of the modelling found that the 300mm uPVC trunk main running past the proposed development has adequate capacity for the additional load for both the current, 2028 and 2058 design horizons without the need for infrastructure upgrades. The modelling indicated that the 150mm mPVC gravity reticulation north of Lake Hayes does not have adequate capacity to carry flows from the proposed residential development and therefore this option has not been progressed.

The proposed hotel development discussed in the BECA modelling report is not included in this submission and is the subject of a separate resource consent (RM180584). The modelling was included so that potential future scenarios could be accounted for in case they impacted on the other development. BECA's conclusion is that the existing infrastructure has adequate capacity for the proposed development. The proposed sewer pump station will be sized during the detailed design phase.

2.4 Wastewater Servicing for the Proposed Development

From the investigations undertaken, it is clear that the existing 300mm uPVC trunk main has adequate capacity to accept sewer flows from the proposed Ayrburn Farm development.

Wastewater servicing for the proposed development will likely comprise of conventional gravity sewer reticulation, falling to the proposed sewer pump station. Wastewater will be pumped from the sewer pump station into the 300mm mPVC trunk main, with a connection indicatively located at the point shown in Figure 3.1. The pipe route is anticipated to be under the proposed access road. The potential location of the pump station is shown on Figure 3.1, but the actual location and number of pump stations will be reviewed at the final design stage for greatest optimisation. The number and location of these pump stations is, therefore, subject to confirmation at the final design stage.

The wastewater pump station will be designed in a way that does not preclude it from being vested to Council.

3.0 Water Supply

3.1 Water Supply System Design

The design, sizing and layout of the water supply network to service the proposed Ayrburn residential development depends on the population served, the facilities to be provided and the water required to maintain the site landscaping. The following aspects relating to the water supply have been investigated to assess water supply requirements:

- Population (i.e. the number of residential dwelling units)
- Density and lot size (as very small lots would have low irrigation demands)
- Water demands - both peak and fire fighting requirements
- Water supply availability
- Water pressure requirements
- Water storage requirements
- Landscaping irrigation requirements
- Water quality requirements

3.2 Water Demand Assessment

3.2.1 Water Demands

The Queenstown Lakes District Council 2015 Land Development and Subdivision Code of Practice uses a design house occupancy of 3 people and a per capita daily water use of 700L/p/d to cover both domestic and irrigation requirements. This demand is considered to be appropriate for the larger lots (>300m^2), where the lots would have gardens that may be irrigated. For the smaller lots (<300m^2) there would be very little space available on the section for a garden and therefore water used for irrigation would be minimal. For these smaller lots a per capita water demand of 350L/p/d is considered to be appropriate.

The proposed maximum development scenario could have an indicative landscaped area of approximately 3,000m^2 and around 300 street trees requiring irrigation. Based on an average irrigation demand of 5mm/m^2/day and 10L/tree/day, this would equate to a total irrigation demand of approximately 18m^3/day. If irrigation is undertaken over a period of approximately eight hours each day, this would result in an irrigation demand of 0.63L/s.

Table 4.1 sets out the assessed domestic and irrigation demands for the proposed development. For this preliminary study, the peaking factors provided in the QLDC Land Development and Subdivision Code of Practice have been used for the peak hour water demand. These factors are considered to be high and will be reviewed further during the detailed design phase. They are however considered appropriate for this preliminary analysis in terms of providing a conservative demand estimate.

Table 4.1: Assessed Water Supply Design Volumes and Flows

Demand Type	No. Dwellings	No. people / dwelling	Daily Water Demand (L/p/d)	Peak Daily Water Demand (m^3/day)	Peak Hour Peaking Factor	Peak Hour Demand (L/s)
Large Sections (>300m^2)	100	3	700	210	6.6	16.04
Smaller Sections (<300m^2)	100	3	350	105	6.6	8.02
Irrigation	n.a.	n.a.	n.a.	18	n.a.	0.63
Total				**333.0**		**24.7**

Based on this assessment the following total peak day and peak hour demands for the maximum development scenario are estimated (excluding fire fighting):

- Peak Day Demand 333m^3/d
- Peak Hour Demand 24.7L/s

Peak day and peak hour demands would vary slightly depending on the development scenario progressed and would be further refined during concept and detailed design.

3.2.2 Fire Fighting Demands

The design of the water supply system is also required to meet the fire fighting flow and pressure requirements of SNZ PAS 4509 - NZ Fire Service Firefighting Water Supplies Code of Practice.

The residential properties would fall under the FW2 water supply classification (Table 1), requiring a minimum firefighting supply (Table 2), of a total of 25L/s from two hydrants, at a minimum pressure of 100kPa.

3.3 Existing Water Supply System

Properties south of the Ayrburn residential development area are supplied from the Lake Hayes water storage reservoir, located east of Lake Hayes. The Lake Hayes water storage reservoir has a minimum water level of 435m, compared to building levels of around 347 - 371m in the Ayrburn residential development area. These levels indicate that there should be adequate pressure available to supply the development for the Lake Hayes reservoir. The existing water reticulation network in the vicinity of the proposed development is shown in Figure 4.1 below. There is a 50mm rider main running along the Arrowtown-Lake Hayes Rd, north of Speargrass Flat Rd however this main is too small to supply the estimated demands from the proposed development. There are several water mains at the intersection of Arrowtown-Lake Hayes Rd and Speargrass Flat Rd, the largest of which is a DN 225mm mPVC water main.

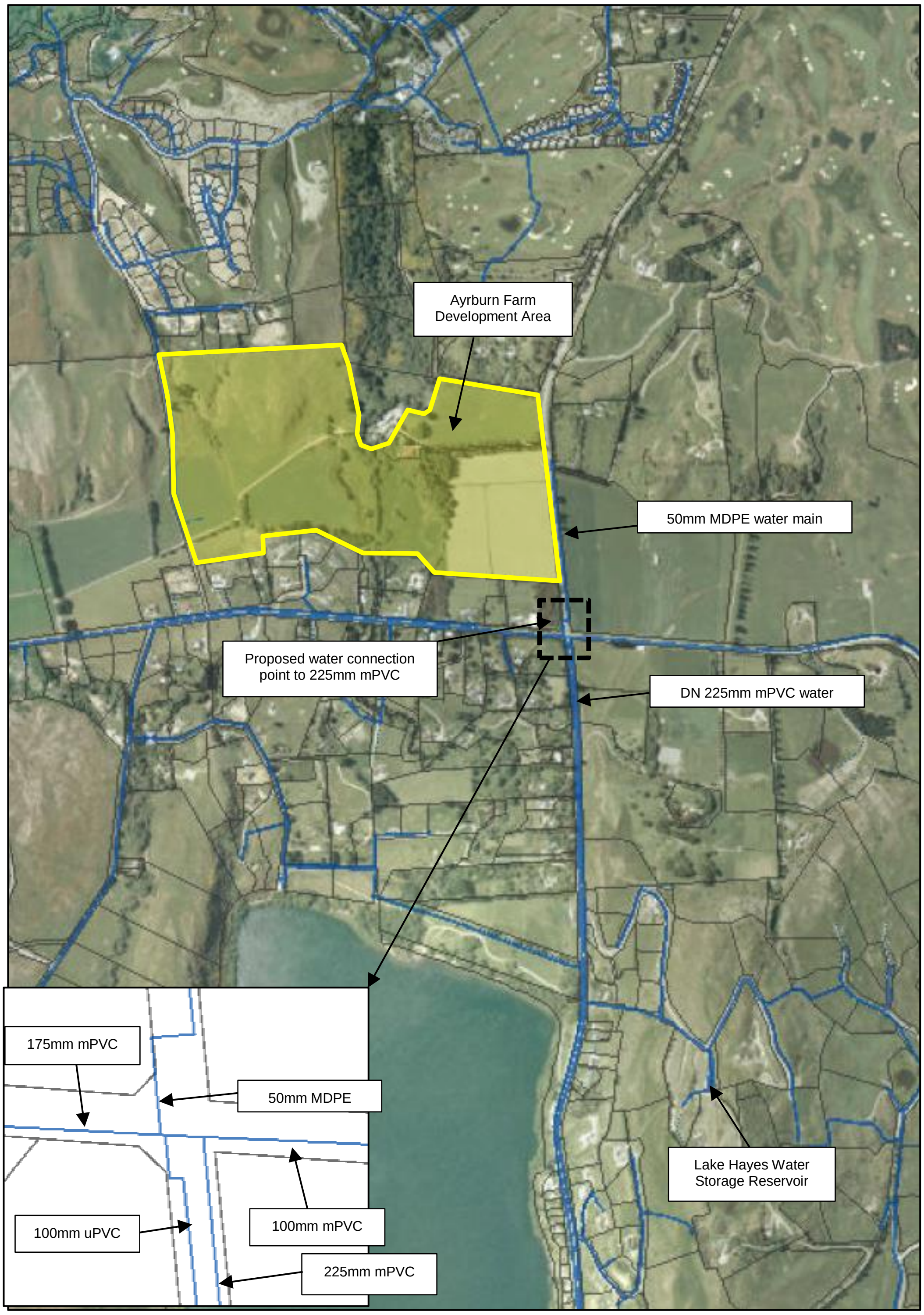


Figure 4.1: Schematic overview of existing water services in the vicinity of the Ayrburn residential development with the potential location of the proposed connection from the Ayrburn residential development

3.3.1 Capacity of existing Infrastructure

The capacity of the existing infrastructure to service the Ayrburn residential development was modelled by QLDC's modelling consultants, Mott MacDonald, during March and April 2018. Their report is provided in Appendix B. Flows modelled from the residential development were slightly higher than those assessed in Section 3.2.1 above due to differences in the maximum development scenario at the time of modelling. The results of the modelling indicated that when both the proposed Waterfall Park Hotel and the proposed Ayrburn residential development are supplied from the DN225mm mPVC water main for the current, 2028 and 2058 design horizons, levels of service in terms of minimum pressure, maximum headloss and fire fighting flows can be provided to the development. The report identifies that the highest elevation that would be serviceable for the residential development would be 395m.

The modelling also identified that with the residential development, headloss in the DN225mm along the Arrowtown-Lake Hayes Rd exceeds the QLDC level of service of 5m/km and identifies that this impact of the proposed residential development will need to be mitigated. The report notes that pressures at all properties that receive the minimum level of service pressure without the development remain above the minimum level of service pressure with the development, even with the high headloss in the DN225. Options for mitigating the high headloss along the Arrowtown-Lake Hayes Road will be discussed with QLDC during the detailed design phase and may include options such as duplicating a section of the DN225mm water main.

The proposed hotel development discussed in the Mott MacDonald modelling report is not included in this submission and is the subject of a separate resource consent (RM180584). The modelling was included so that potential future scenarios could be accounted for in case they impacted on the proposed development.

3.4 Water Servicing for the Proposed Development

From the investigations undertaken, it is clear that with some mitigation of the high headloss in the existing DN225 mPVC along the Arrowtown Lake Hayes Road, the existing network will have adequate capacity to provide the demands that are estimated to be required by the maximum development scenario of up to 200 lots at Ayrburn Farm. The DN225 mPVC main will be extended north along the Arrowtown-Lake Hayes Road to a suitable connection point adjacent to the proposed Ayrburn residential development area. The exact location of the connection point will be subject to detail design. Water servicing within the proposed development area will comprise of conventional gravity water reticulation.

The potential requirement for a pressure reducing valve to manage pressures to the lower-elevation areas of the development will be investigated during the detailed design phase.

4.0 Stormwater

4.1 Background

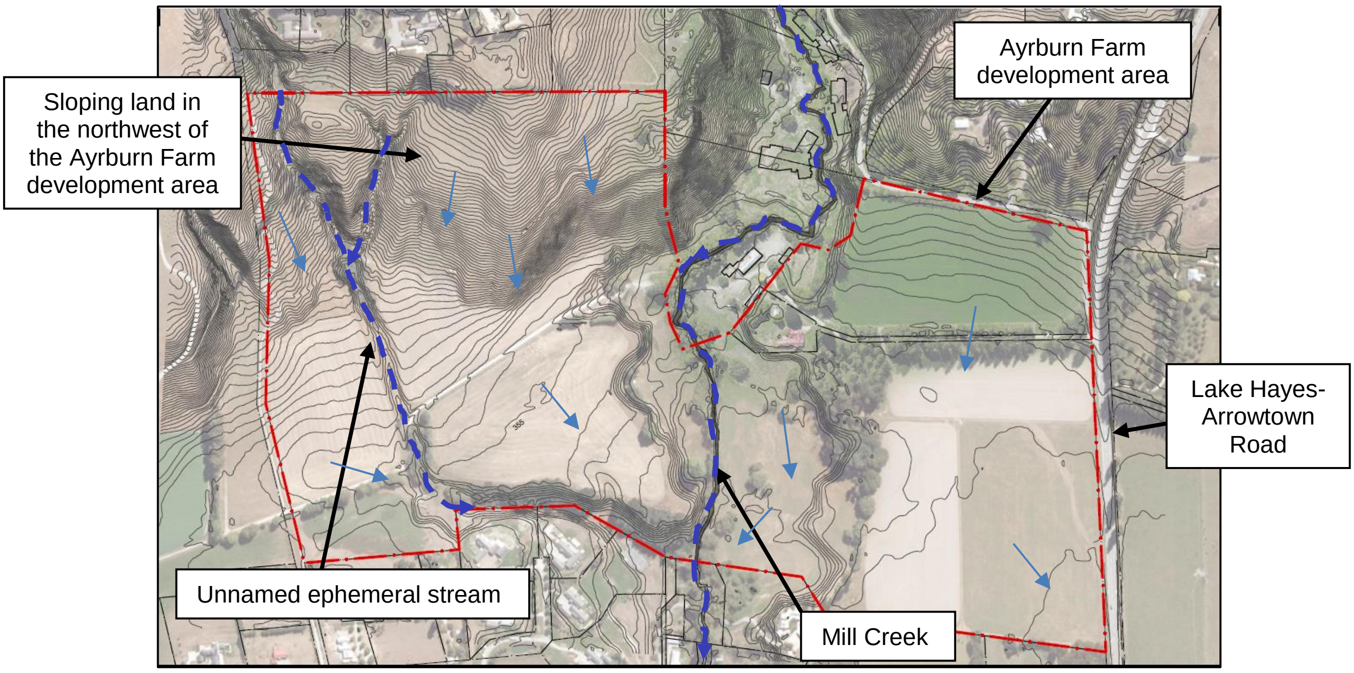
The topography of the land form at the proposed Ayrburn Farm residential development site means that the site drains towards Mill Creek and the unnamed ephemeral stream via natural overland flow paths. The site slopes down from the north to Mill Creek and to the Lake Hayes - Arrowtown Road. The northwestern stormwater catchments within the site have relatively steep topography. The soils in the area are characterised as predominantly alluvium being typically silty, sandy gravels overlying pond sediments and outside of the Mill Creek channel, the alluvium can be expected to have reasonable infiltration (Geosolve 2018, ref 150098.03). Groundwater seepage was encountered at depths between 0.6 - 4.4m below ground level for the neighboring road and hotel development.

The flood flow in Mill Creek at the Ayrburn Farm residential development is predominantly defined by the wide and flat valley floor upstream of the Waterfall Park waterfall. The valley floor upstream of the waterfall absorbs runoff from the surrounding catchment areas and delays and moderates the flood response in Mill Creek at Ayrburn. The stormwater runoff from the proposed Ayrburn Residential Development site into Mill Creek would be immediate compared to the flood response from the greater Mill Creek catchment upstream of the waterfall and therefore the peak stormwater runoff would typically discharge to Mill Creek hours before the peak flood flow from the upper Mill Creek catchment arrives. The stormwater and flood peak flows would not be coincident. For further information on flood flows refer to the Fluent Solutions memorandum MEM-18-05-21 AOP Q000391, May 2018.

There is no existing stormwater infrastructure on the Ayrburn Farm development site. There is also no existing stormwater infrastructure on the Arrowtown Lake Hayes Road, in the vicinity of the site.

The key features of the site are highlighted in the Figure 5.1 below.

Figure 5.1: Key features of Ayrburn Farm (arrows indicate overland flow direction)



4.2 Proposed Stormwater Management Concept

The proposed residential development would have four key surface types that generate runoff. These are:

- The access road
- Internal roads, parking areas and footpaths
- Roofs and other impervious areas within each lot
- Sloping land north of the development area

Each of these surfaces will generate stormwater with different characteristics. Although the detail has not been advanced yet, there are multiple design options available to properly manage and treat (if required) stormwater from each of these surfaces. Potential stormwater management options for each surface are described below. Assessment of these options and sizing and specific location of the stormwater management elements would be confirmed during the detailed design phase.

4.2.1 Access Road

Stormwater management for the Ayrburn Farm development access road has been set out in a separate report which is attached to this submission.

4.2.2 Internal Roads, Parking Areas and Footpaths

Stormwater generated from the road, parking areas and footpaths would typically be conveyed via kerb and channel to collector sumps, referred to as mud tanks, distributed along the roads. The primary stormwater conveyance system would likely be pipes or road side swales. Secondary overland flow paths would be either roads or specifically designed open channel flow paths. The primary conveyance and secondary flow paths would discharge to stormwater detention basins that would be provided with a controlled discharge structure to provide flow attenuation and stormwater quality management before discharge to Mill Creek.

The mud tanks, swales (if used) and detention basin would intercept contaminants prior to discharge to Mill Creek. Soakage areas may also be proposed depending on the layout and density of the residential development. If soakage to groundwater is considered further soil and ground water investigations would be required during the preliminary design phase.

4.2.3 Roofs and other Impervious Areas within each Lot

Management of stormwater generated from the house roofs and other impervious areas within each lot would depend on the density of the proposed residential development. If the maximum development scenario of up to 200 lots is progressed, each lot would likely have a stormwater lateral to convey the stormwater collected from the roof and other impervious areas into the road stormwater collection and conveyance system. If, however a less dense development scenario is progressed, the option of onsite stormwater disposal within each lot would be investigated.

4.2.4 Sloping Land North of the Development Area

Sheet flow running off the sloping land in the northwest of the development area would likely be collected in cut-off drains to protect the northern-most lots, depending on the location of the lots over this area. The water collected in the cut-off drains would be conveyed via dedicated overland flow paths to Mill Creek. It is likely that the stormwater from within the residential development would be kept separate to the runoff from the sloping land to facilitate stormwater quantitative and quality management control.

4.3 Stormwater Quality Management

The stormwater management approach will provide for comprehensive management of stormwater that falls on the residential lots, is intercepted by the road alignments and from the upper catchments to the north-west of the development area. The majority of the catchment area entering Mill Creek upstream of the Ayrburn Farm development is from pervious areas that are grassed and would have plantings of riparian vegetation or trees established as part of the landscape planning.

The primary potential contaminant of concern anticipated to be present in the stormwater generated from the residential development would be elevated suspended solids. Due to the relatively small size of the proposed development and relatively low traffic volumes that would be generated, oil and grease and heavy metal contamination load would be relatively low. The proposed stormwater treatment approach includes stormwater detention basins and possibly grass swales to facilitate the removal of suspended solids that would reduce contaminant loads to less than minor levels. Dedicated treatment devices will be considered during concept design based on the density and layout of the development scenario progressed and the associated contaminant loads and the relative levels of environmental risk.

Lead, zinc and copper metal contaminants are typically associated with road runoff. Any road contaminants would combine with suspended sediments and would be settled out in the swales. Nutrients (Nitrogen and Phosphorus) are not generated by vehicle activities and are unlikely to be generated from the lots and are therefore not of concern.

During the construction period there would be an increased risk of erosion, increased suspended solids load and increased hydrocarbon spill risk. An Earthworks Management Plan would be developed for the construction period and would be provided to QLDC for acceptance. The Plan would detail specific measures for sediment and erosion control during earthworks. The Earthworks Management Plan would also specify dedicated areas for refueling and storage of contaminants to mitigate the potential risk of hydrocarbon spills reaching Mill Creek.

4.4 Regional Plan: Water for Otago

Section 12.B.1.8 of the RPW provides rules relevant to the discharge of stormwater to water, or to land where it may enter water. The discharge of stormwater is a permitted activity provided that conditions (a) to (d) are met. Table 5.4 below lists each of these conditions and specifies how compliance with these conditions is achieved.

Table 5.4: Compliance with Rule 12.B.1.8:

Rule 12.B.1.8 Conditions	**Compliance with Conditions**
The discharge of stormwater from a reticulated stormwater system to water, or onto or into land in circumstances where it may enter water, is a permitted activity, providing:	
(a) Where the system is lawfully installed, or extended, after 28 February 1998: *(i) The discharge is not to any Regionally Significant Wetland; and* *(ii) Provision is made for the interception and removal of any contaminant which would give rise to the effects identified in Condition (d) of this rule; and*	(i) The discharge is not to a Regionally Significant Wetland. (ii) Mud tanks, detention basins and potentially grassed swales will be provided for the removal of suspended solids.
(b) The discharge does not contain any human sewage; and	The stormwater would not contain human sewage. Sewage is to be discharged to the QLDC wastewater collection and treatment network.
(c) The discharge does not cause flooding of any other person's property, erosion, land instability, sedimentation or property damage; and	The design of the stormwater management system would ensure that the discharge does not cause flooding, erosion, land instability, sedimentation or property damage.
(d) The stormwater discharged, after reasonable mixing, does not give rise to all or any of the following effects in the receiving water: *(i) The production of any conspicuous oil or grease films, scums or foams, or floatable or suspended materials; or* *(ii) Any conspicuous change in the colour or visual clarity; or* *(iii) Any emission of objectionable odour; or* *(iv) The rendering of fresh water unsuitable for consumption by farm animals; or* *(v) Any significant adverse effects on aquatic life.*	The proposed use of primary treatment, detention storage with controlled discharge would mean the stormwater discharge would not give rise to these effects after reasonable mixing.

The conclusion of the stormwater discharge assessment of effects, see Section 4.4.1, demonstrates compliance with the permitted activity rules for RPW.

4.4.1 Stormwater Effects Mitigation

The stormwater quality mitigation measures are considered to be adequate to ensure that stormwater discharge from the road would comply with rule 12.B.1.8 of the RPW and the effects on Mill Creek would be less than minor. Implementation of an Earthworks Management Plan would ensure compliance with rule 12.B.1.8 of the RPW during the earthworks period.

4.5 2015 QLDC Land Development and Subdivision Code Practice

4.5.1 Key Clauses

The 2015 QLDC Land Development and Subdivision Code of Practice (COP) (Cl 4.3.5) requires that a primary stormwater system be designed to convey, as a minimum, a 20 year ARI runoff flow taking into account climate change. Where a secondary flow path is available, the secondary flow path is required to convey the balance of a 100 year ARI flow without damage to property and with freeboard. If a secondary flow path is not available, the primary system is required to convey a 100 year ARI flow with freeboard.

The Mill Creek catchment immediately upstream of Ayrburn Farm is relatively confined and flows to Mill Creek. Mill Creek is therefore the primary flow path for stormwater discharging from the site. The road crossings over Mill Creek along the access road have been designed for the 100 year ARI flow in Mill Creek. Other stormwater conveyance facilities would, where appropriate, be designed for a 20 year ARI flood flow. Building floor levels would be set based on the freeboard required by the COP. The flood management design for facilities affected by Mill Creek is the subject of a separate report prepared by Fluent Solutions.

Compliance with these requirements will be ensured during the concept and detailed design phases.

4.6 Comments on Mr Langham's Statement of evidence

In response to paragraph 61.2 of Mr Langham's Statement of Evidence, the 'wedge' of land sought to be rezoned Waterfall Park Zone has been assessed from an infrastructure perspective as part of the Waterfall Park Developments Ltd (WPDL) hotel resource consent (RM180525). There is no infrastructure reason not to rezone this 'wedge' to Waterfall Park Zone.

5.0 Summary

A high level three-waters infrastructure overview of the proposed Ayrburn Farm Residential Development has found that all infrastructure requirements for the development can be met by existing and new services.

Wastewater servicing will be met by an internal gravity sewer collection network that will run to a wastewater pumpstation delivering to a connection point to existing sewer reticulation at Arrowtown-Lake Hayes Rd.

Water demand can be met by gravity supply from the Lake Hayes scheme via a connection point to existing water reticulation at the intersection of Speargrass Flat and Arrowtown-Lake Hayes Rd. Mitigation of high head loss in the water main along the Arrowtown-Lake Hayes Road would likely be required.

Stormwater generated from the road, parking areas and footpaths would typically be conveyed via kerb and channel to collector sumps distributed along the roads and conveyed via pipes or road side swales. Stormwater generated from the building roofs and other impervious surfaces within each lot would either be discharged to the primary stormwater conveyance system or disposed of onsite, depending on the density of the development. Secondary overland flow paths would be either roads or specifically designed open channel flow paths. The primary conveyance and secondary flow paths would discharge to stormwater detention basins that would be provided with a controlled discharge structure to provide flow attenuation and stormwater quality management before discharge to Mill Creek. Suspended sediment would be settled out in mud-tanks, swales (if used) and detention basins and further treatment requirements would be assessed during the concept and detailed design phases.

APPENDIX A

Wastewater Modelling Report

Report

Waterfall Park Development Wastewater Modelling

Prepared for Queenstown Lakes District Council (Client)

By Beca Limited (Beca)

7 February 2018

Revision History

Revision Nº	Prepared By	Description	Date
A	Tracey Myers	Draft Report	8/2/18
B	Tracey Myers	Report updated with Developer's Comments	16/2/18
C	Tracey Myers	Final Report	19/04/18

Document Acceptance

Action	Name	Signed	Date
Prepared by	Tracey Myers	T Myers	23/04/18
Reviewed by	Dan Stevens		24/04/18
Approved by	Dan Stevens		24/04/18
on behalf of	Beca Limited		

Contents

Appendices

1 Background

Beca Limited (Beca) have been engaged by Queenstown Lakes District Council (QLDC) to model a new development at Waterfall Park, Lake Hayes (see Appendix A, Development Plan). Modelling work has been completed previously for this development. However, the development has now expanded, and further modelling work is required.

2 Demand and Loads to the Wastewater Network

2.1 Development Demand Assessment

We have been given average, and peak flow information by the developer. We have converted these flows into population equivalents, as this is what the model uses. The daily flow per person in the QLDC Land Development and Subdivision Code of Practice is 250 L/day. The population equivalent for the average flows are given in Table 1 below.

Table 1 - Population Equivalent for Flows

Development Type	Average Daily Flows (L/s)	Total Daily Flows (m^3)	Population Equivalent (rounded)
Hotel	2.9	247.1	988
Residential	1.8	156.4	626

We have, therefore, used a population equivalent of 1,614 in the wastewater model to represent the flows.

Appendix A, **Figure 1** shows the sewer network in the vicinity of the new development, and includes the modelled network for the development.

2.2 Loads in the Wastewater Network

The peak wet weather flows entering the Lake Hayes #1 and #2, and Bendemeer pump stations are given in Table 2 below. Appendix B, **Figures 2 to 10**, show the peak wet weather flows entering the pump stations during the 2 year ARI event. Appendix C, **Figures 11 to 19**, show the flows discharging from the pump stations during the same period. No pump curve has been provided for the Lake Hayes #2 pump station, and a fixed flow rate has been set at 16 L/s for both pumps.

Table 2 - Peak Flows Entering Lake Hayes #1 and #2 Pump Stations

Pump Station	Current WWF (L/s)	2028 WWF Including Growth Model (L/s)	2028 WWF with Growth Model and Waterfall Park Flows (L/s)
Lake Hayes #1	15	21	21
Lake Hayes #2	24	25	25
Bendemeer	146	148	157

We removed the Waterfall Park flows that were previously included in the growth model before we simulated the runs. The Waterfall Park development has a peak dry weather flow of 11.7 L/s, and a peak wet weather flow of 23.4 L/s.

3 Design Horizon Checks

We have simulated three scenarios, using the 2028, and 2058 design horizons. The simulations have been run with a 2year ARI design storm event, which is the standard Level of Service for QLDC. Appendix D, **Figures 20 to 23** show the peak wet weather flow in the long sections.

3.1 Scenario 1 – DWF Gravity Fed to Speargrass Flat Road

This is the developer's preferred option. In the previous modelling work, the network had insufficient capacity to take the extra flows from Waterfall Park. Therefore, we were requested to initially simulate dry weather flow from the development, but with wet weather flows in the rest of the model. Simulating the dry weather flow only allows us to see the impact of minimising the development inflow and infiltration on the existing network.

Without the development, one manhole (SM11957) floods downstream of the Lake Hayes #1 PS.

When the full development is added, three manholes flood upstream of the Lake Hayes #1 PS. These manholes are SM11804, SM11807, and SM11930.

The capacity in the current network is 7.1 L/s. Adding a peak residential flow of 4.5 L/s leaves the remaining capacity as 2.6 L/s, without adding any storage at the development. Therefore, the remaining flow from the development will need to be stored.

3.1.1 Scenario 1a – Residential DWF Gravity Fed to Speargrass Flat Road

We simulated the DWF for only the residential development, with the wet weather flows in the rest of the model. The network upstream of the Lake Hayes #1 pump station has capacity to take these flows.

3.1.2 Scenario 1b – Hotel DWF Gravity Fed to Speargrass Flat Road

We simulated the DWF for only the hotel development, with the wet weather flows in the rest of the model. One manhole (SM11930) floods. Therefore, the network upstream of the Lake Hayes #1 pump station does not have the capacity to take the hotel flows.

3.2 Scenario 2 – DWF Pumped to Arrowtown-Lake Hayes Road

We modelled a pump station, and 300mm diameter rising main to take the flows to connect into the existing network on Arrowtown-Lake Hayes Road. The pump rate is 15 L/s. We then simulated the model with dry weather flow from the development, but with wet weather flows in the rest of the model. We considered whether or not the new pump station could run at the same time as the peak flows from the Arrowtown-Lake Hayes pump station. We found that the new pump station has insignificant impact on the existing pump station.

Without the development, one manhole (SM11957) floods downstream of the Lake Hayes #1 PS. Adding the development does not create any more areas of flooding.

3.3 Scenario 3 – WWF Pumped to Arrowtown-Lake Hayes Road

This scenario is the same as scenario 2, except we simulated the 2 year ARI event through the development as well. The pump rate remains 15 L/s. As before, we managed the pumping from the development using Real-Time Control. We also simulated the model without the Real-Time Control.

During the 2028 design horizon, SM11957 floods. This is regardless of whether the development is modelled or not. The flood volume is 75m^3, during the 2028 design horizon.

During the 2058 design horizon, two manholes flood (SM11952 and SM11957) downstream of the Lake Hayes #1 PS without the development. The flood volume is 75m^3.

With the development included, no extra manholes flood. As with Scenario 2, the new pump station has an insignificant impact on the existing pump station. Table 3 below details the pressure in the 300mm diameter pipe at the connection point for the 2058 design horizon.

Table 3 – Pressure at Connection Point for Scenario 3

Design Horizon	Static Pressure (m)	Pressure with No Waterfall Park Flow (m)	Pressure with Arrowtown and Waterfall Park Flows (m)
2058	4.6	4.8	5

4 Future Upgrades Required

Jayne Richards at Fluent Solutions Ltd requested that we look at the maximum flow that can be added to both Scenarios 1 and 3.

4.1 Scenario 1a

The capacity in the current network is 7.1 L/s. Adding a peak residential flow of 4.5 L/s leaves the remaining capacity as 2.6 L/s, without adding any storage at the development. Therefore, the remaining flow from the development will need to be stored.

4.2 Scenario 3

A Capital Scheme, Lake Hayes #2 PS, is already included in the current Capital Programme. This scheme includes upgrades that will relieve the flooding anticipated in 2028. In terms of effect on the network, we would recommend that Scenarios 2 and 3 are taken further. Neither of those scenarios affect the current flooding.

No other upgrades are required to contain the extra flows from Waterfall Park development during the 2028 or 2058 design horizons.

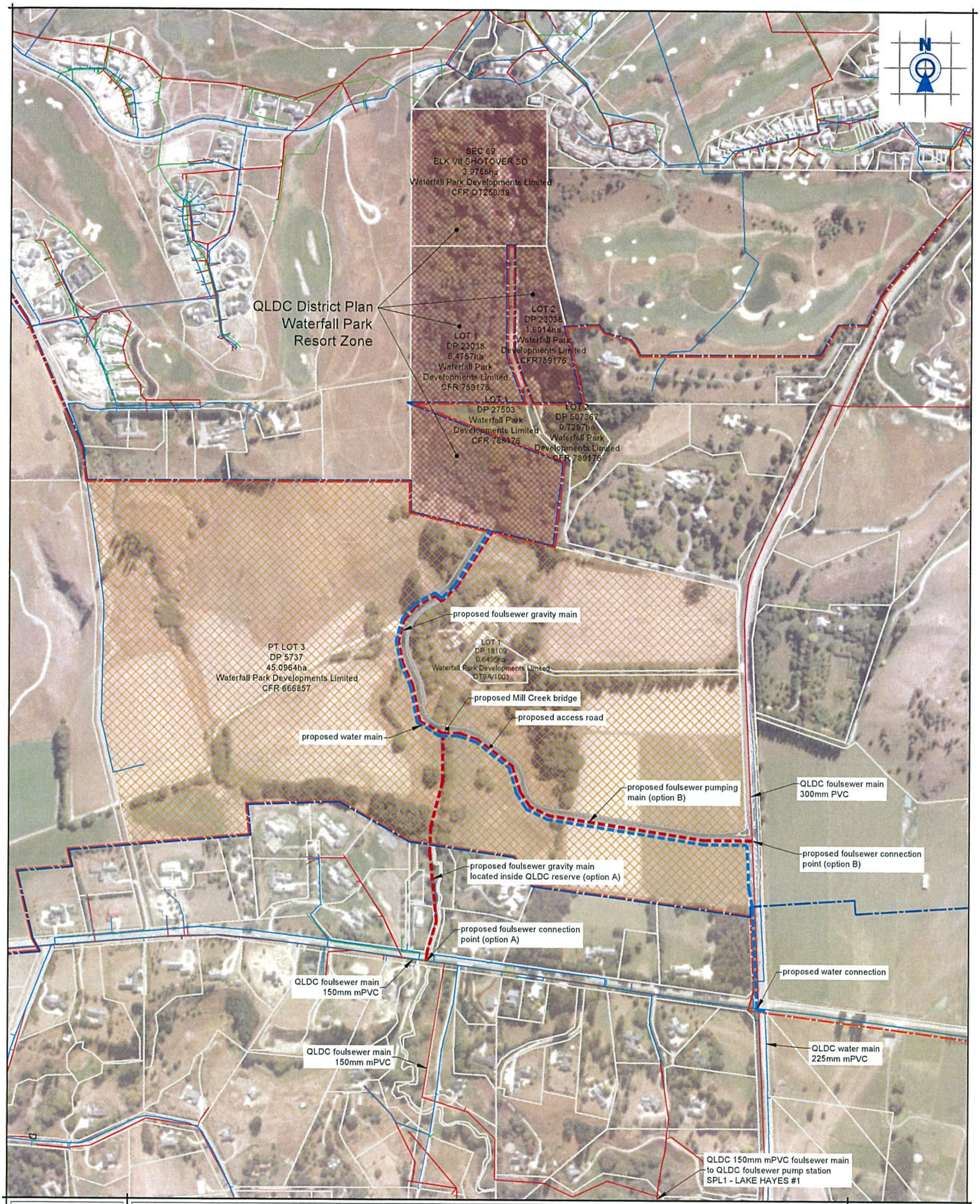
5 Conclusion

The sewer network between Speargrass Flat Road and Lake Hayes #1 PS has insufficient capacity to take all of the dry weather flows from the Waterfall Park development. After adding the residential development only, there is spare capacity of 2.6 L/s peak flow in the Speargrass Flat Road network.

A Capital Scheme, Lake Hayes #2 PS, is already included in the current Capital Programme. This scheme includes upgrades that will relieve the flooding anticipated in 2028. In terms of effect on the network, we would recommend that Scenarios 2 and 3 are taken further. Neither of those scenarios affect the current flooding, and no other upgrades would be required to the sewer network.

Appendix A

Plans



QUEENSTOWN:
Terrace Junction,
1092 Frankton Road,
PO Box 2645,
Queenstown 9349.
T 03 441 4715
E queenstown@ppgroup.co.nz

Notes:
1. Lot boundaries shown in grey
2. Existing QLDC services from QLDC GIS (15/12/17)

QLDC sewer scheme boundary
QLDC water scheme boundary
QLDC sewer infrastructure
QLDC water infrastructure
QLDC drainage infrastructure

0m 100m 200m 300m

PATERSONPITTSGROUP
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0800 PPGROUP

Client/Location:
Waterfall Park Developments Ltd
Sec 69 Blk VII Shotover SD, Lots 1 & 2
DP 23038, Lot 1 DP 27503, Lots 1 & 2
DP 507367, Pt Lot 3 DP 5737 and Lot 1 DP 18109

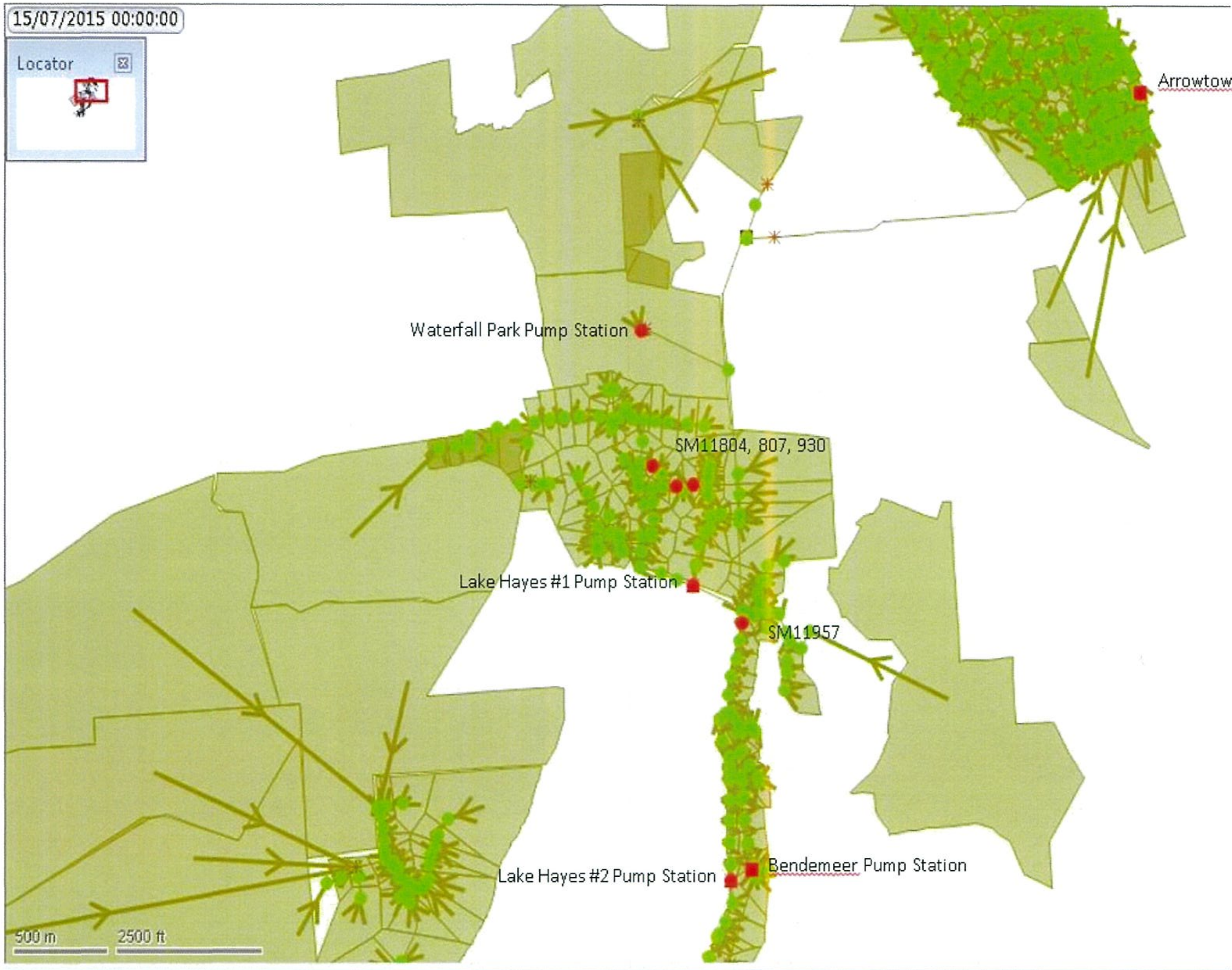
Purpose/Drawing Title:
Proposed Connections to Council Services
Plan for Discussion

Surveyed by:		Original Size:	Scale:
Designed by:		A3	1:5000
Drawn by:	SJP		
Checked by:			
Approved by:			DO NOT SCALE
Job Ref: Q6388 - 26 - 2		Sheet No: 1	Revision No: C / Date Created: 19/12/2017

PLOT DATE: 19 December 2017 - 6:25 PM BY: Stephen Popenhagen

J:\JOBS\Q_6388__WINTON PARTNERS WATERFALL PARK\CAD\Q6388-26-2-1C_SERVICES CONNECTIONS.DWG

Figure 1: Sewer network, with pump stations, and flooding manholes highlighted (add note showing SM11957)



Appendix B

Inflows to the Lake Hayes Pump Stations

Figure 2: Current peak wet weather inflow to Lake Hayes #1 PS

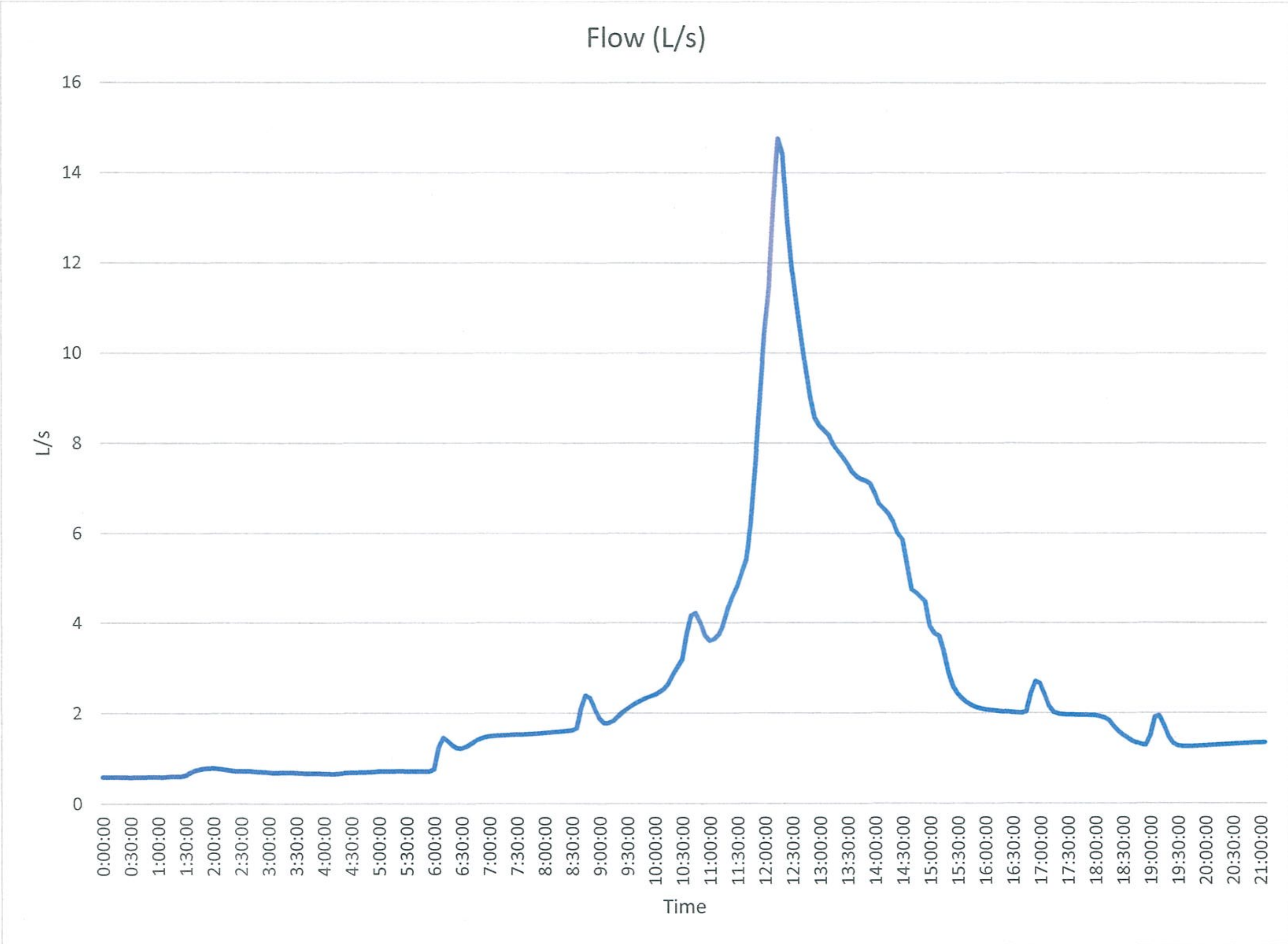


Figure 3: Peak wet weather inflow to Lake Hayes #1 PS using 2028 Growth Model (No Waterfall Park Development)

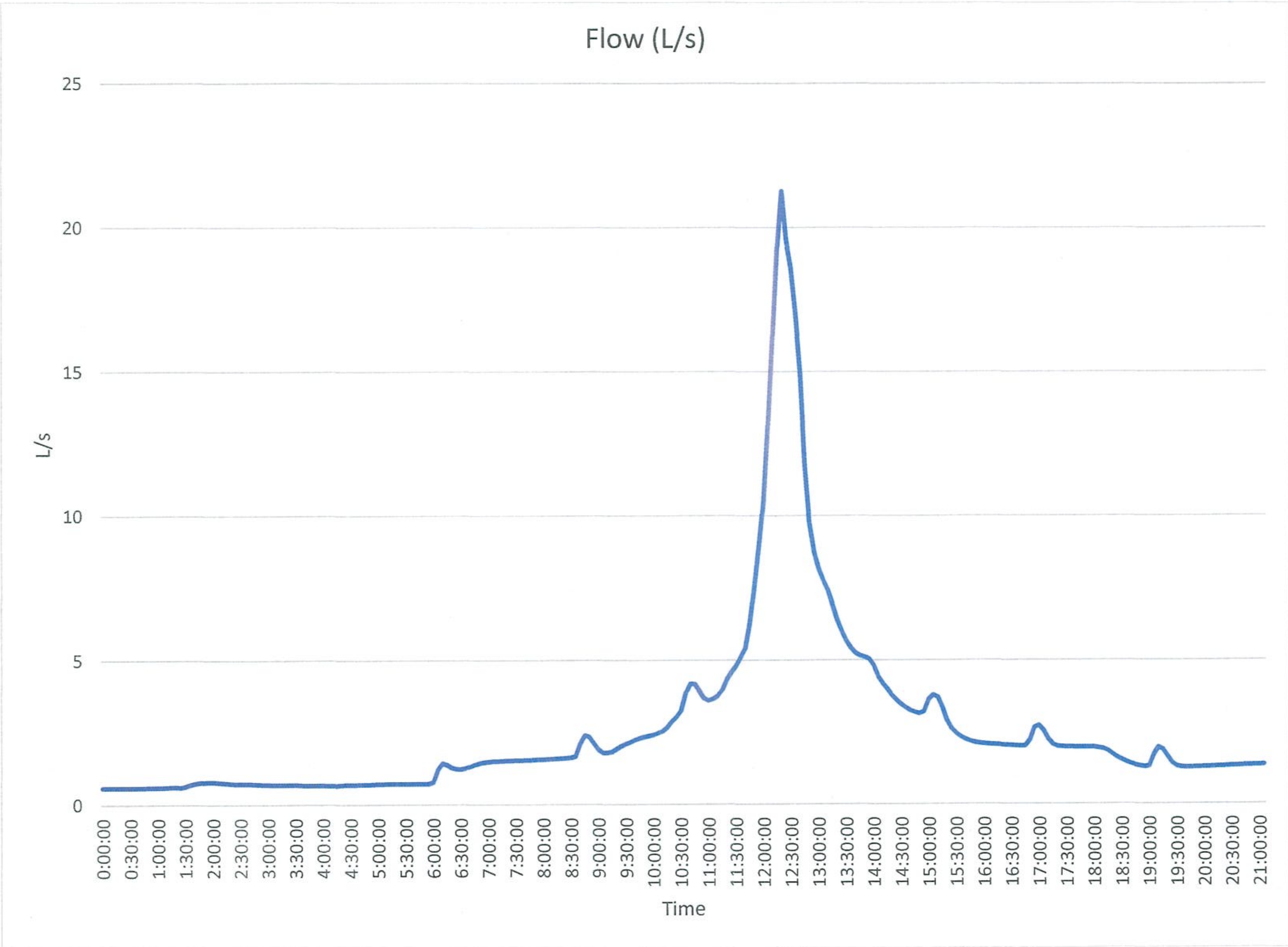


Figure 4: Peak wet weather inflow to Lake Hayes #1 PS using 2028 Growth Model, including Waterfall Park Development

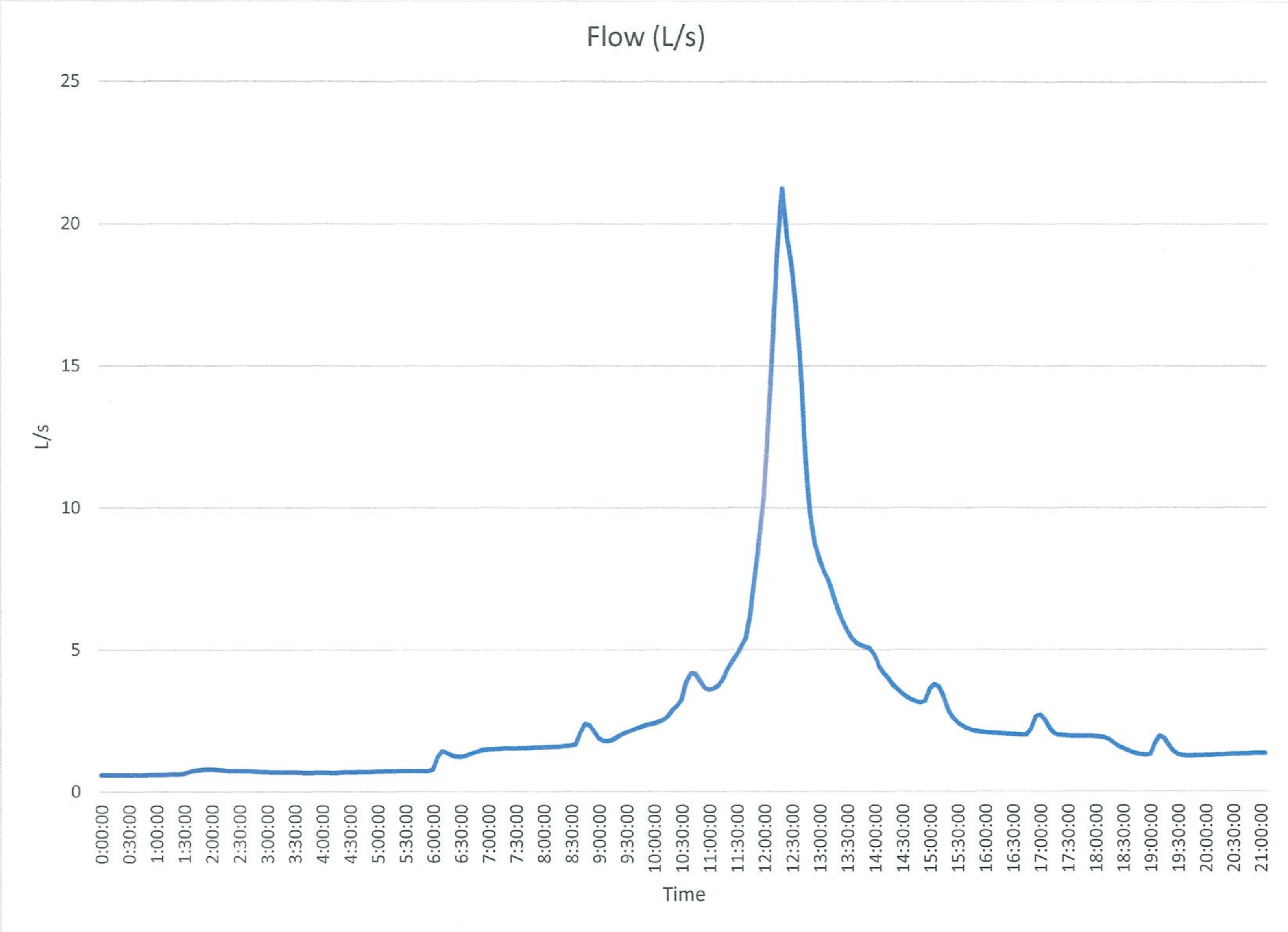


Figure 5: Current peak wet weather inflow to Lake Hayes #2 PS

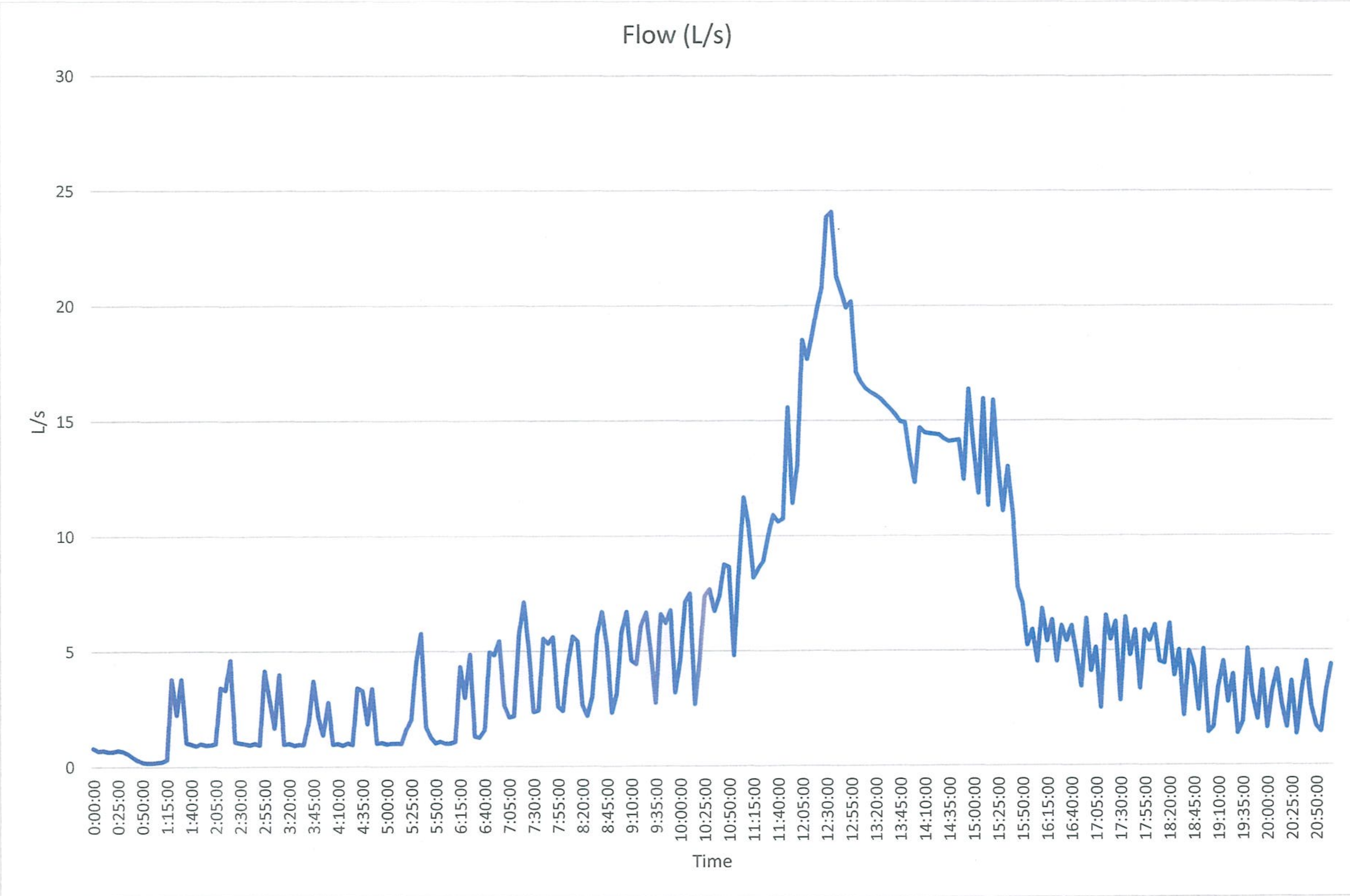


Figure 6: Peak wet weather inflow to Lake Hayes #2 PS using 2028 Growth Model (No Waterfall Park Development)

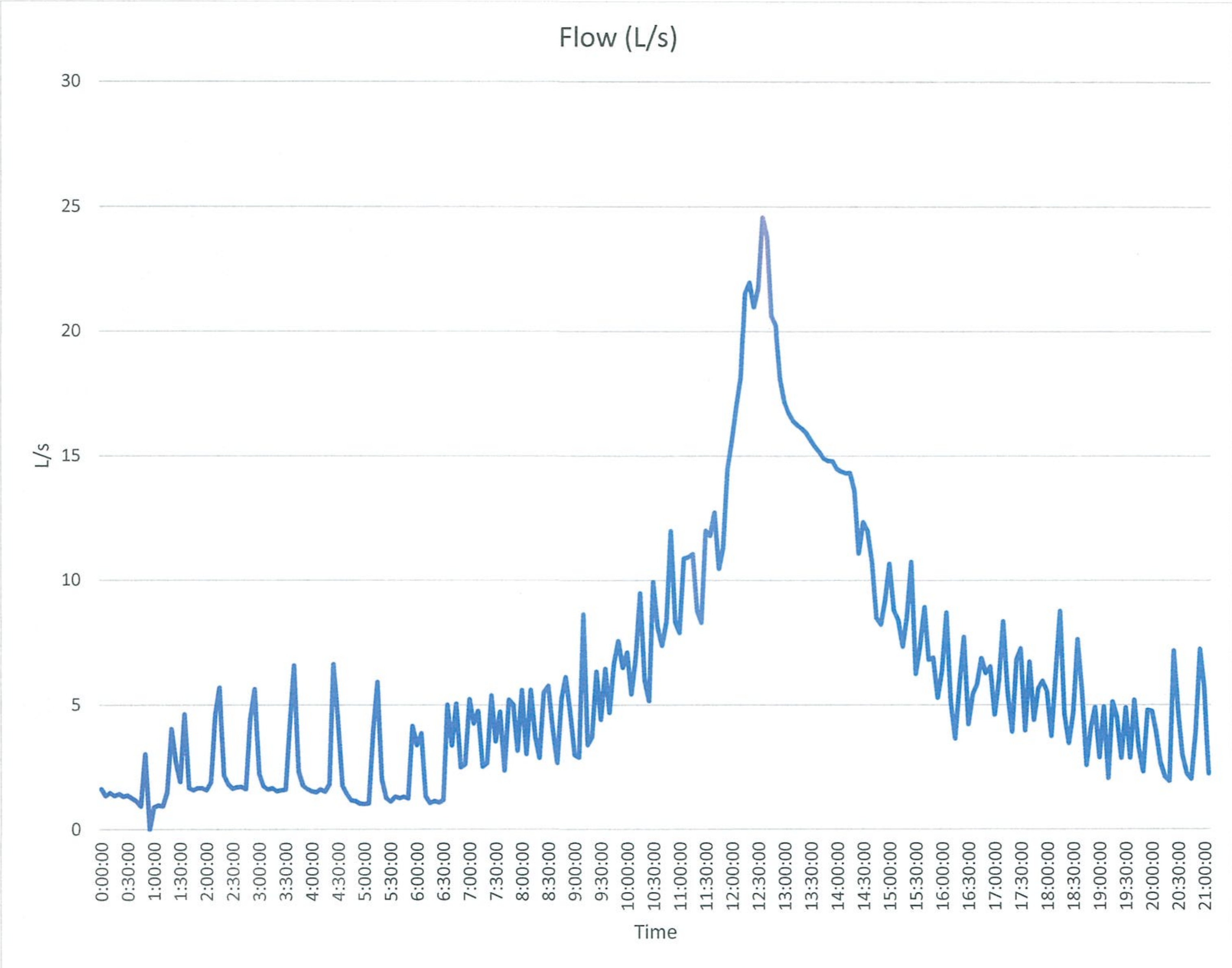


Figure 7: Peak wet weather inflow to Lake Hayes #2 PS using 2028 Growth Model, including Waterfall Park Development

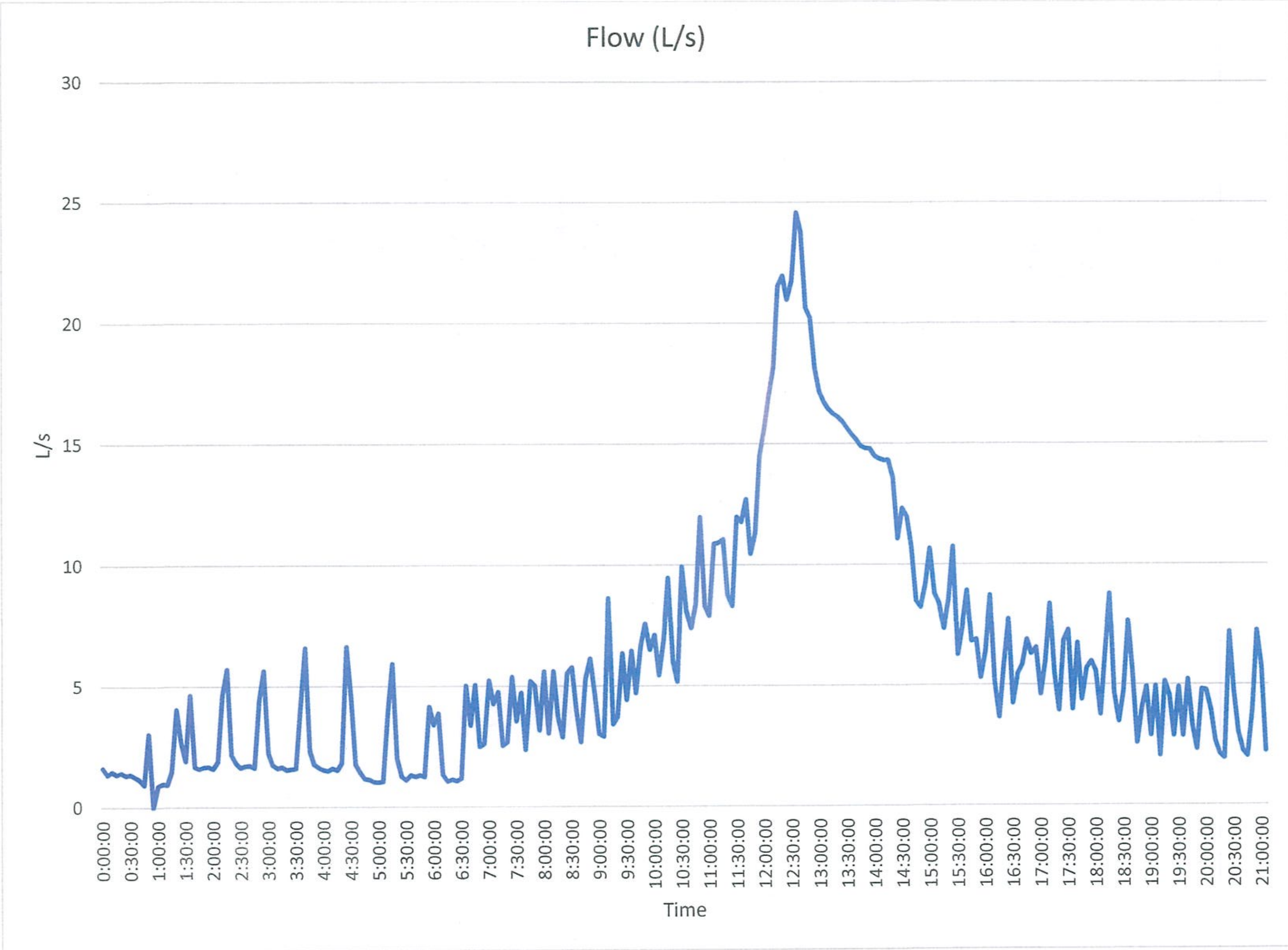


Figure 8: Current peak wet weather inflow to Bendemeer PS

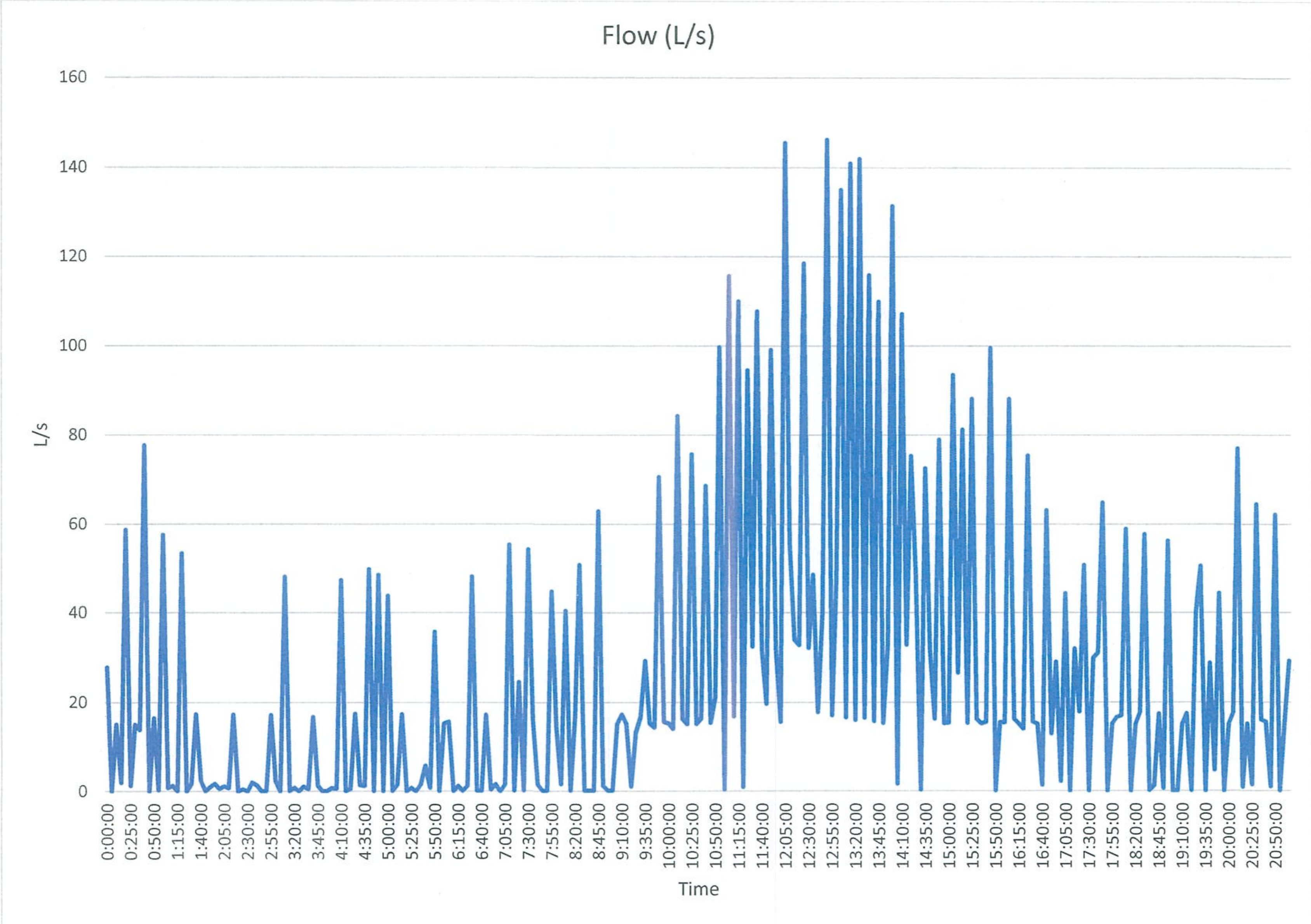


Figure 9: Peak wet weather inflow to Bendemeer PS using 2028 Growth Model (No Waterfall Park Development)

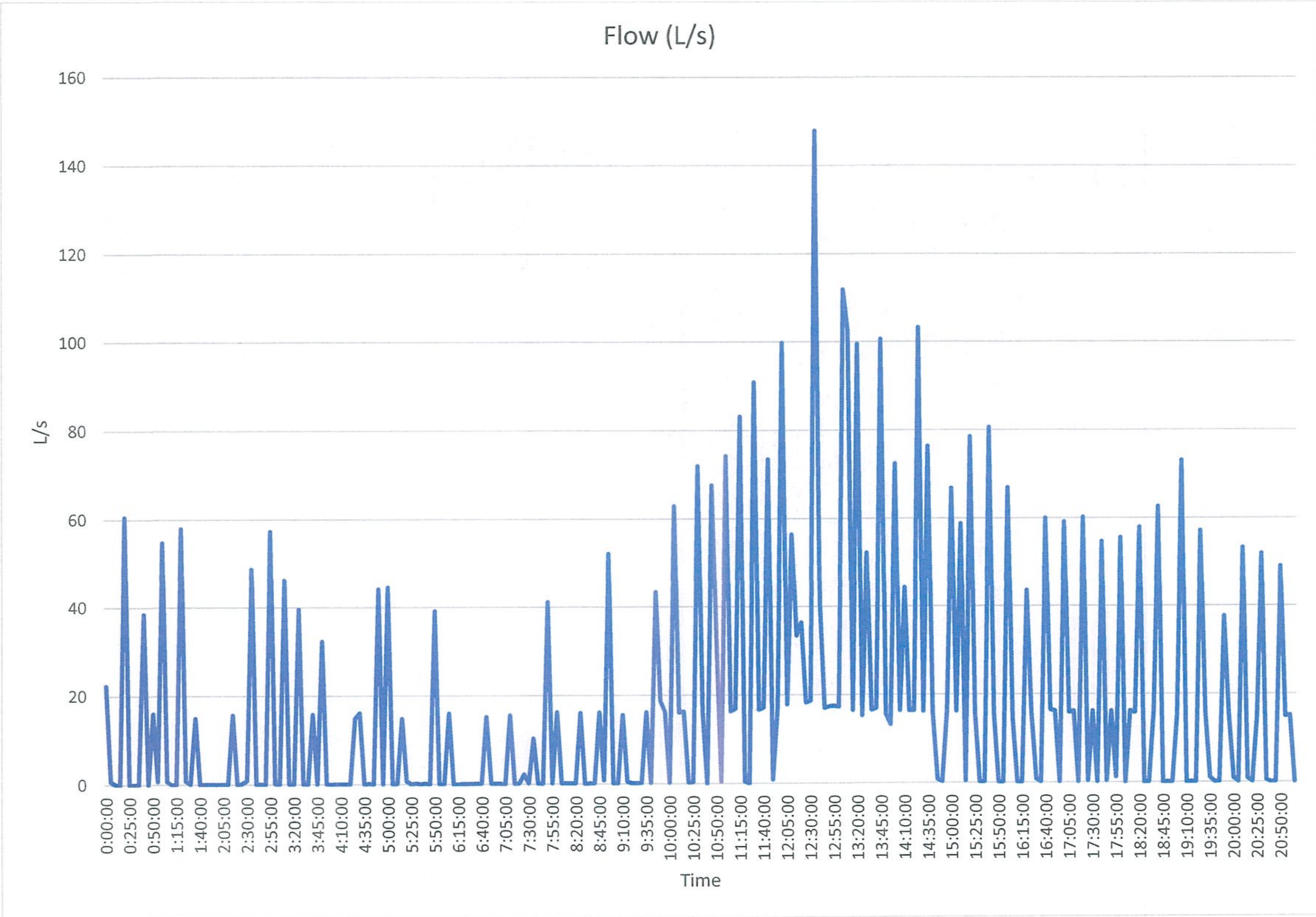
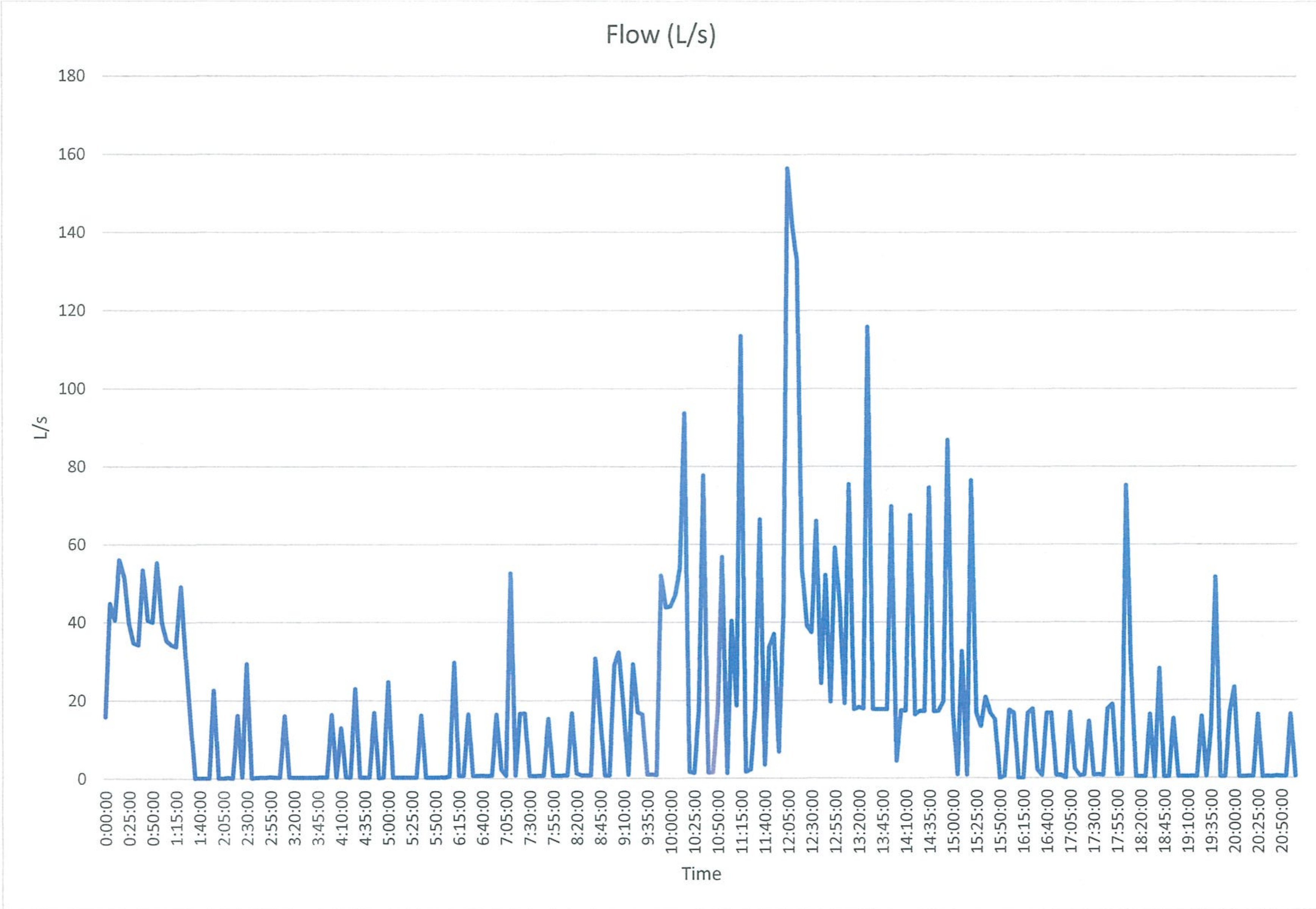


Figure 10: Peak wet weather inflow to Bendemeer PS using 2028 Growth Model, including Waterfall Park Development



Appendix C

Outflows from the Lake Hayes Pump Stations

Figure 11: Current peak wet weather outflow from Lake Hayes #1 PS

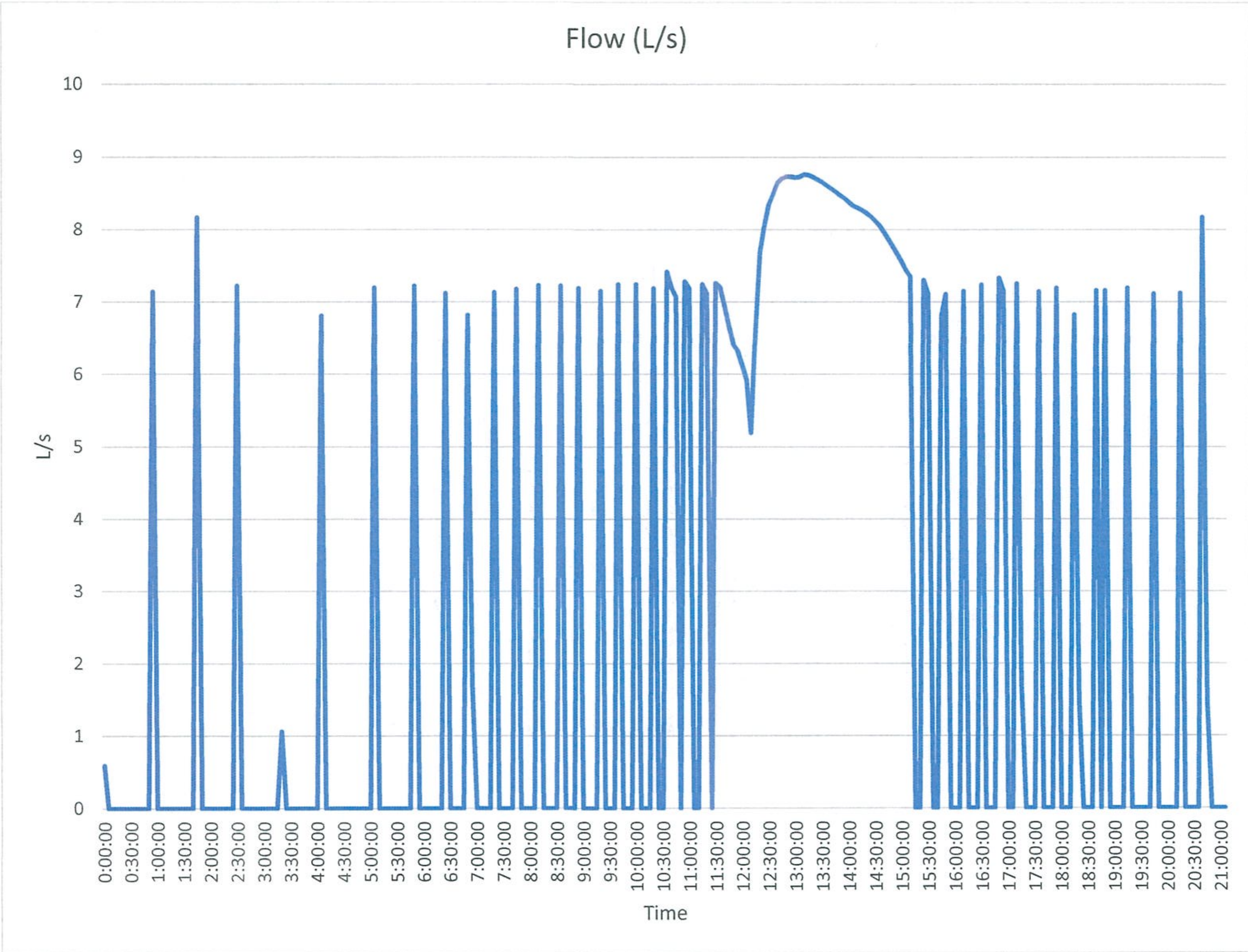


Figure 12: Peak wet weather outflow from Lake Hayes #1 PS using 2028 Growth Model (No Waterfall Park Development)

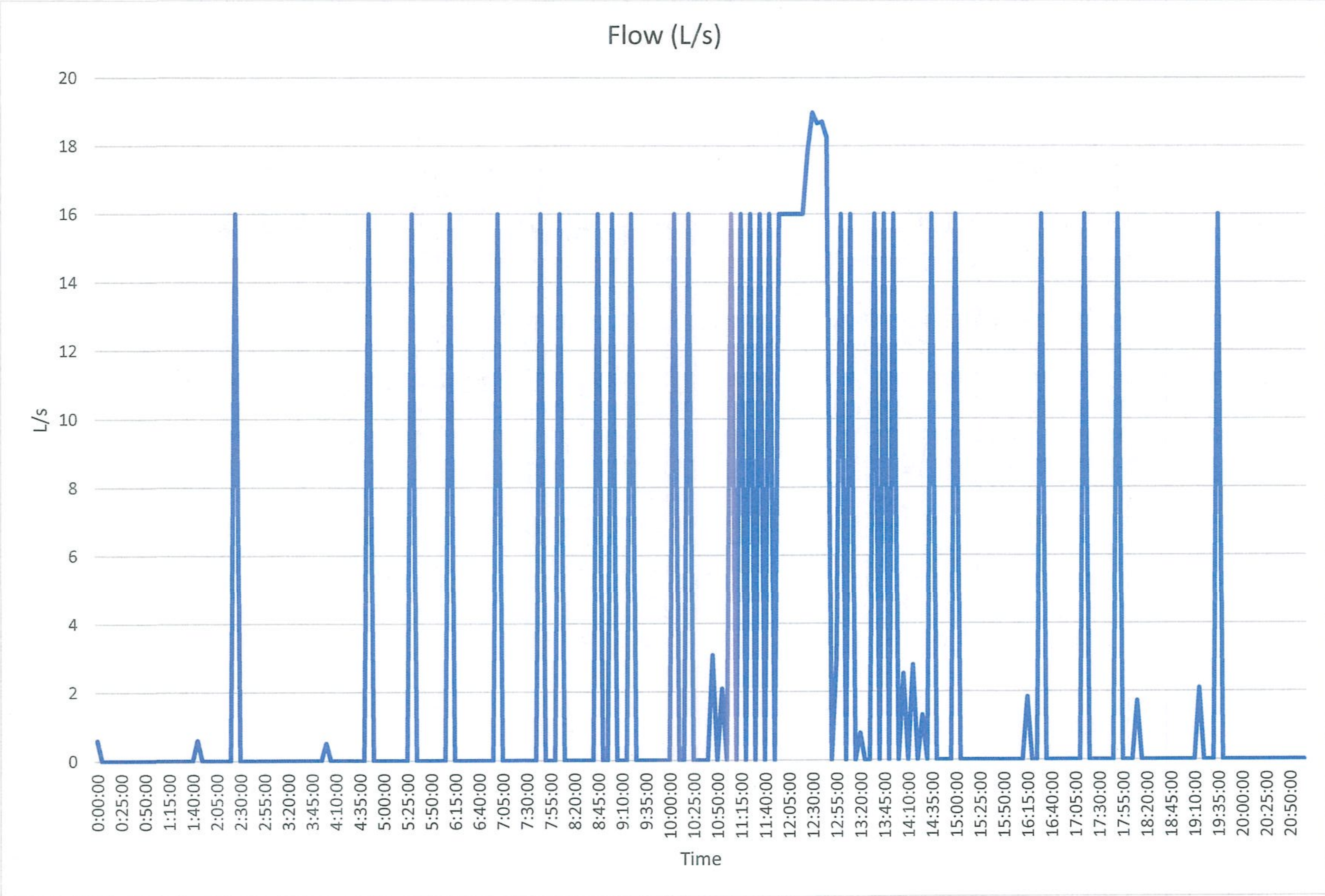


Figure 13: Peak wet weather outflow from Lake Hayes #1 PS using 2028 Growth Model, and Including Waterfall Park Development

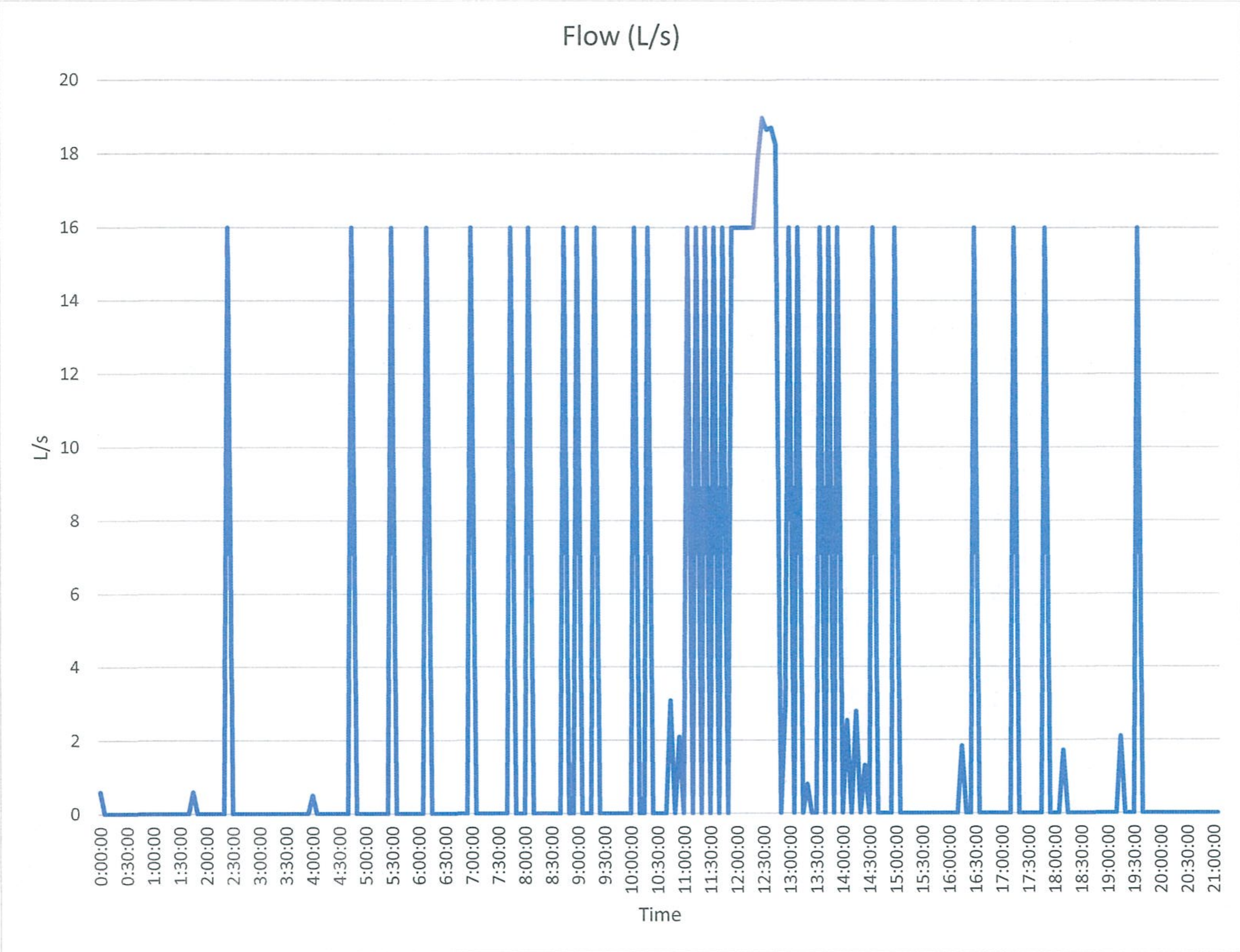


Figure 14: Current peak wet weather outflow from Lake Hayes #2 PS

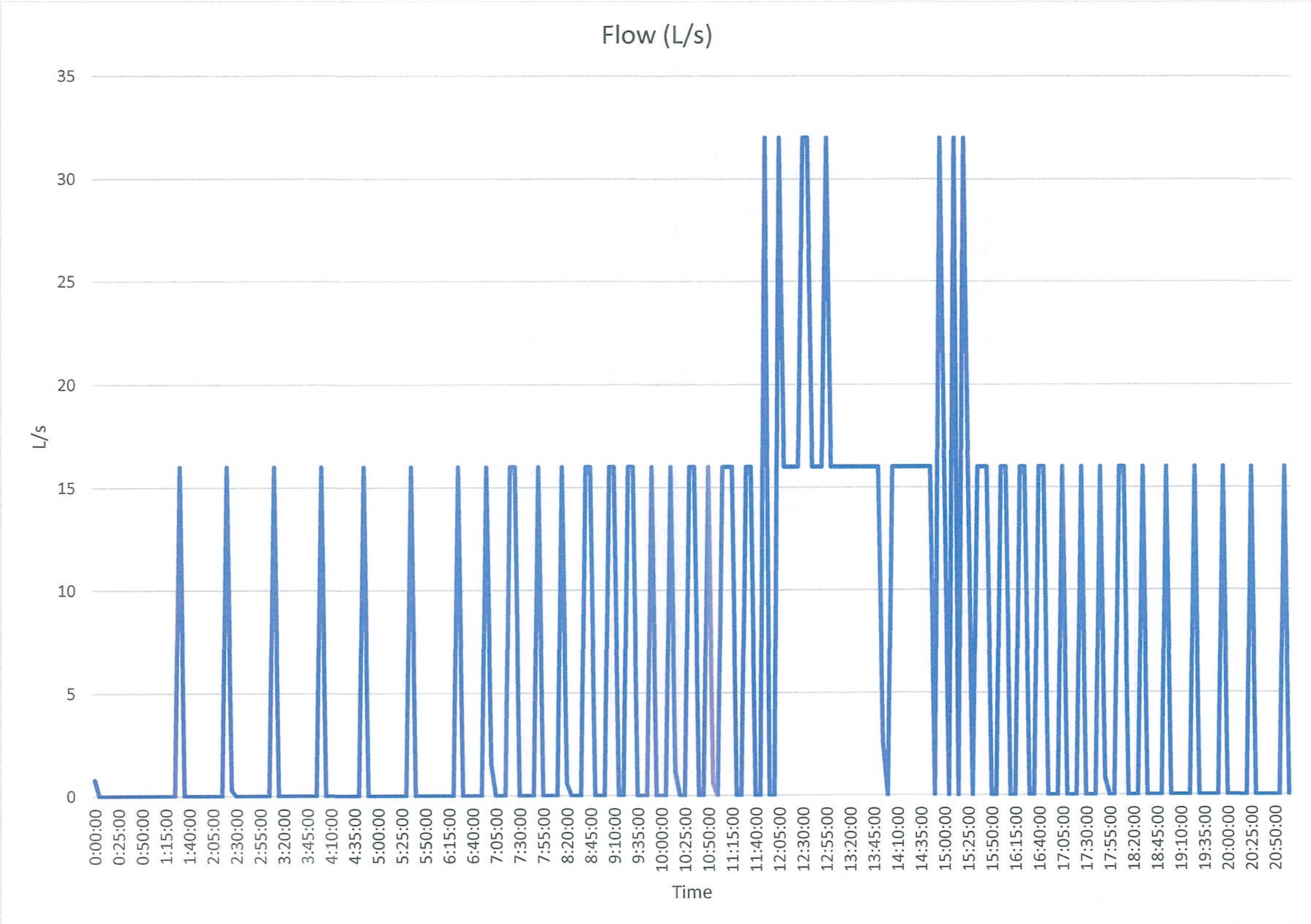


Figure 15: Peak wet weather outflow from Lake Hayes #2 PS using 2028 Growth Model (No Waterfall Park Development)

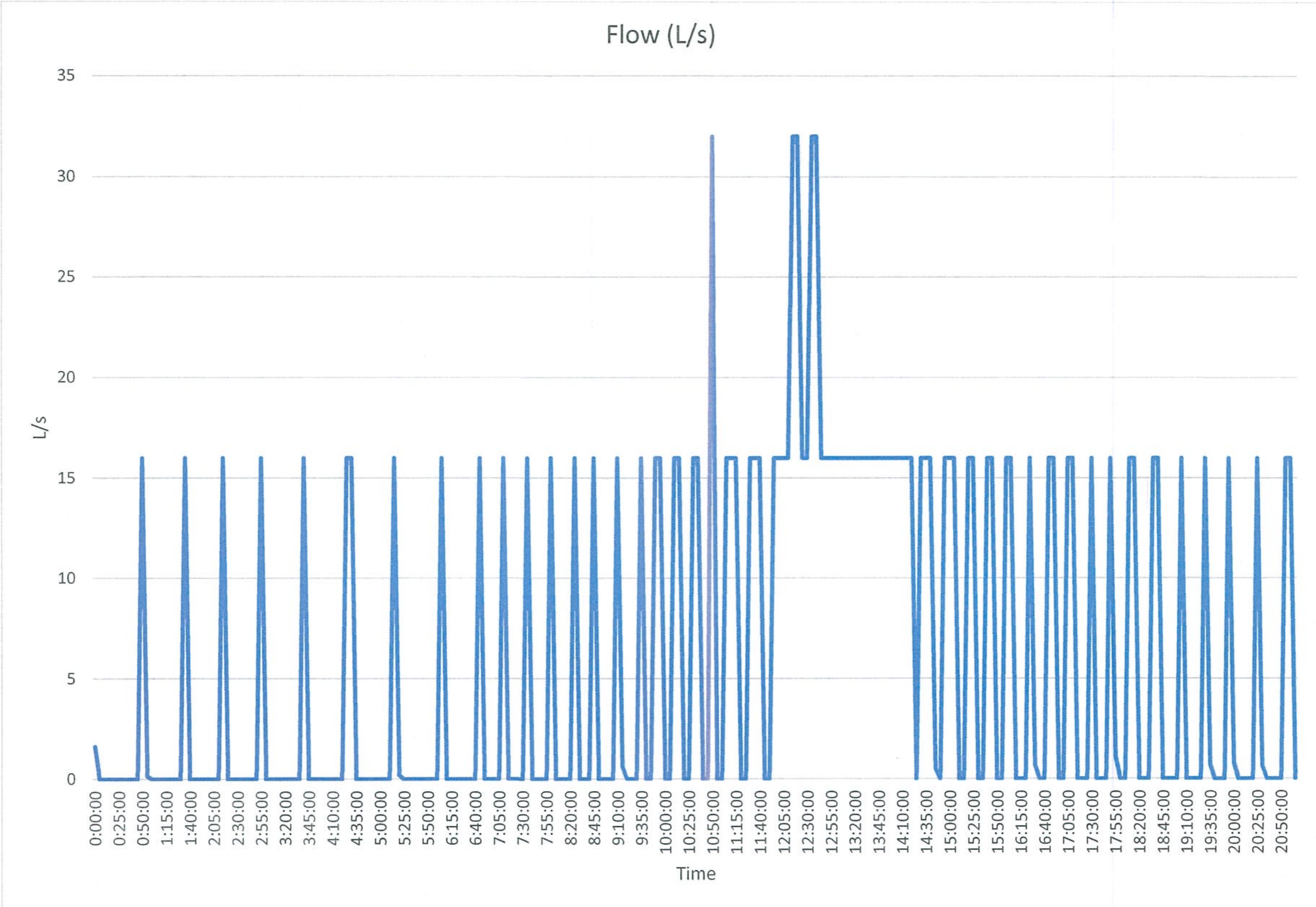


Figure 16: Peak wet weather outflow from Lake Hayes #2 PS using 2028 Growth Model, and including Waterfall Park Development

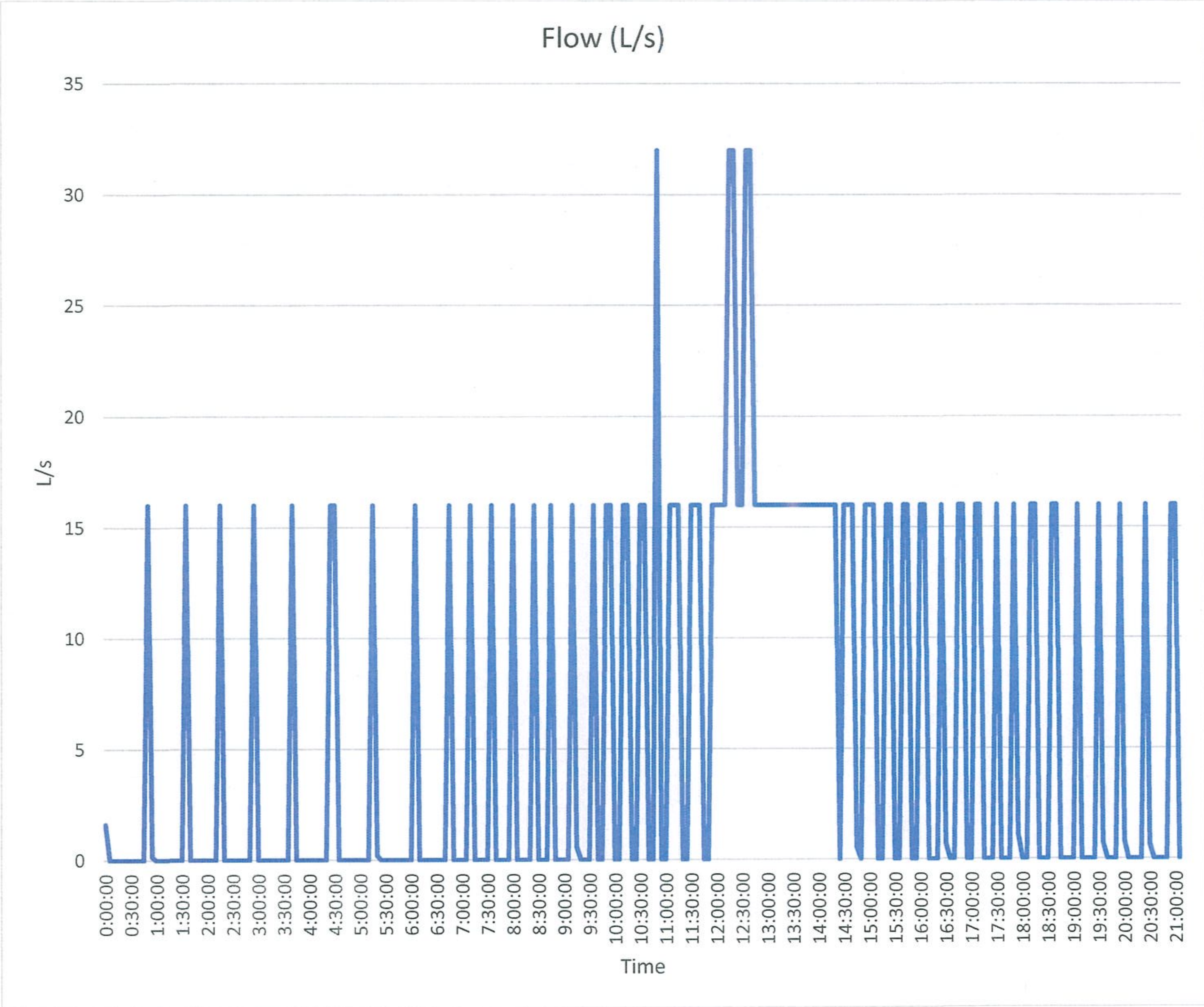


Figure 17: Current peak wet weather outflow from Bendemeer PS

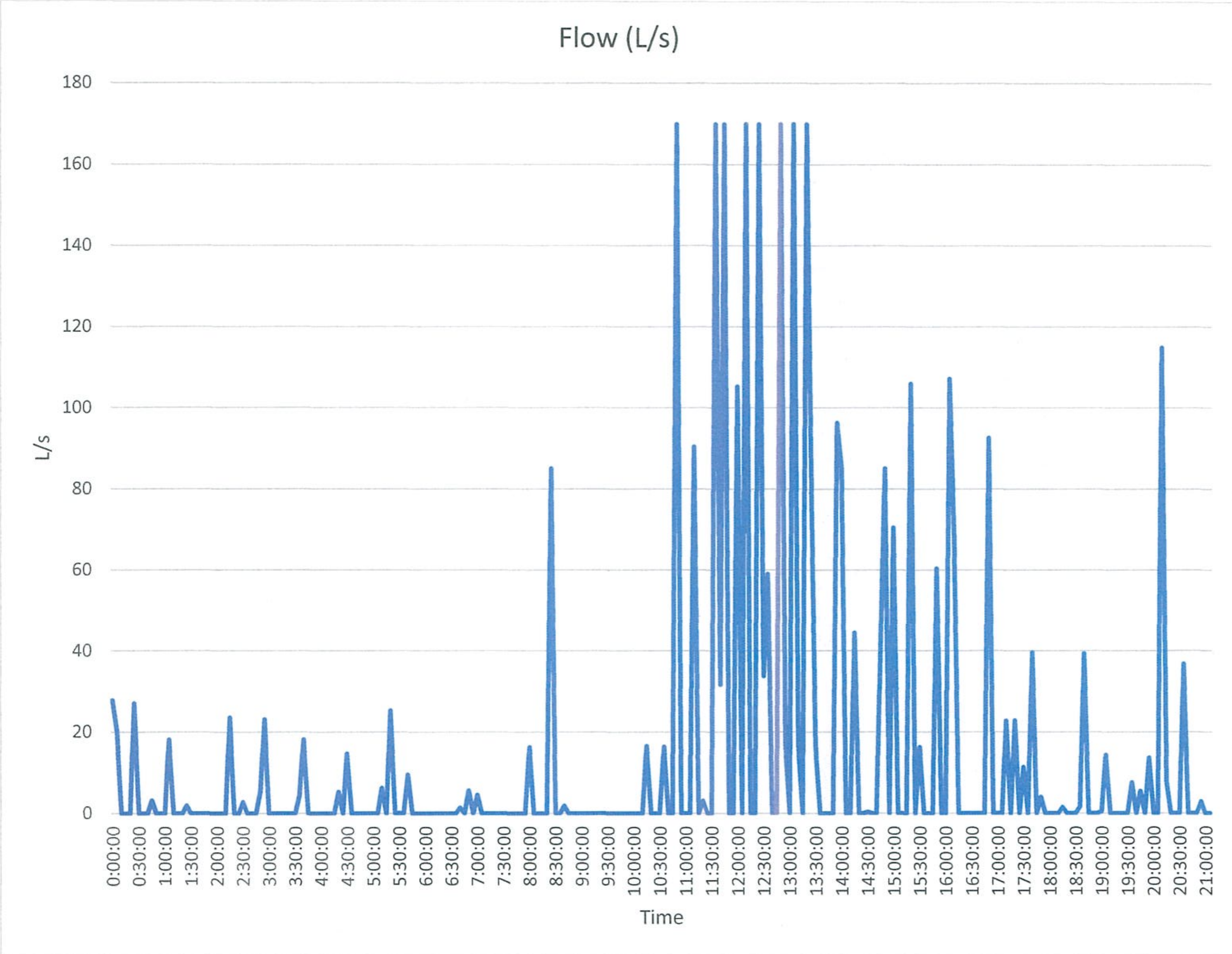


Figure 18: Peak wet weather outflow from Bendemeer PS using 2028 Growth Model (No Waterfall Park Development)

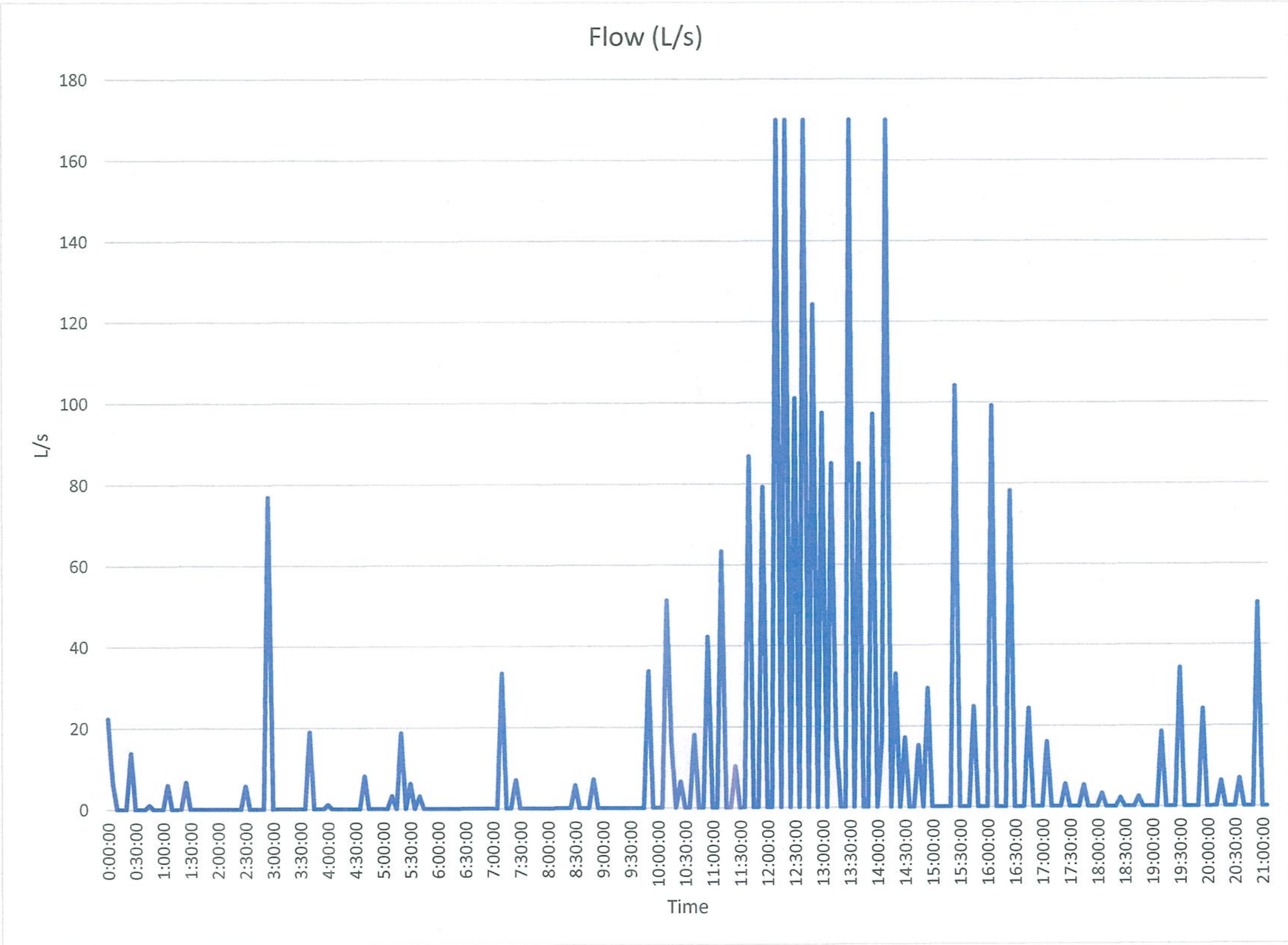
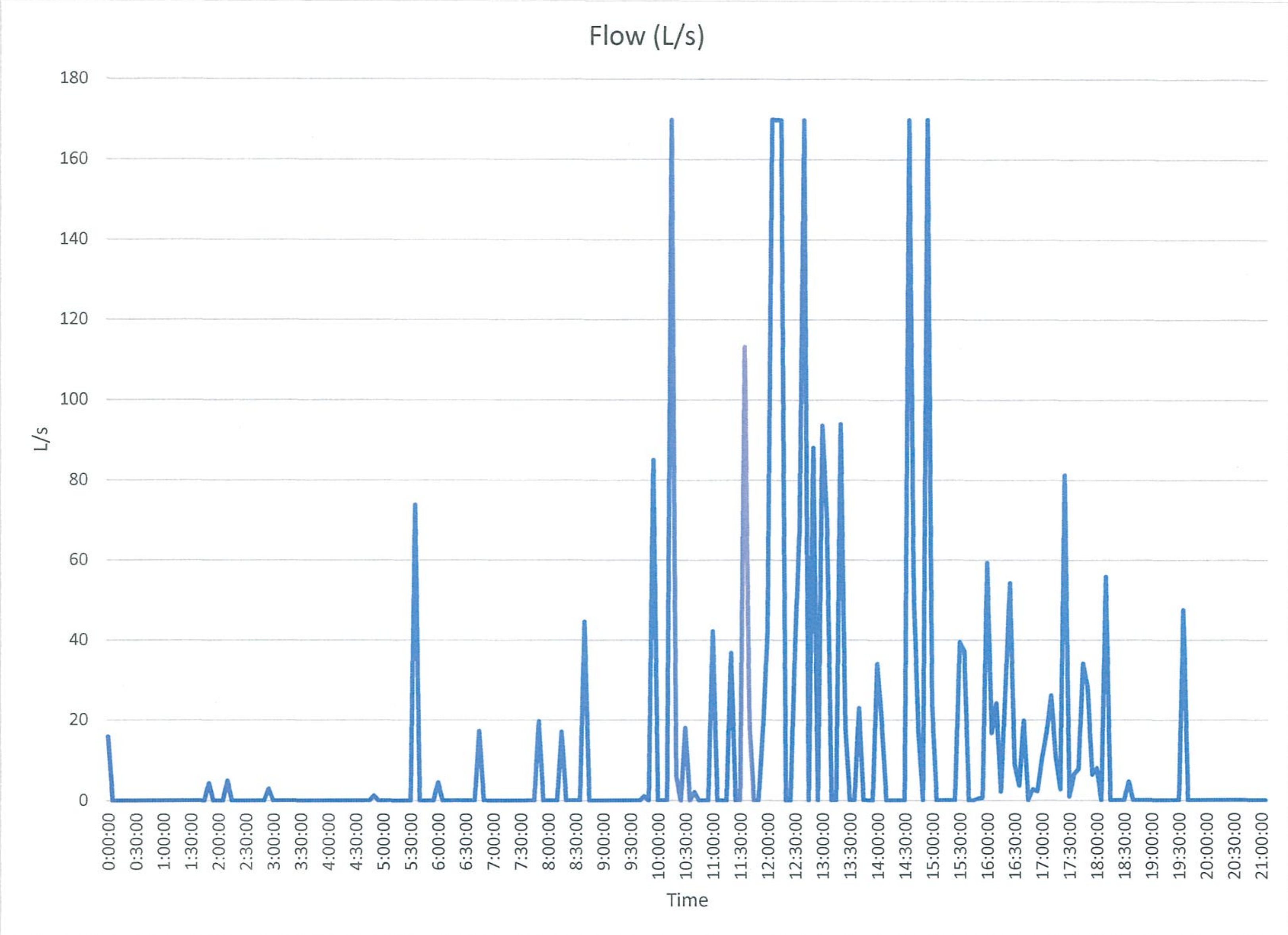


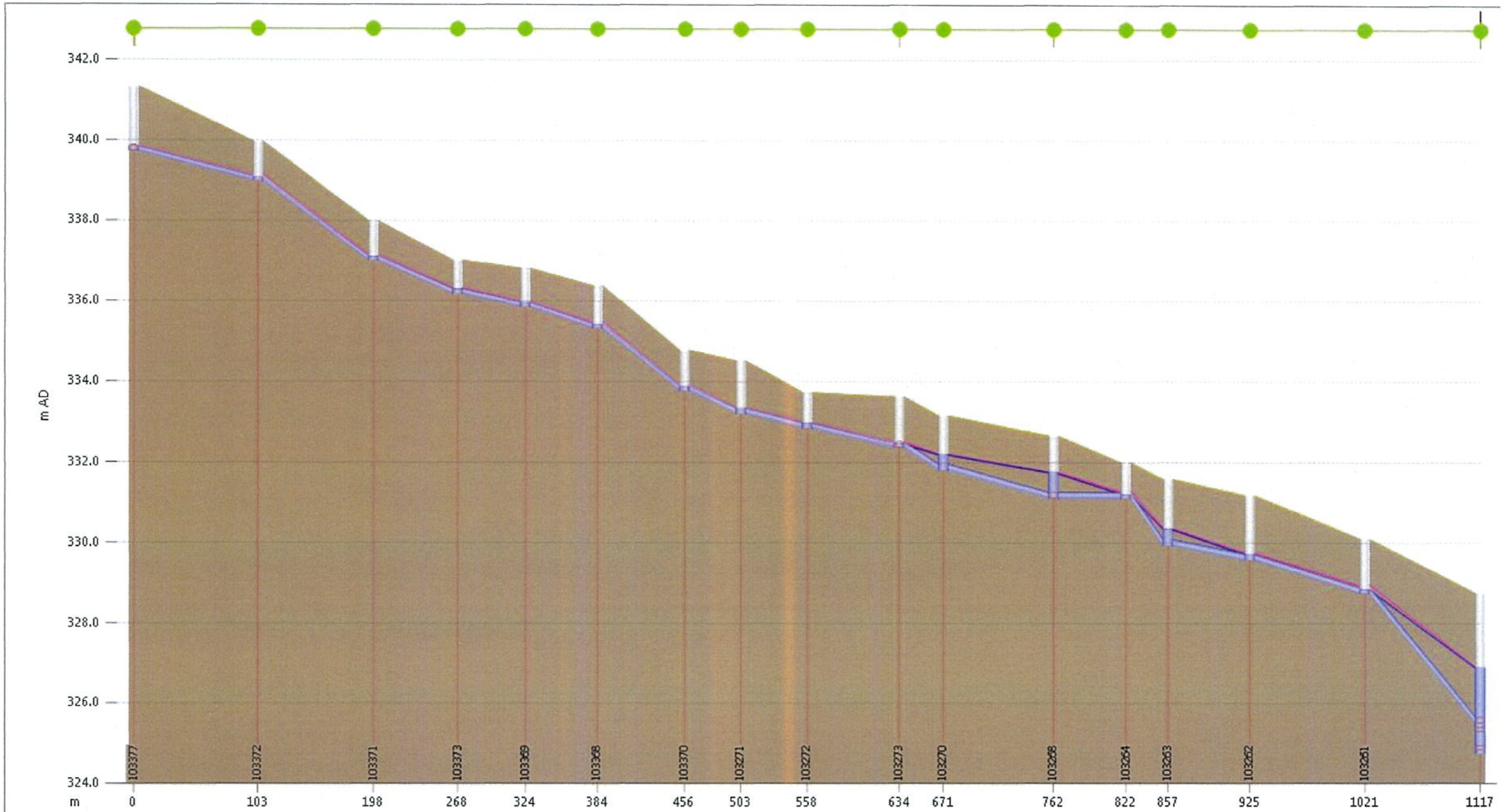
Figure 19: Peak wet weather outflow from Bendemeer PS using 2028 Growth Model, and Including Waterfall Park Development



Appendix D

Long Sections

Figure 20: Long Section Upstream of Lake Hayes #1 PS without Development



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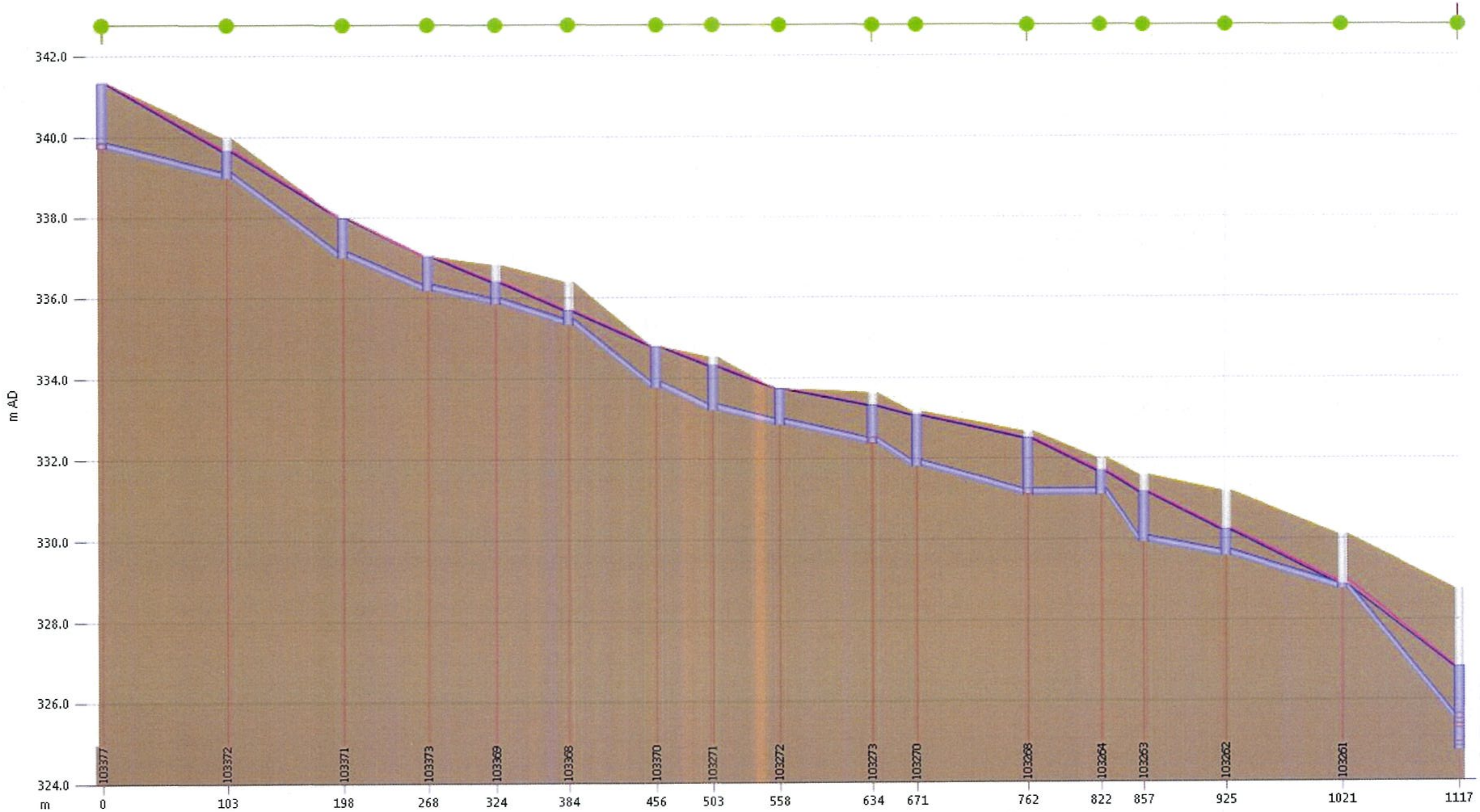
width (mm)	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
us inv (m AD)	339.720	338.990	337.010	336.190	335.860	335.340	333.780	333.220	332.850	-	331.810	331.100	-	329.930	329.580	328.760
ds inv (m AD)	338.990	337.010	336.190	335.860	335.340	333.780	333.220	332.850	332.370	-	331.100	331.100	-	329.580	328.760	325.500

Node	103377	103372	103371	103373	103369	103368	103370	103271	103272	103273	103270	103268	103264	103263	103262	103261	107909
ground (m AD)	341.330	339.970	337.990	337.020	336.820	336.390	334.780	334.520	333.730	333.630	333.160	332.660	-	331.580	331.180	330.100	-
level (m AD)	339.831	339.063	337.103	336.303	335.962	335.414	333.870	333.335	332.966	332.466	332.194	331.749	-	330.344	329.718	328.841	-

Figure 21: Long Section Upstream of Lake Hayes #1 PS with Maximum Development



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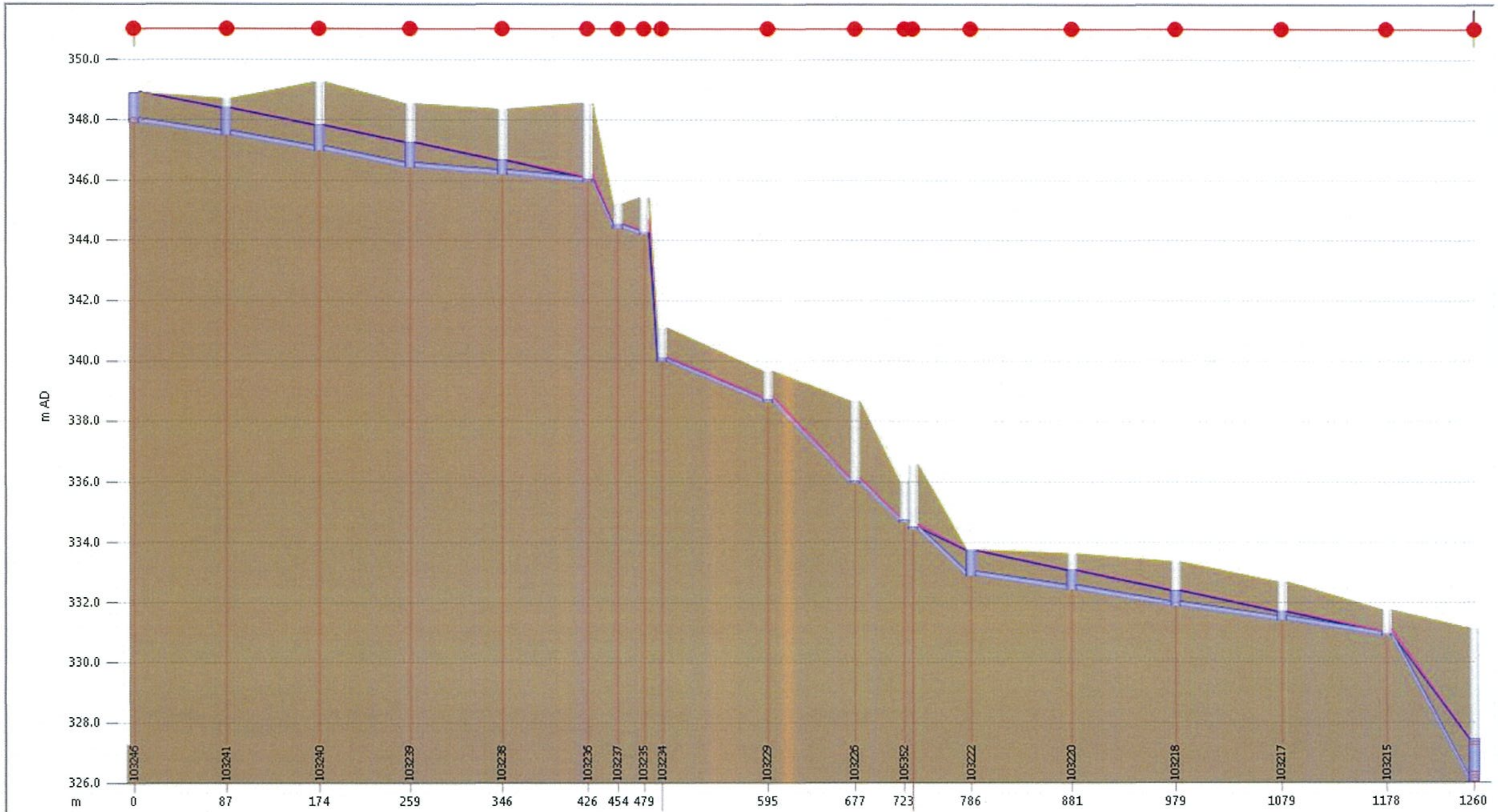
width (mm)	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150
us inv (m AD)	339.720	338.990	337.010	336.190	335.860	335.340	333.780	333.220	332.850	-	331.810	331.100	-	329.930	329.580	328.760
ds inv (m AD)	338.990	337.010	336.190	335.860	335.340	333.780	333.220	332.850	332.370	-	331.100	331.100	-	329.580	328.760	325.500

Node	103377	103372	103371	103373	103369	103368	103370	103271	103272	103273	103270	103268	103264	103263	103262	103261	107909
ground (m AD)	341.330	339.970	337.990	337.020	336.820	336.390	334.780	334.520	333.730	333.630	333.160	332.660	-	331.580	331.180	330.100	-
level (m AD)	341.330	339.652	337.990	337.020	336.400	335.689	334.780	334.314	333.730	333.326	333.076	332.500	-	331.157	330.226	328.854	-

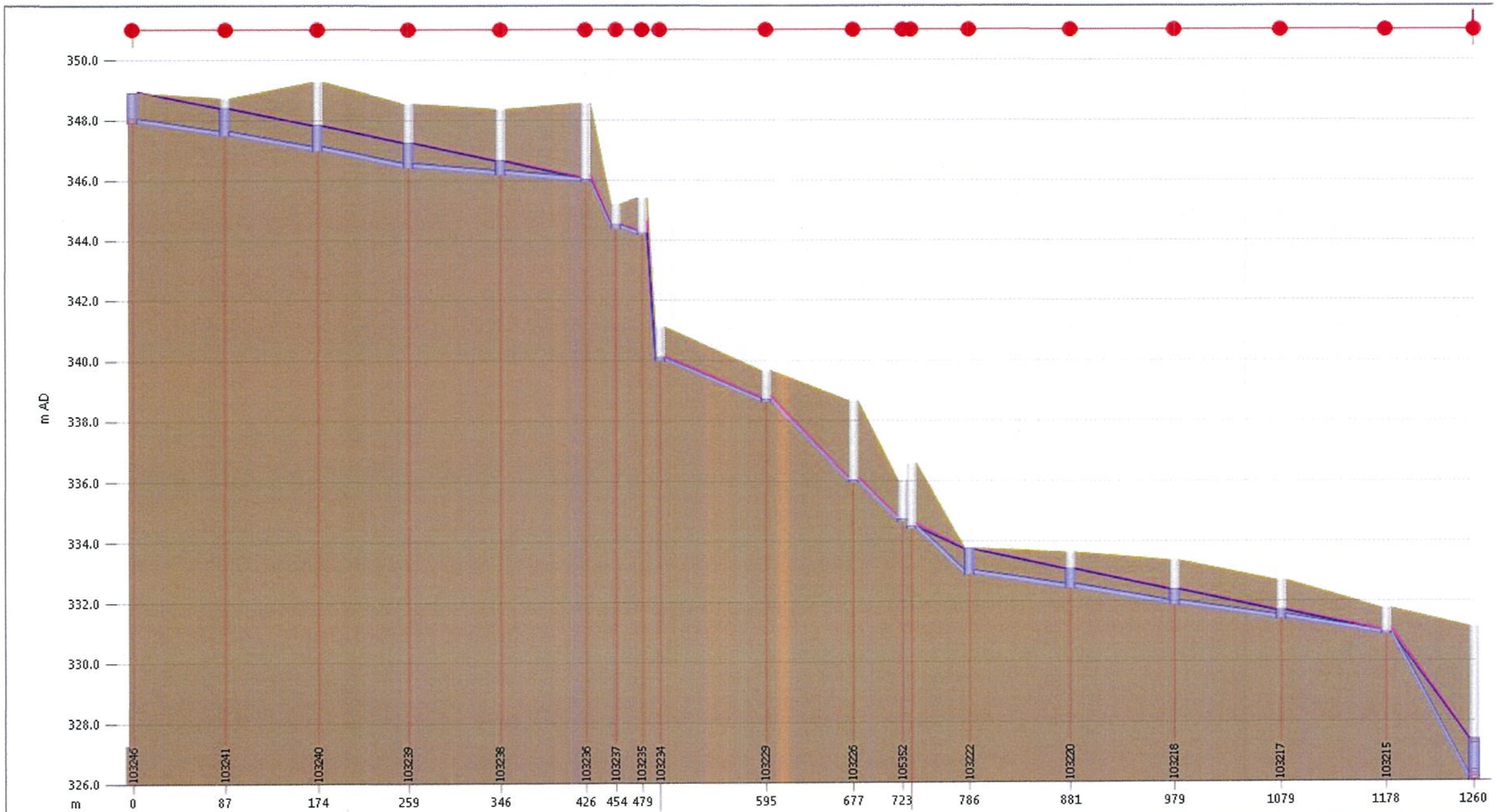
Figure 22: Long Section Upstream of Lake Hayes #2 PS without Development



Link	103246.1	103241.1	103240.1	103239.1	103238.1	-	-	-	103234.1	103229.1	-	105353.1	103222.1	103220.1	103218.1	103217.1	103215.1
width (mm)	150	150	150	150	150	150	150	-	150	150	150	150	150	150	150	150	150
us inv (m AD)	347.910	347.500	346.990	346.440	346.200	-	-	-	340.000	338.650	-	334.420	332.890	332.440	331.880	331.400	330.890
ds inv (m AD)	347.500	346.990	346.440	346.200	345.976	-	-	-	338.650	335.950	-	332.890	332.440	331.880	331.400	330.890	326.030

Node	-	103241	103240	103239	103238	103236	-	-	103234	103229	103226	-	-	103222	103220	103218	103217	103215	-
ground (m AD)	-	348.710	349.260	348.540	348.360	-	-	-	341.110	339.670	338.670	-	-	333.750	333.620	333.350	332.680	331.750	-
level (m AD)	-	348.398	347.831	347.254	346.657	-	-	-	340.110	338.725	336.031	-	-	333.733	333.076	332.393	331.690	330.953	-

Figure 23: Long Section Upstream of Lake Hayes #2 PS with Maximum Development



Link	103246.1	103241.1	103240.1	103239.1	103238.1	-	-	-	103234.1	103229.1	-	105353.1	103222.1	103220.1	103218.1	103217.1	103215.1
width (mm)	150	150	150	150	150	150	150	-	150	150	150	150	150	150	150	150	150
us inv (m AD)	347.910	347.500	346.990	346.440	346.200	-	-	-	340.000	338.650	-	334.420	332.890	332.440	331.880	331.400	330.890
ds inv (m AD)	347.500	346.990	346.440	346.200	345.976	-	-	-	338.650	335.950	-	332.890	332.440	331.880	331.400	330.890	326.030

Node	-	103241	103240	103239	103238	103236	-	-	103234	103229	103226	-	-	103222	103220	103218	103217	103215	-
ground (m AD)	-	348.710	349.260	348.540	348.360	-	-	-	341.110	339.670	338.670	-	-	333.750	333.620	333.350	332.680	331.750	-
level (m AD)	-	348.399	347.831	347.254	346.657	-	-	-	340.110	338.725	336.031	-	-	333.735	333.078	332.394	331.691	330.953	-

APPENDIX B

Water Modelling Report

MOTT MACDONALD

Queenstown Lakes District Council
Private Bag 50072
Queenstown 9348,
New Zealand

Waterfall Park Development – Water Impact Assessment

19 March 2018

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This letter summarises the results of the assessment undertaken for a proposed development consisting of mixed land use, including a hotel (380 rooms) and a residential development of 125 units (double dwelling). The project is located on the northwest side of Arrowtown-Lake Hayes Rd and Speargrass Flat Rd.

1 Background

In January 2018 Mott MacDonald was commissioned by Queenstown Lakes District Council (QLDC) to assess the system performance in terms of Level of Service (LOS) and firefighting capacity in the proposed development.

In this analysis, the latest Lake Hayes water supply model was used. Three scenarios were investigated, with and without additional demand from the proposed development for existing and future conditions. These are further detailed in the scenarios investigation section of this letter.

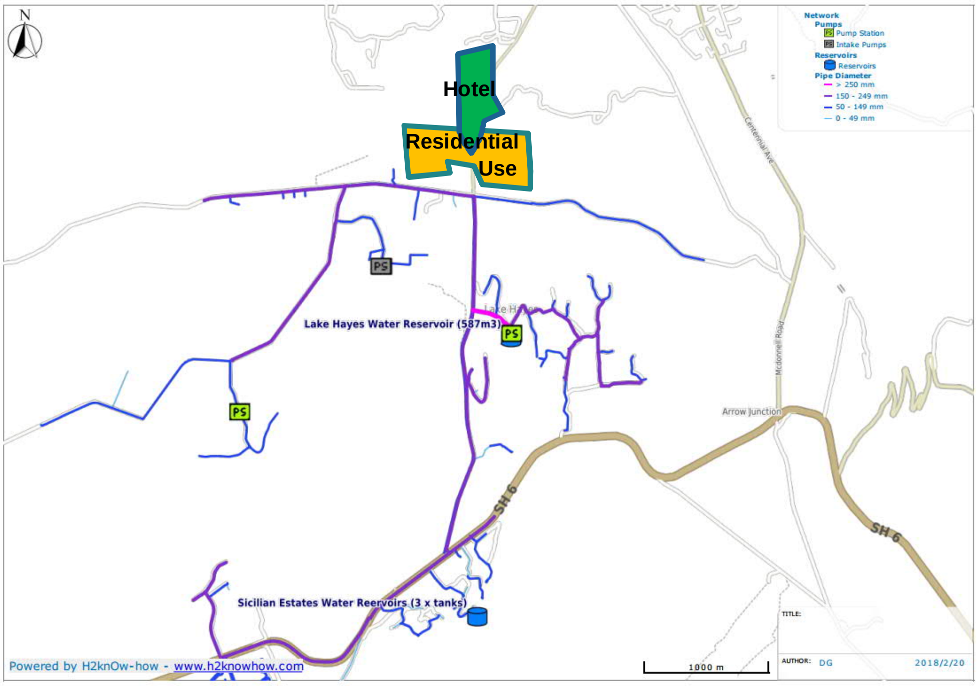

Figure 1 - Proposed Development Location

Mott MacDonald New Zealand Limited Registered in New Zealand no. 3338812

MOTT MACDONALD

2 Assumptions

2.1 Demand Calculations

A demand assessment was provided by the client as summarised in Table 1 below. The detailed calculation is attached in appendix.

Table 1 - Demand Calculation

Hotel Facility (Elevation: RL 368m)	
No. Hotel rooms	380
Maximum people per room	2
Peak daily consumption (l/day/room)	440
Peak water demand (m^3/day) - room	167.2
Additional demand (conference centre, restaurant, irrigation, etc) (m^3/day)	205.2
Instantaneous Peak Flow (l/s)	**18.9**

Residential Development (Elevation: RL 367m)	
No. Primary Dwelling (3 people)	125
No. Secondary Dwelling (2 people)	125
Peak consumption Primary Dwelling (l/day/property)	2,100
Peak consumption Secondary Dwelling (l/day/property)	700
Peak water demand (m^3/day)	350
Instantaneous Peak Flow (l/s)	**26.7**

The calculated demand seems conservative when compared to the observed consumption in Queenstown (2000l/property/day) and Lake Hayes (see table below).

Table 2 - Lake Hayes Demands

DMA Zone	Total demand (m^3/day)	Number of connections	Average demand per connection (l/prop/day)
Shotover Country	374	495	756
Lake Hayes Estate	822	596	1379
Lake Hayes	928	421	2204
Bendeemer	17	13	1308
Terraces	25	9	2778
DMAs Combined	**2,166**	**1,534**	**1,412**

As shown in the table above, the proposed development peak day demand is equivalent to a third of the current peak day demand in the entire service area.

2.2 Proposed Connection Point

The minimum and maximum elevations within the proposed development areas of the lots are shown in the table below:

Table 3 - Proposed Development Elevations

	Min elevation in proposed development area	Max elevation in proposed development area
Hotel Development	347.5m (with 4 story hotel building ~12.8m height)	368m (with single story building only)
Residential Development	342m	367m

Overall, the maximum elevation within the lot proposed for the residential development is 423m.

As suggested by the developer, it was assumed that the proposed development would be connected to the 235 mm ID main at the Arrowtown-Lake Hayes Rd and Speargrass Flat Rd junction. Figure 2 below shows the development location, and the proposed network and connection point considered in this study.

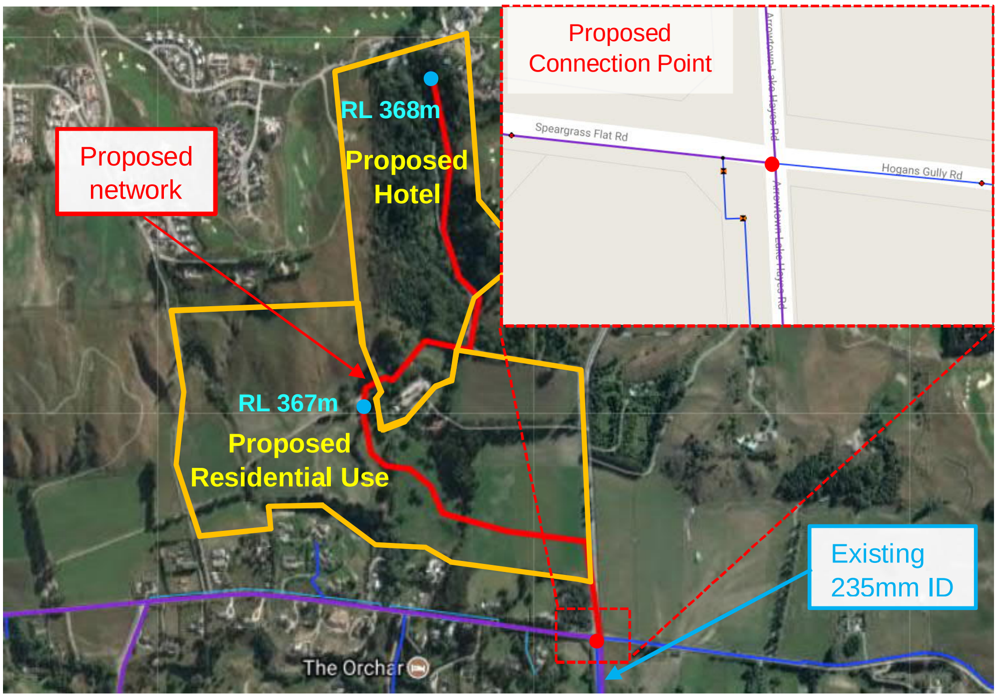

Figure 2 – Proposed Development Location, Network and Connection Point

3 Scenario Investigated

Three scenarios were investigated, including the above demand and the current network operations:

- Existing peak day scenario.
- 2028 peak day scenario.
- 2058 peak day scenario.

Planned upgrades along Frankton Ladies Mile Highway were included in the future 2028 and 2058 scenarios.

To ensure head losses in the proposed network remain between 1 and 3 m/km (recommended head losses for pipeline design), it was assumed that the proposed development would be serviced through a 260mm (ID) pipe connected to the supply point. The proposed network layout was provided by the client and is attached in appendix.

Two elevation points were included, one for the hotel (max. elevation:368m) and one for the residential development (max. elevation:367m). Respective demands were assigned to each point.

Fire flow capacity was assessed based on FW2 requirement plus sprinklers flow of 16.6l/s, as defined by the client.

4 Model Results

4.1 System Performance Analysis in the Proposed Development

This section describes the results of the system performance analysis undertaken for the above scenarios after including the proposed development demands. Results have been analysed to verify whether levels of service can be met in the proposed development without any network modification. The table below summarises the results in terms of minimum and maximum pressure, maximum head losses in the proposed network (260mm pipe) and fire flow capacity.

Table 4 - Minimum Pressure and Maximum Head Losses in Proposed Development

Scenario	Minimum Pressure (m)	Maximum Pressure (m)	Maximum Head Losses (m/km)	Fire Flow
Existing	60.9	97.1	3.0	Can meet residential fire flow (FW2 –25 l/s + 16.6l/s sprinklers flow)
2028	59.9	97.1		
2058	58.0	97.0		

The normal operating pressure set by QLDC addendum to NZS4404:2004 (Development ad Subdivision Engineering Standards) is 30 to 90m. As shown in the table above, minimum pressure in the proposed development is predicted to meet the recommended LOS for all scenarios. However, pressures higher than the recommended LOS are predicted in areas below 349m.

FW2 fire flow was tested at the end of the proposed 260mm (ID) line. The model predicts that residential fireflow (FW2 – 25l/s) plus the sprinkler flow required can be provided with a residual pressure of 47m at RL 368m.

The highest elevation that would be serviceable for the residential development is 395m. Recommended LOS in terms of pressure and fire flow are predicted to be met up to this point.

4.2 System Performance Analysis in the Remaining of the Network

The section below describes the results of the system performance in the remaining of the Lake Hayes network. Results have been analysed to assess the effect of the proposed development for each scenario.

Figure 3 to Figure 8 below show the system performance for current operational conditions, including current, 2028 and 2058 peak demand.

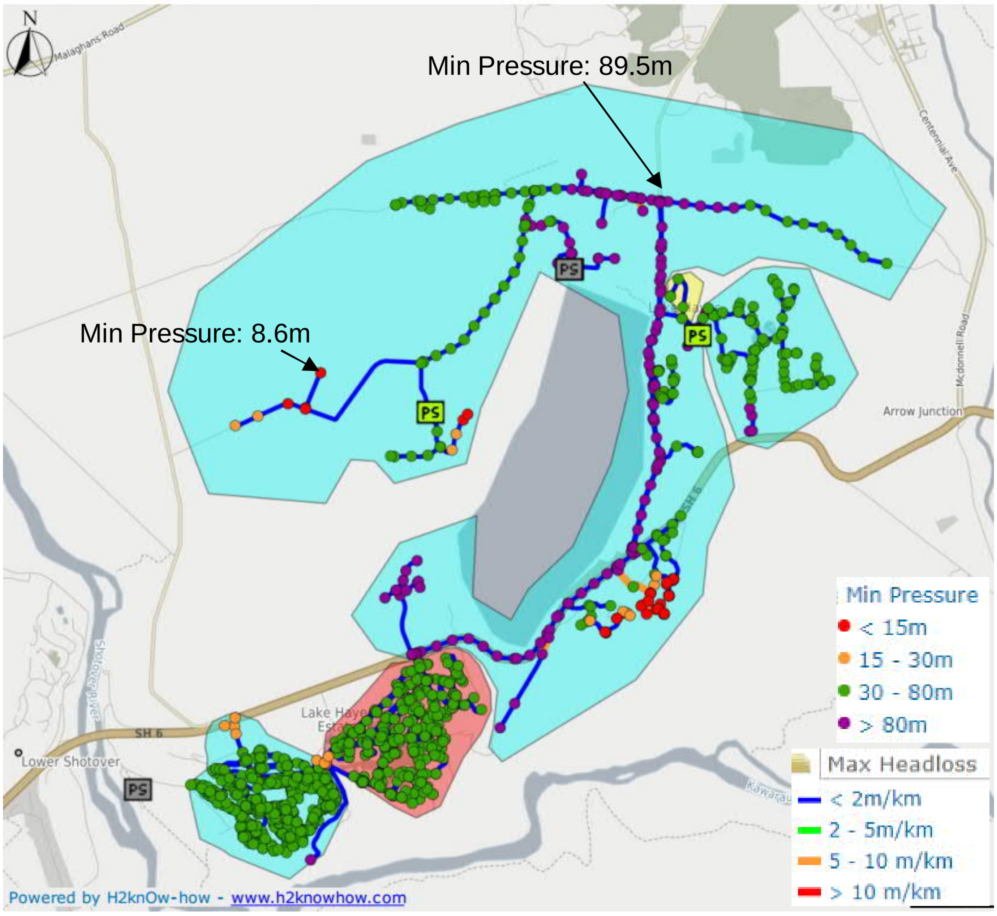

Figure 3 – Current Peak Day System Performance – Prior Development

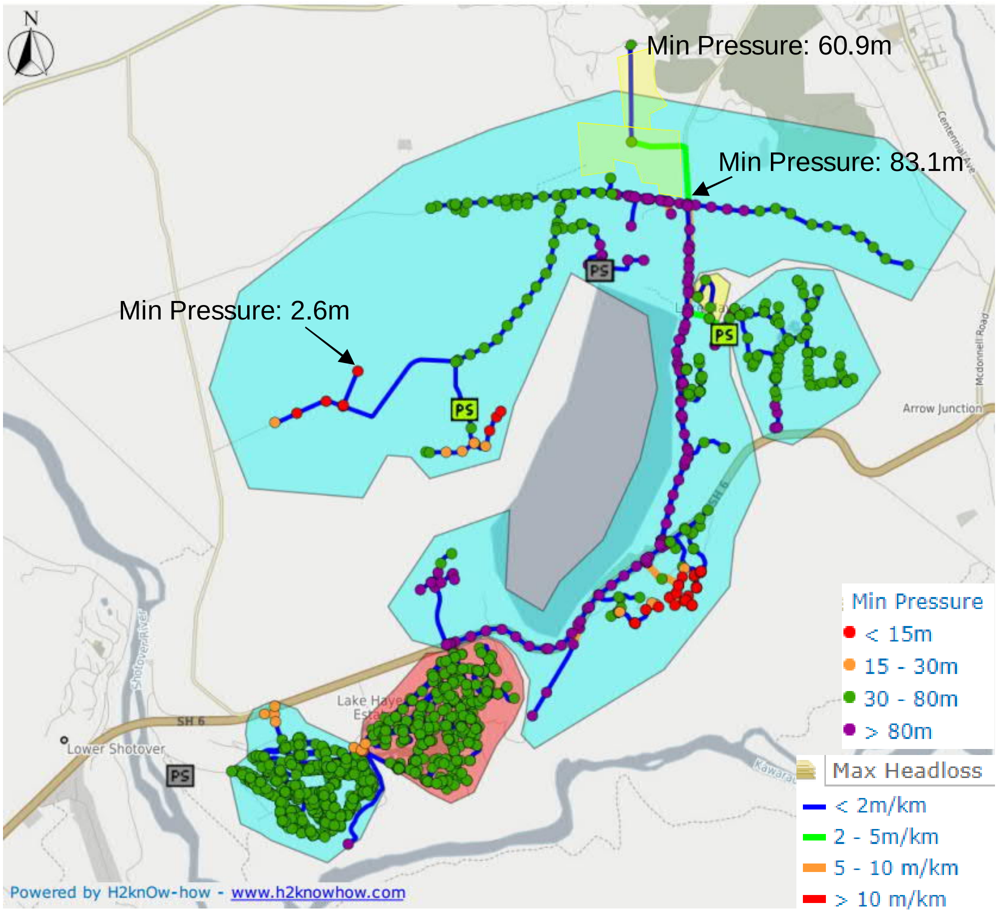

Figure 4 – Current Peak Day System Performance - Post Development

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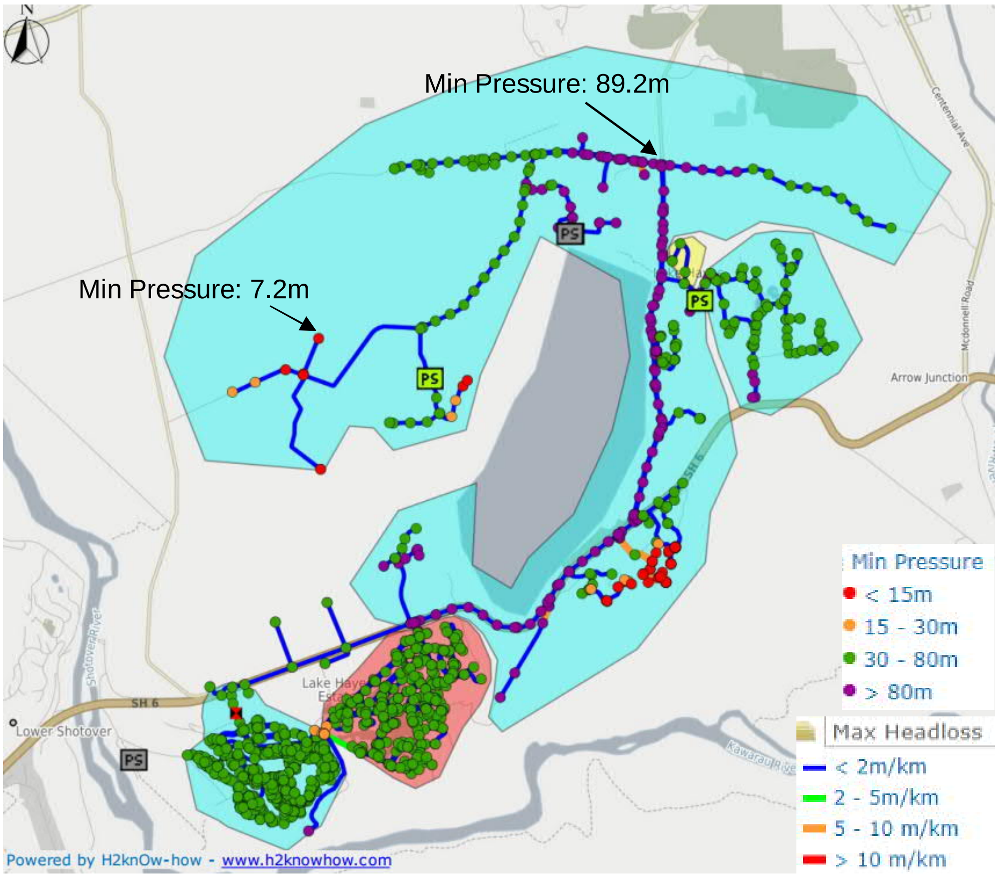

Figure 5 - 2028 Peak Day System Performance - Prior Development

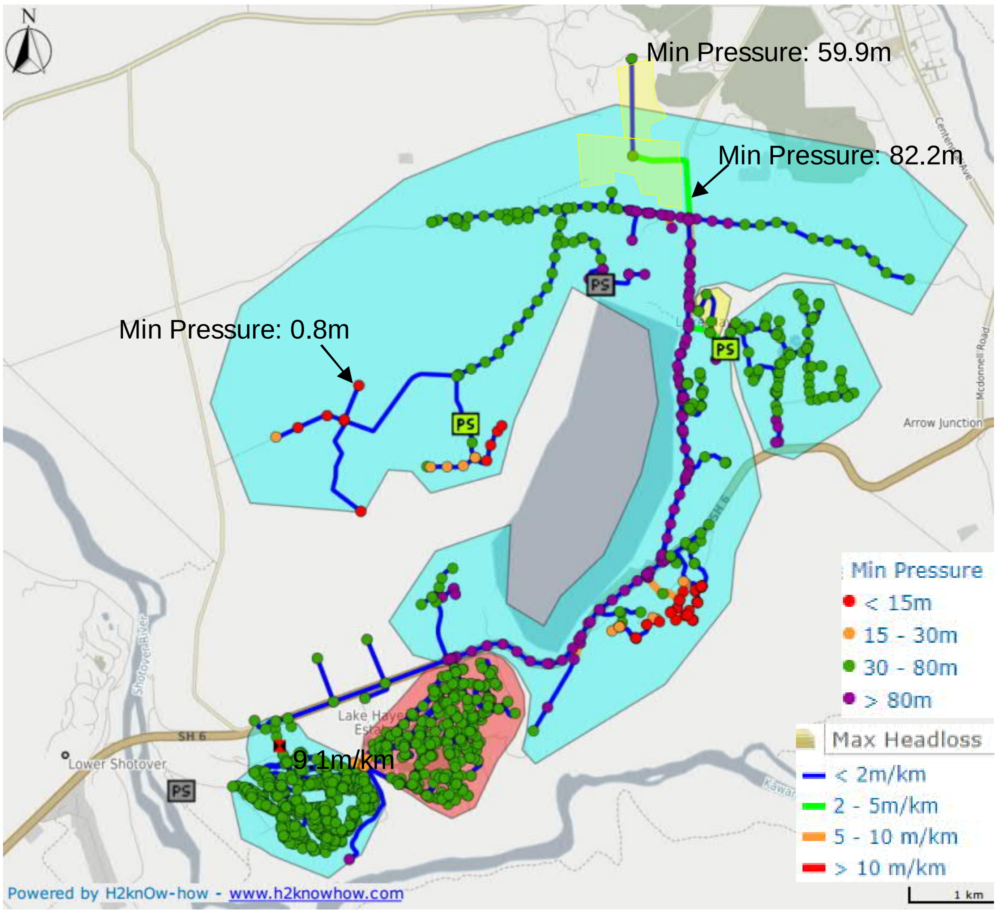

Figure 6 - 2028 Peak Day System Performance - Post Development

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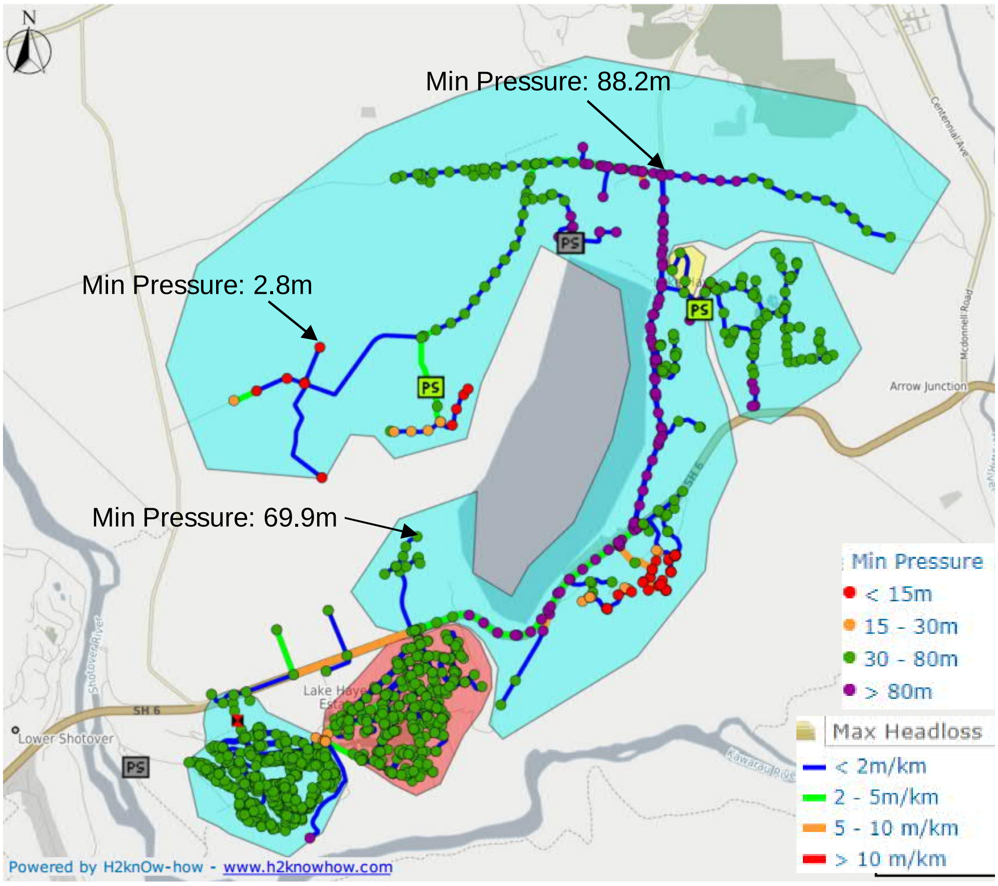

Figure 7 - 2058 Peak Day System Performance - Prior Development

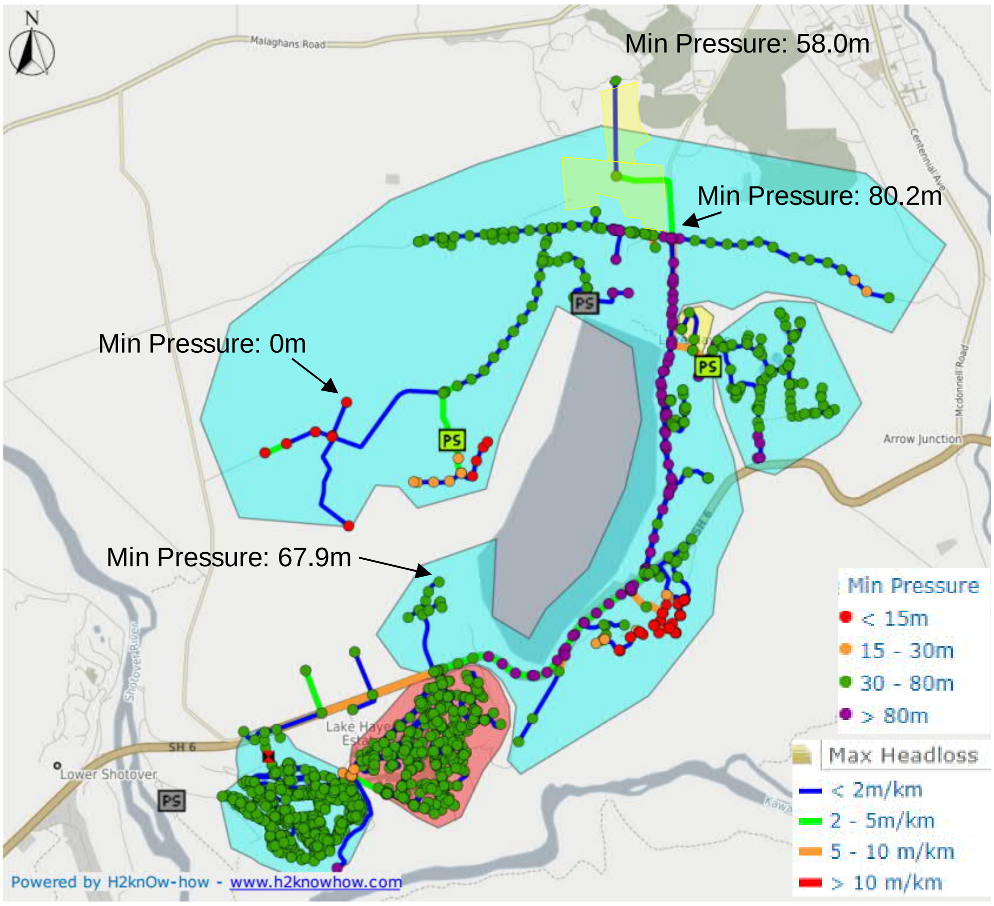

Figure 8 - 2058 Peak Day System Performance - Post Development

The table below summarises the maximum head losses in the existing 235mm ID pipe along Arrowtown Lake Hayes Rd and the minimum pressure forecasted at the supply point, before and after the proposed development:

Table 5 - Minimum Pressure at Supply Point

Demand	Min pressure before development (m)	Min pressure after development (m)	Pressure drop (m)
Current Peak Day	89.5	83.1	6.4
2028 Peak Day	89.2	82.2	7.0
2058 Peak Day	88.2	80.2	8.0

Table 6 - Maximum Head Losses in 235mm ID Pipe

Demand	Max head losses before development (m/km)	Max head losses after development (m/km)	Head losses increase (m/km)
Current Peak Day	0.4	6.0	5.6
2028 Peak Day	0.6	6.6	6.0
2058 Peak Day	1.1	7.8	6.7

As shown in the pictures and above tables, the proposed development is predicted to have a noticeable impact on the remaining of the water network with a maximum pressure drop of 8.0m. Pressures are generally high along Arrowtown Lake Hayes Rd and Speargrass Flat Rd, so pressure remains well above the recommended LOS in this area, for current and future scenarios. However, pressures below the recommended LOS are predicted in the properties located in the elevated areas of Slope Hill Rd and Threewood Rd. This is an existing LOS issue that needs to be addressed.

Head losses are predicted to increase by up to 6.7m/km reaching 7.8m/km in the 235mm (ID) along Arrowtown Lake Hayes Rd due to the additional demand. The predicted head losses exceed the recommended LOS, 5m/km. This LOS issue needs to be addressed.

5 Conclusions and Recommendations

Demand from the proposed Waterfall Park development has been added to the network for the current, future 2028 and 2058 peak day models to determine if suitable levels of service could be obtained.

Levels of service are expected to be met in terms of minimum pressure and head losses in the proposed development, however pressures higher than the recommended LOS are predicted in areas below 349m. The model predicts that fireflow requirements (FW2 – 25l/s and 16.6l/s sprinklers flow) can be provided with a residual pressure of 47m at RL 368m, for current and future scenarios. The highest elevation that would be serviceable for the residential development is 395m.

The system performance in the remaining of the network has been verified. The proposed development is predicted to cause a maximum pressure drop of 8m at the connection point. Since pressures are high in this area recommended LOS can still be met in terms of pressure. However, pressures dropping to zero are predicted in 2058 in properties located in the elevated areas of Slope Hill Rd and Threewood Rd due to the additional demand. These areas already experience pressures below the recommended LOS, the additional demand causes the pressure to deteriorate even further.

Maximum head losses greater than 5 m/km are predicted along Arrowtown Lake Hayes Rd for all scenarios. This system performance issue is related to the additional demand, the proposed development impact needs to be mitigated.

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Revision	Date	Originator	Checker	Approver	Description
A	23/02/2018	Diana Galindo	Julie Plessis	Julie Plessis	Draft for client review
B	19/03/2018	Diana Galindo	Julie Plessis	Julie Plessis	Draft for client review
C	30/05/2018	Diana Galindo	Nasrine Tomasi	Nasrine Tomasi	Final

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6 Appendix - Demand Calculation and proposed Pipe Layout

Waterfall Park Water Demand Estimate Summary

Table 1: Waterfall Park Hotel Complex - Water Demand Estimate

Hotel facility	No. Facilities	Max no. People / Facility	Average Daily Water Demand (L/p/d)	Average Daily Water Demand (m3/day)	Average Daily Water Demand (L/s)	Peak Hour Peaking Factor	Peak Hour Demand (L/s)	Peak Day Peaking Factor	Peak Day Demand (L/s)	Comment / Reference
Hotel Room	380	2	220	167.2	1.94	6.6	12.77	3.30	6.39	*AS/NZS 1547:2012, Table H4.*
Conference Centre	1	600	30	18	0.21	6.6	1.38	3.30	0.69	*Metcalfe and Eddy, Table 3-2. Wedding can occur at same time as conference*
Restaurants	1	1520	30	45.6	0.53	6.6	3.48	3.30	1.74	*AS/NZS 1547:2012, Table H4. Restaurants can seat 270 people. Assume hotel full (760 people) asssume each person eats two meals at hotel, total no. diners = 1520 over a day*
Lounge Bar and bar	1	250	20	5	0.06	6.6	0.38	3.30	0.19	*AS/NZS 1547:2012, Table H4. Lounge and bar can accommodate 115 people, assume 250 people max over a day*
Chapel / wedding venue	1	100	40	4	0.05	6.6	0.31	3.30	0.15	*Assume 40L/guest. Wedding can occur at same time as conference.*
Wellness centre - pool, gym, spa	1	100	40	4	0.05	6.6	0.31	3.30	0.15	*Metcalfe and Eddy Table 3-4 for swimming pools. Assume pool is filled overnight when irrigation is not running.*
Non residential staff	1	120	30	3.6	0.04	6.6	0.28	3.30	0.14	*AS/NZS 1547:2012, Table H4.*
Irrigation demand	1	n/a	n/a	125	1.45	n/a	n/a	n/a	4.35	*Based on calculated irrigation requirements with irrigation over an eight hour period overnight*
Total				**372.59**	**4.31**		**18.90**		**13.80**	

Table 2: Waterfall Park Residential Development - Water Demand Estimate

Hotel facility	No. Dwellings	No. people/ dwelling	Average Daily Water Demand (L/p/d)	Average Daily Water Demand (m3/day)	Average Daily Water Demand (L/s)	Peak Hour Peaking Factor	Peak Hour Demand (L/s)	Peak Day Peaking Factor	Peak Day Demand (L/s)	Comment / Reference
Primary Dwelling	125	3	700	262.5	3.04	6.6	20.05	3.30	10.03	*Total of 125 lots*
Secondary Dwelling	125	2	350	87.5	1.01	6.6	6.68	3.30	3.34	*Assume each lot may also have a secondary dwelling. Assume average of 2 person occupancy per secondary dwelling, assume no irrigation requirements for secondary dwelling*
Total				**350.00**	**4.05**		**26.74**		**13.37**	

Notes:

- Average day to peak hour peaking factor of 6.6 has been applied as per QLDC CoP Section 6.3.5.6

- The average day to peak day peaking factor is assumed to be 50% of average day to peak hour peaking factor

- It is assumed that each residential lot may have a primary dwelling and a secondary dwelling

References:

Metcalfe and Eddy, 2003, Wastewater Engineering: Treatment and Reuse, McGraw-Hill

AS/NZS 1547:2012 - Onsite wastewater management

QLDC Land Development and Subdivision Code of Practice, 2015