Waterfall Park Developments Ltd

Northbrook Retirement Village

Water, Wastewater, and Stormwater Infrastructure and Flood Assessment

May 2020

Fluent SOLUTIONS

Waterfall Park Developments td

Northbrook Retirement Village
Water Wastewater and Stormwater Infrastructure and Flood Assessment

Task	Responsibility	Signature
Project Manager:	Alexis Patrylak	
Prepared By:	Jayne Richards / Klemens Markiewicz / Alexis Patrylak	
Reviewed By:	Anthony Steel / Gary Dent	
Approved For Issue By:	Jayne Richards / Alexis Patrylak	

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Prepared By:
Fluent Infrastructure Solutions Ltd
Suite 2, First Floor
23-27 Beach Street
Queenstown 9300
Telephone: + 64 3 974 4586
Email: office@fluentsolutions.co.nz
Web: www.fluentsolutions.co.nz

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1.0 Executive Summary

This report covers a high level three–waters infrastructure overview of the proposed Northbrook (Arrowtown) Retirement Village Development. It finds 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 pump station delivering to existing wastewater reticulation along the Waterfall Park Access Road and the connection point to existing sewer reticulation at Arrowtown–Lake Hayes Road. A small number of residential units will require a small package pump station to convey their wastewater into the gravity reticulation network.

Water demand can be met by gravity supply from the Lake Hayes scheme via a connection point to existing water reticulation installed along the Waterfall Park Access Road.

Stormwater within the Northbrook site will be collected in a pipe conveyance system and treated before being discharged to Mill Creek after undergoing sufficient treatment to reduce contaminant loading through the use of treatment ponds and swales. Flood mitigation has been achieved to ensure floor levels have sufficient freeboard and post–development flows at the downstream boundary of the site are estimated to be less than the pre–development flows or would have minimal effect.

2.0 Introduction

2.1 eneral

Fluent Infrastructure Solutions Limited (FS) has been engaged by Waterfall Park Developments Ltd to undertake a water, wastewater, and stormwater infrastructure assessment and flood hazard assessment for the proposed Northbrook Retirement Village development. Infrastructure and flood mitigation for the Access Road and adjacent Waterfall Park Hotel development were assessed in previous resource consent applications (RM171280, RM17.302.01–02, RM18.088.01–0.5, and RM180584).

This report has been prepared to support an application for resource consent for the Northbrook Retirement Village. Note that this report does not address the ecology of Mill Creek or its tributaries in relation to the proposed works. An ecological assessment is being provided in a separate report prepared by Ryder Environmental.

2.2 Site ocality and Features

The proposed Northbrook Retirement Village development area is located to the north of Lake Hayes and approximately 3km southwest of Arrowtown, as shown in Figure 2.1 below.

The Northbrook Retirement Village is situated on relatively gently sloping land. To the north of the development extent, there is a hill catchment characterised by grassed pastures on a relatively steep slope. To the east, the retirement village is bounded by the main Waterfall Park Access Road and the adjacent Mill Creek. A small, spring–fed tributary to Mill Creek runs through the development site and discharges towards the southeast.

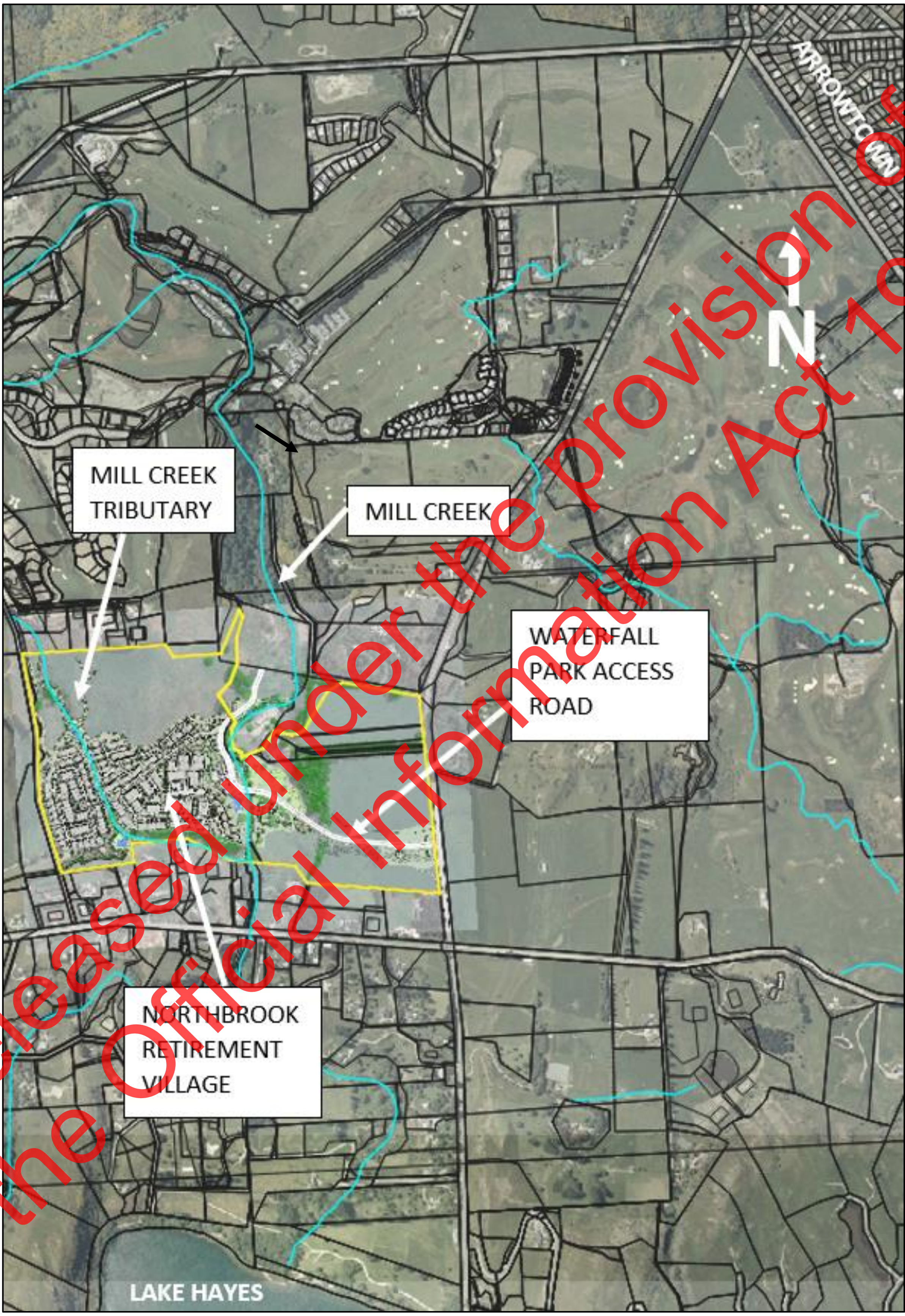

Figure 2.1: Site   ocation and Features

2.3 Site Ha ards Information

The Mill Creek tributary runs through an area defined as an active debris–dominated alluvial fan according to the Queenstown Lakes District Council (QLDC) hazard maps. Reviewing the topography of the site, the hill slopes to the north of the site do not show visible signs of debris flows. An assessment of the site hazards has been described in the Geosolve Geotechnical Report (February 2020).

Additionally, there is also an indicated flooding hazard located along Mill Creek. The flood hazard has been addressed through the construction of the main Waterfall Park Access Road and was assessed in previous consents.

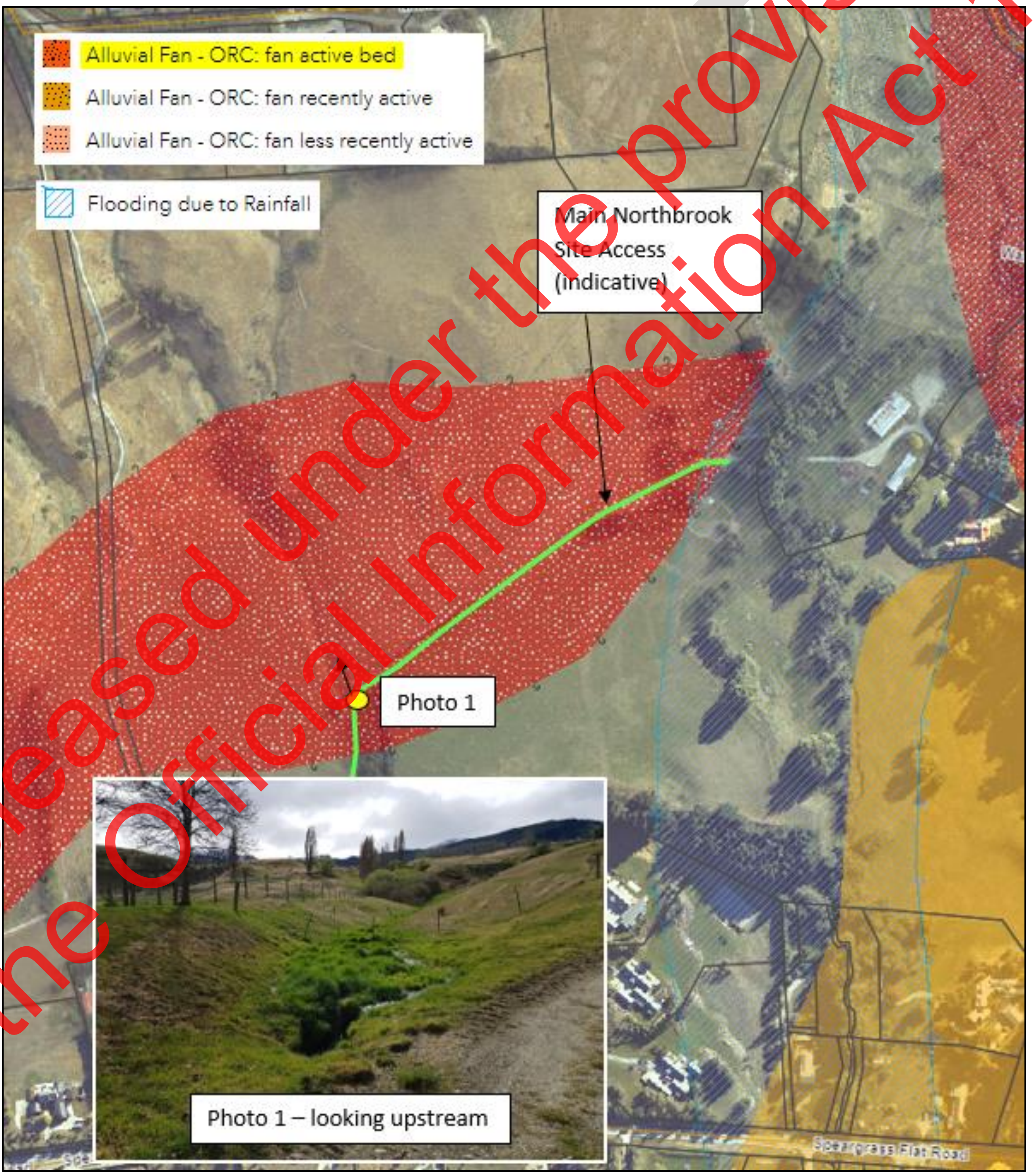

Figure 2.2: Site Ha ard Map – Q DC IS Mapping (RC Ha ard Data)

3.0 The Proposed Development Plan

Figure 3.1 shows the general layout of the proposed retirement village development.

The proposed development on which this infrastructure assessment has been undertaken comprises:

- 162 Residential Units
- 36 rooms within an Aged Care Facility
- Clubhouse
- Reception and BoH Facilities
- Active Recreation Building (gym, pool, and fitness)
- Childcare Centre
- Medical Centre
- Mobility Scooter Parking and Bus Stop
- Outdoor Recreation Area and Golf Holes – located on the east side of Mill Creek

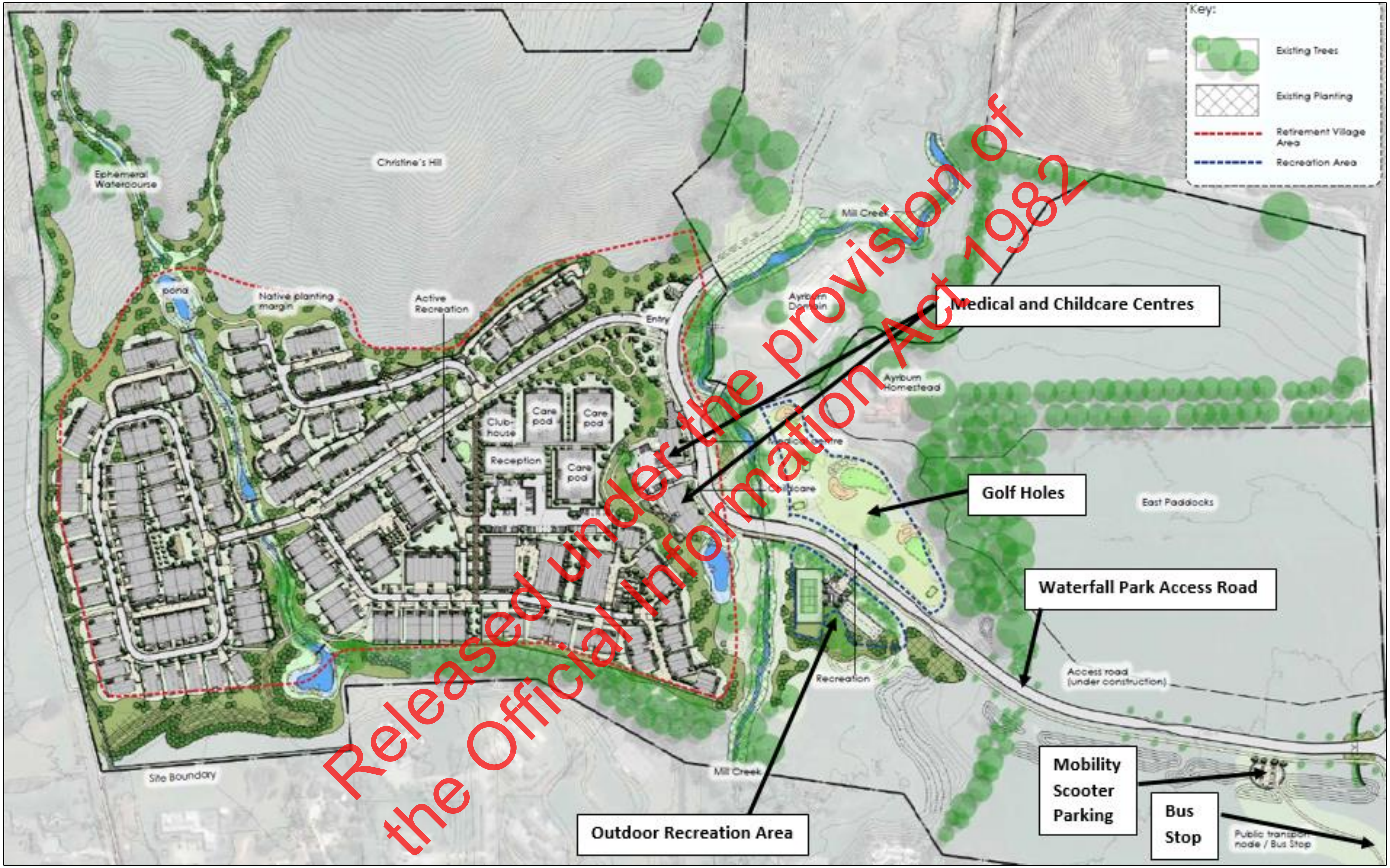

Figure 3.1: Proposed Northbrook Retirement Village Development Plan

4.0 Wastewater

4.1 Wastewater Collection and Conveyance System Design

The design, sizing, and layout of the wastewater collection and conveyance network to service the proposed Northbrook Retirement Village is related to 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 units and aged care residents and the number of patrons of the other proposed facilities);
- Wastewater production – both peak wet weather and peak dry weather;
- Capacity of the existing QLDC infrastructure to convey the wastewater loads; and
- Wastewater pumping requirements.

4.2 Wastewater Flows

The following wastewater design flows have been established for the proposed Northbrook Retirement Village as shown in Table 4.1 below.

Table 4.1: Wastewater Design Flows

Northbrook Village Facility	No. of Buildings	Max No. of People / Facility/Day	Average Per Capita Daily Wastewater Production (L/p/d)	Daily Wastewater Production (m^3/d)	Dry Weather Diurnal Peaking Factor	Peak Dry Weather Flow (L/s)	Wet Weather Peaking Factor	Peak Wet Weather Flow (L/s)
Residential Units	162	2	250	81	2.5	2.34	2	4.69
Aged Care Rooms	36	1	250	9	2.5	0.26	2	0.52
Aged Care Staff	1	22	50	1.1	2.5	0.03	2	0.06
Childcare Centre	1	60	30	1.8	2.5	0.05	2	0.10
Childcare Staff	1	10	50	0.5	2.5	0.01	2	0.03
Medical Centre	1	192	20	3.84	4	0.18	2	0.36
Medical Centre Staff	1	10	50	0.5	4	0.02	2	0.05
Active Recreation (including staff)	1	330	40	13.2	4	0.61	2	1.22
Clubhouse (including staff)	1	580	30	17.4	4	0.81	2	1.61
Other Staff	1	14	50	0.7	2.5	0.02	2	0.04
Total				**129.04**		**4.34**		**8.68**

Building occupancies in Table 4.1 above have been selected to reflect maximum estimated daily wastewater production. These design occupancies may vary from building occupancies relevant to fire safety/vehicle numbers etc.

As the Northbrook development is not a conventional residential subdivision, it is not directly covered by QLDC's Land Development and Subdivision Code of Practice 2020 (QLDC COP 2020). The design criteria for the development has therefore generally been established from first principles, but with reference to the QLDC COP 2020 and AS/NZS 1547:2012 *On–site Domestic Wastewater Management*.

Average dry weather design flows are based on 250 litres per person per day (L/p/d) for the residential units and aged care residents, with a peaking factor of 2.5 for the dry weather diurnal and a dilution/infiltration factor of 2 for wet weather. For non–residential facilities and staff, varying wastewater production volumes have been selected based on AS/NZS 1547:2012 as well as estimated water demands. A dry weather diurnal peaking factor of 4 has been applied to the Medical Centre, Active Recreation building, and Clubhouse.

For the purpose of a conservative wastewater assessment this takes into consideration the operating hours of these facilities, estimated to be between 12–16 hours each per day. Like the residential units and Aged Care Centre, these facilities and staff have a dilution/infiltration factor of 2 for wet weather.

The assessment identifies a peak daily wastewater production of just over 129m^3 and a peak wet weather wastewater flow of 8.7L/s.

4.3 Existing Q DC Infrastructure

Wastewater from Arrowtown is currently pumped to a manhole located east of the proposed development on the Arrowtown–Lake Hayes Road (manhole ID SM11784, refer to Figure 4.1). 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 Road 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.

An existing 150mm mPVC sewer main drains wastewater from the properties south of the Waterfall Park development area and north of Lake Hayes to the Lake Hayes Sewer Pump Station #1 (located north of Lake Hayes). From there, 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.

A 160 OD PE100 PN12.5 wastewater rising main has been installed along the Waterfall Park Access Road to service the proposed Waterfall Park Hotel development and proposed Northbrook Retirement Village. A connection of the new rising main to the Arrowtown–Lake Hayes wastewater trunk main has been approved.

Figure 4.1 presents an overview of the main existing sewer infrastructure in the vicinity of the Waterfall Park Development area.

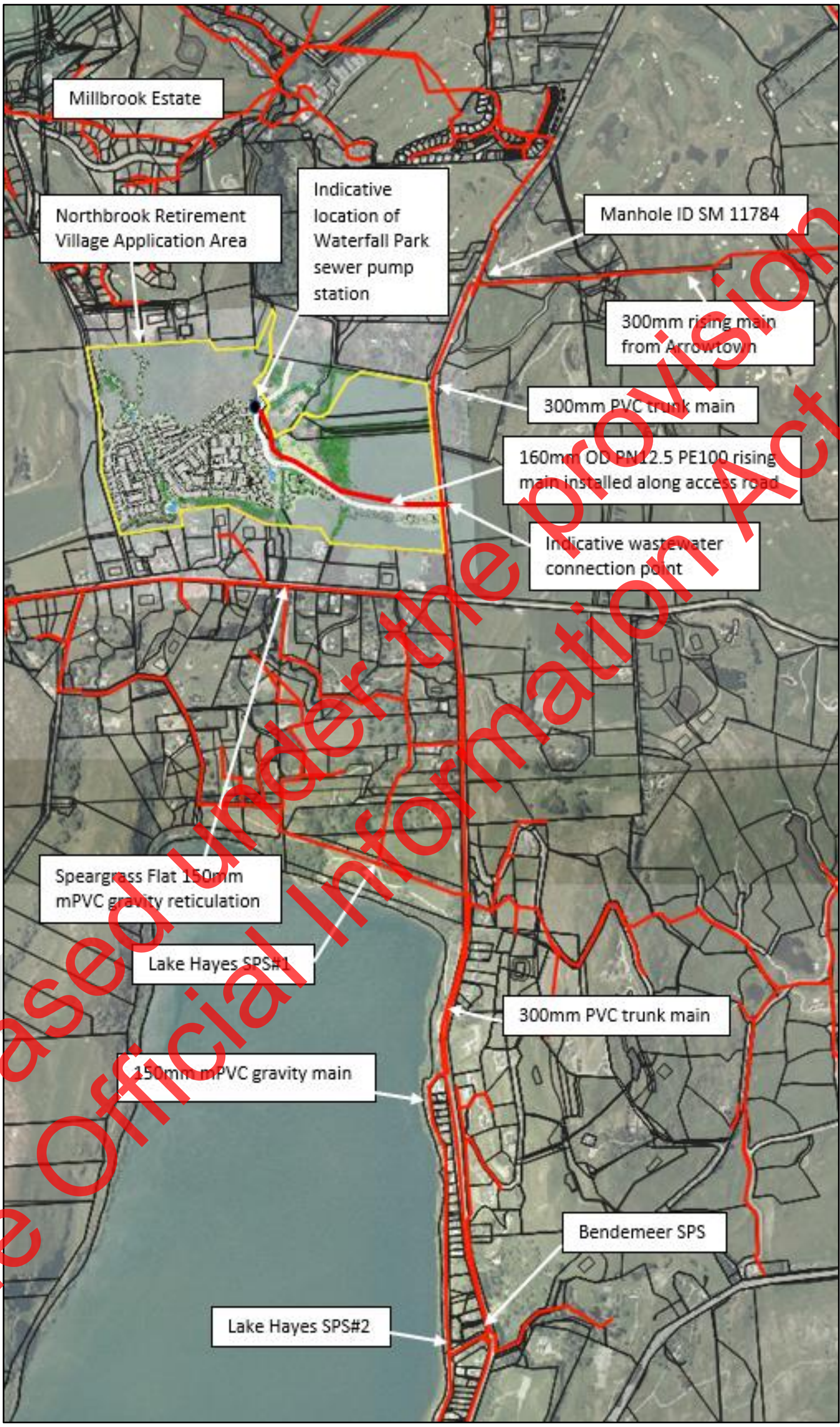

Figure 4.1: Schematic overview of existing sewer services in the vicinity of the Northbrook Retirement Village development with the location of a sewer pump station indicated

4.3.1 Capacity of Existing Infrastructure

The capacity of the existing wastewater infrastructure to carry the additional flows from the proposed Waterfall Park Hotel and an adjacent residential development was modelled by QLDC's modelling consultants, BECA, during February 2018. An Addendum to the report was provided by HAL consultants in January 2019. The reports are provided in the Appendices.

At the time of the wastewater modelling, Waterfall Park Developments Ltd were considering a residential development (referred to as Ayrburn Farm) at the Northbrook Retirement Village site. The residential development was estimated to have a peak wet weather flow of 9L/s. The peak wet weather flow estimated for the proposed Northbrook Retirement Village is less than the design flow previously modelled (8.68L/s compared with 9L/s).

The results of the modelling found that the existing 300mm uPVC trunk main running along Arrowtown–Lake Hayes Road has adequate capacity for the additional load from the Waterfall Park Hotel and the residential scenario (now Northbrook Retirement Village) for both the current, 2028, and 2058 design horizons without the need for any infrastructure upgrades. The modelling also indicated that the 150mm mPVC gravity reticulation north of Lake Hayes did not have adequate capacity to carry flows from the hotel and therefore this option has not been progressed.

A new 160 OD PE100 PN12.5 wastewater rising main, now installed alongside the new Waterfall Park Access Road, has been sized to accommodate the flows for both the Waterfall Park Hotel and the Northbrook Retirement Village. The proposed sewer pump station located as shown in Figure 4.1 has been sized to accommodate the Northbrook Retirement Village as part of the Waterfall Park Hotel detailed design.

4.4 Wastewater Servicing for the Proposed Development

From the investigations and modelling undertaken, it is clear that the existing 300mm uPVC trunk main along Arrowtown–Lake Hayes Road has adequate capacity to accept sewer flows from the proposed Northbrook Retirement Village as well as the Waterfall Park Hotel. The 160 OD PE 100 PN12.5 rising main also has capacity to convey flows from the retirement village to the trunk main.

Wastewater servicing for the large majority of the proposed development will comprise of conventional gravity sewer reticulation, falling to the proposed main wastewater pump station located adjacent to the Waterfall Park Hotel Access Road (refer Figure 4.1). Wastewater will be pumped from the main wastewater pump station through the existing 160 OD wastewater rising main and into the 300mm PVC trunk main in the Arrowtown Lake Hayes Road.

Wastewater from six residential units in the southeast corner of the development and wastewater from the Medical Centre and Childcare Centre buildings is not able to be conveyed by gravity. Small package style pump stations are proposed to convey wastewater from these two areas to the gravity–fed wastewater network. The six residential units, Medical Centre, and Childcare Centre buildings that are proposed to feed the small package pump stations are shown on the Paterson Pitts Drawings (refer specifically to Sheet 404).

The main wastewater pump station will be a private pump station but will be designed to meet QLDC's standards such that does not preclude it from being vested to Council in the future, if required.

5.0 Water Supply

5.1 Water Supply System Design

The design, sizing, and layout of the water supply network to service the proposed Northbrook development is related to 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 units and aged care residents and the number of patrons of the other proposed facilities);
- Water demands – both peak and fire fighting requirements;
- Water supply availability;
- Water pressure requirements;
- Water storage requirements;
- Landscaping irrigation requirements; and
- Water quality requirements.

5.2 Water Demand Assessment

5.2.1 Domestic and Irrigation Water Demands

As noted in the wastewater assessment presented above, the proposed Northbrook Retirement Village development differs from a conventional residential subdivision in regard to both domestic/commercial water demands and irrigation requirements. For normal residential subdivisions, the property occupancy varies from house to house and can vary seasonally. Water for irrigation use is in the hands of individual households and is largely uncontrolled. For this reason, QLDC sets criteria to cover irrigation requirements on a per capita basis at 700L/p/d, noted in the QLDC COP 2020.

For the Northbrook Retirement development, however, there is greater control over water consumption and irrigation is controlled by the retirement village management company rather than individual residents. The water demand is therefore assessed on a more direct first principles approach. This allows the estimated domestic water demand of the residents to be reduced. The domestic water demand of a 250L/p/d has been adopted (the same as the wastewater demands). Other water demands have been assessed in regard to more specific activities within the development.

Table 5.1 sets out the assessed domestic/commercial demands for the proposed development. The peaking factors provided in the QLDC COP 2020 have been used for the peak hour water demand for most facilities. For the purpose of a conservative assessment, some non–residential facilities have a peak hour factor of 10 applied, as the operating hours (estimated to be between 12–16 hours each day) would impact the peak hour demand. These factors are considered appropriate for this preliminary analysis in terms of providing a conservative demand estimate. Specific irrigation demands are outlined further in Table 5.2 below.

Table 5.1: Assessed Water Supply Design Volumes and Flows

					Case 1: Domestic Peak Hour (day time only, no irrigation)		Case 2: Domestic Peak Hour (overnight time, with irrigation)	
Northbrook Village Facility	No. of Buildings	Max No. of People / Facility/Day	Daily Water Demand Per Capita (L/p/d)	Daily Water Demand (m3/d)	Peak Hour Peaking Factor	Peak Hour Demand (L/s)	Peak Hour Peaking Factor (50% Peak)	Peak Hour Demand (L/s)
Residential Units	162	2	250	81	6.6	6.19	3.3	3.09
Aged Care Rooms	36	1	250	9	6.6	0.69	3.3	0.34
Aged Care Staff	1	22	50	1.1	6.6	0.08	3.3	0.04
Childcare Centre	1	60	30	1.8	6.6	0.14	3.3	0.07
Childcare Staff	1	10	50	0.5	6.6	0.04	3.3	0.02
Medical Centre	1	192	20	3.84	10	0.44	5.0	0.22
Medical Centre Staff	1	10	50	0.5	10	0.06	5.0	0.03
Active Recreation (including staff)	1	330	40	13.2	10	1.53	5.0	0.76
Clubhouse (including staff)	1	580	30	17.4	6.6	1.33	3.3	0.66
Other Staff	1	14	50	0.7	6.6	0.05	3.3	0.03
Added Daily Irrigation Volume				**165**				**5.73**
Total				**294.04**		**10.55**		**11.00**

Building occupancies in Table 5.1 above have been selected to reflect maximum estimated daily water demand. These occupancies may vary from building occupancies relevant to fire safety/vehicle numbers etc.

Table 5.2 sets out the assessed irrigation requirement for the Northbrook Retirement Village development. A weekly averaged irrigation application rate of 5mm/day on lawns and landscaped areas has been adopted. This is a conservative allowance for concept design purposes. The irrigation will be on a managed basis over an 8–12 hour period per day, generally overnight, and more particularly avoiding peak domestic water demand periods during the day. This means that the daily irrigation demand will be relatively constant and not subject to the peaking characteristics typical of the domestic demands.

Two peaking factor scenarios have been considered:

- Case 1 – peak hour, with no irrigation (i.e. daytime peak)
- Case 2 – peak hour (50% domestic peak) plus irrigation over 8 hours (i.e. night time peak)

In Case 2, the peak hour has been reduced by 50% as it considers the night time peak, which would be significantly lower than the day time peak.

The irrigation demands were estimated based on an irrigation rate of 5mm/m²/day over the landscaped area, as shown in Table 5.2 below.

Any irrigation required during the early years of the development for plant establishment has not been included in the overall demand estimates in Table 5.1 as this irrigation will not occur when the buildings are occupied.

Table 5.2: Irrigation Assessment

Site	andscaped Area (m^2)	Daily Irrigation Rate ($mm/m^2/d$)	Irrigation Demand (m^3/day)	Case 2: 50 Peak Water Demand – Night Time with Irrigation (/s)
Northbrook Village (Permanent Irrigation)	33,000	5	165	5.73
Northbrook Village (Temporary Irrigation)	5,000	5	25	0.87

A point of note; albeit that Waterfall Park Developments Ltd has an existing water take consent to take up to 232.26m³/day from Mill Creek for irrigation, which will not be used for the Northbrook Development. It will be used for the proposed Waterfall Park Hotel.

From Tables 5.1 and 5.2, the following water demand requirements (excluding fire fighting) have been established:

- Peak Day Demand 294.04m³/day
- Domestic Peak Hour (daytime only, no irrigation) (Case 1) 10.55L/s
- Domestic Peak Hour (overnight, with irrigation) (Case 2) 11.00L/s

5.2.2 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 2013.* Assessment of the development's facilities and the building layouts has resulted in various fire fighting requirements as per *SNZ PAS 4509*, which are detailed in Table 5.3 below.

Table 5.3: Fire Fighting Re uirements of uildings

uilding	Water Supply Classification	Sprinkler System Re uired
Single Level Apartments	FW2	No
Multi–level Apartments	FW3	No
Aged Care Centre & Clubhouse	FW2	Yes
Active Recreation Building	FW3	No
Childcare Centre	FW3	No
Medical Centre	FW3	No
Maintenance	FW3	No

Facilities that fall under the FW2 water supply classification require a minimum fire fighting supply of a total of 25L/s from two hydrants, at a minimum pressure of 100kPa. An FW 3 water supply classification requires a building to have a minimum fire fighting supply of a total of 50L/s from a maximum of three hydrants at a minimum pressure of 100kPa.

The sprinkler requirements of the Aged Care Centre and Clubhouse have been assessed by Cosgroves Ltd to be a maximum of 12L/s at a pressure of 450kPa. A copy of their correspondence is in the Appendices. As the sprinkler system is in addition to the FW2 requirement, the total fire fighting demand of the Aged Care Centre is 37L/s (12 + 25L/s), in which the minimum residual pressure of 100kPa with hydrants at full flow is required at this location.

The ability of the existing water supply network to provide these firefighting demands is discussed in Section 5.3.2 below.

5.3 Existing Water Supply System

Properties south of the Waterfall Park 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 347–358m in the Waterfall Park Development area. These levels indicate that there should be adequate pressure available to supply the development from the Lake Hayes reservoir.

The existing water reticulation network in the vicinity of the proposed development is shown in Figure 5.1 below. A 315 OD PE100 PN12.5 water main has been installed along the Waterfall Park Access Road to service the consented Waterfall Park Hotel development. A connection has been made from QLDC's DN225 Arrowtown–Lake Hayes Road water main to the new 315 OD water main in the Waterfall Park Access Road. This 315 OD can also service the proposed Northbrook Retirement Village.

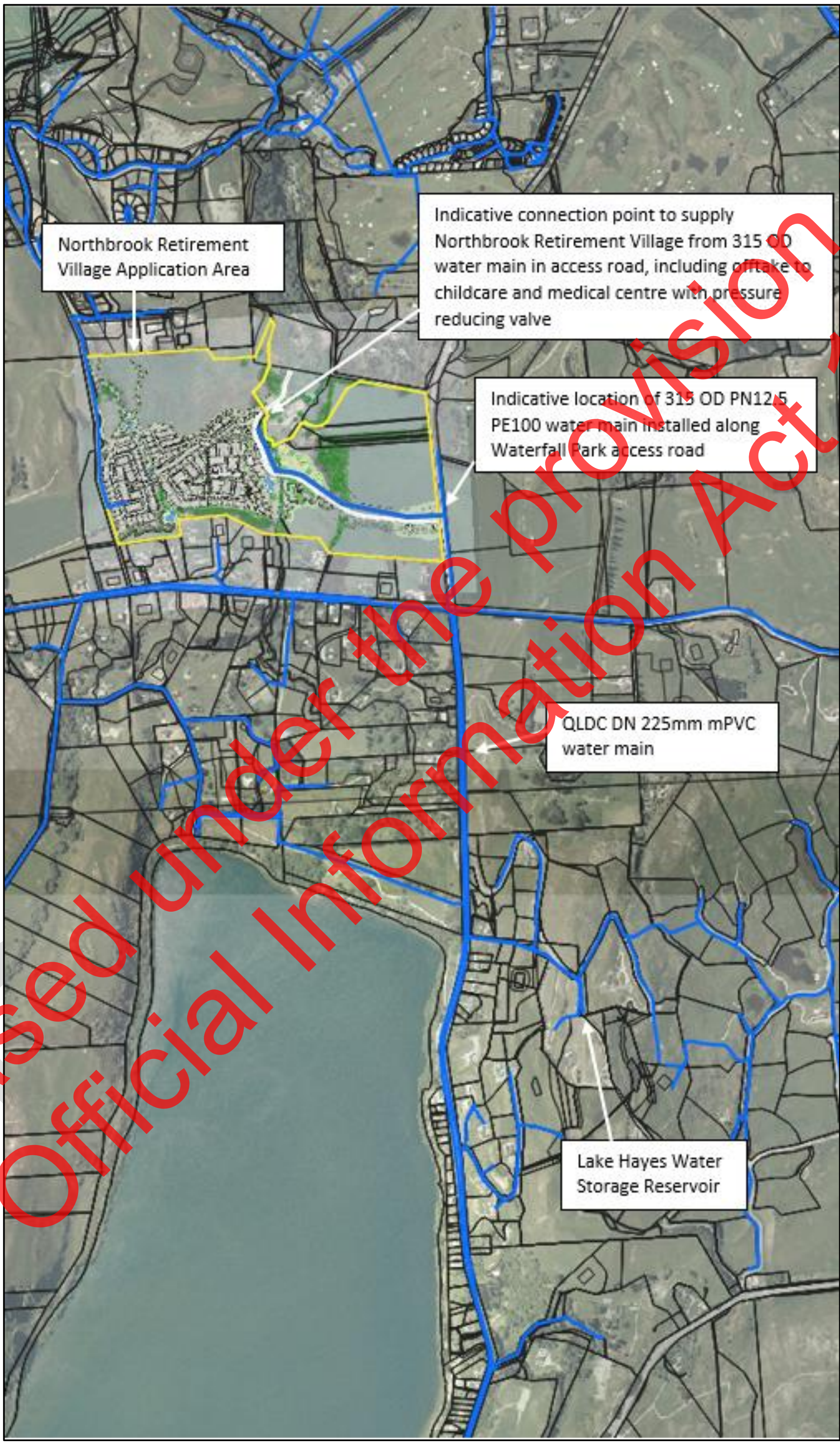

Figure 5.1: Schematic overview of existing water services in the vicinity of the Waterfall Park Development with the potential location of the proposed connection from the 315 D along the Waterfall Park Assess Road

5.3.1 Capacity of Existing Infrastructure – Peak Hour Demand

The capacity of the existing water supply infrastructure to service the Waterfall Park Hotel and an adjacent residential development was modelled by QLDC's modelling consultants, Mott MacDonald, during March and April 2018. Their report is provided in Appendix B.

At the time of the water modelling, Waterfall Park Developments Ltd were considering a residential development at the Northbrook Retirement Village site (called the Ayrburn Farm residential development). In 2018, Mott Macdonald modelled a combined peak flow of 45 L/s including 18.9 L/s for the Waterfall Park Hotel, 1.4 L/s for Ayrburn Domain and 24.7 L/s for the Ayrburn Farm residential development (refer to Table 5.4 below).

The Ayrburn Farm residential development is no longer proposed and has been replaced by the Northbrook Retirement Village (the present application). The estimated combined peak hour flows for the proposed Northbrook Retirement Village (11 L/s) are significantly lower than the estimated peak flows for the previously proposed Ayrburn Farm residential development (24.7 L/s), due to the control of irrigation (as stated in Section 5.2.1) refer Table 5.4. The overall peak hour demand for the Waterfall Park Hotel, the Northbrook Retirement Village, and Ayrburn Domain is approximately 31.3 L/s compared to the 45 L/s modelled during 2018 (a reduction of 13.7 L/s).

The results of the 2018 modelling found that the existing DN225mm mPVC Arrowtown–Lake Hayes Road water main has adequate capacity for the additional demand for both the Waterfall Park Hotel and proposed residential development, for both the current and 2028 design horizons without the need for any infrastructure upgrades. The modelling also identified high headlosses in the DN225 Arrowtown–Lake Hayes Road water main during the 2058 design horizon that exceeded the QLDC levels of service.

As the new proposed Northbrook Retirement Village has a lower peak hour demand (11 L/s) than the previously modelled residential development (24.7 L/s), the impact of the combined demand for the hotel plus the Northbrook Retirement Village on the water supply is significantly lower. A hydraulic review using the lower peak hour flow rate for the hotel and retirement village has found that the estimated headloss in the DN225 pipe along the Arrowtown–Lake Hayes Road reduces significantly during the 2058 design horizon and only slightly exceeds the QLDC levels of service. This is summarised in Table 5.4 below and the hydraulic calculations are provided in the Appendices.

Table 5.4: Summary of peak hour flows modelled by Mott MacDonald in 2018 and updated peak hour flows considering the Northbrook Retirement Village

	Waterfall Park Hotel Peak Hour (/s)	Ayrburn Farm Residential Peak Hour (/s)	Northbrook Retirement Village Peak Hour (/s)	Ayrburn Domain Peak Hour (/s)	Combined Peak Hour Demand (/s)	2058 Headloss in DN225 (m/km)
Original Flows Modelled By Mott MacDonald – 2018	18.9	24.7	–	1.4	**45**	7.8
Updated Flows with Northbrook Retirement Village	18.9	–	11	1.4	**31.3**	5.4*

**Assuming roughness coefficient k of 0.015mm. Headloss in DN225 is calculated based on Mott Macdonald's predicted 'existing' flows plus the additional flows from the Waterfall Park Hotel and Northbrook Retirement Village.*

The estimated 2058 headloss in the DN225 water main along the Arrowtown Lake Hayes Road is considered to be acceptable due to the high level of uncertainty associated with estimating flows 40 years in the future.

5.4 Water Servicing for the Proposed Development

From the investigations undertaken, it is clear that the existing DN225 mPVC water main in the Arrowtown-Lake Hayes Road and the 315 OD PE100 PN12.5 water main installed in the Access Road to service the consented Waterfall Park Hotel development has adequate capacity to provide the combined demands to the proposed Waterfall Park Hotel and Northbrook Retirement Village developments.

Water servicing within the proposed Northbrook Retirement Village area will comprise of conventional water reticulation sized to ensure that domestic, fire, and irrigation flows can be maintained at adequate pressures meeting the QLDC COP.

Pressure reducing of the water supply will be required where it services the medical centre and childcare facility as the pressure to these areas of the development has the potential to exceed the QLDC level of service of 90m due to their elevation, especially during periods of low demand. The pressure reducing valve has already been installed as part of the Access Road works.

6.0 Stormwater and Flood Management

6.1 ocal Catchment Stormwater and Flood Flows

6.1.1 Analysis Methodology Summary

In order to evaluate the effects of the development and the appropriate management mechanisms, a hydraulic model of the local catchment in the area around the Northbrook Retirement Village was developed.

The hydraulic and hydrological modelling program Infoworks ICM (ICM) was used to derive the overland flow patterns based on 2D hydraulic calculation algorithms built from 3D ground surface information and soil parameters.

Soil Parameters and Roughness

The Horton methodology was used for estimating infiltration losses to the soil based on soil infiltration values adopted from data provided by Akan (1993). The adopted soil values were based on a dry silty loam soil with little to no vegetation, an initial infiltration (f_0) of 101.6mm/hr, an ultimate infiltration (f_c) of 7.6mm/hr, and a decay rate of 4.1/hr.

Additional to the soil characteristics, the site roughness was also assessed. A roughness Manning's n (n) of 0.075 was chosen to represent the sheet and shallow flow, which delays the flow of water through the pastoral hill catchment upstream of the site. In the spring–fed tributary a roughness of 0.1 was chosen to represent the channel areas with thick vegetation and tree growth as observed by site visits.

Lastly, the impervious areas of the proposed development site were altered to allow for a high rate of runoff in the post–development scenario.

Rainfall and Climate Change

A series of triangular rainfall hyetographs (rainfall depth versus time graph) were developed for a range of storm durations and used in the model. The triangular hyetograph methodology, adopted by the Christchurch City Council "Advanced Analysis" method provided in the "Waterways, Wetlands and Drainage Guideline," using recorded data at the Queenstown Airport has been applied for this assessment.

The QLDC COP 2020 requires that climate change be a design consideration. The current QLDC COP 2020 requires inclusion of a temperature increase of 2.1°C to be included as part of the design rainfall hyetographs for the site.

It is understood the NIWA High Intensity Rainfall Distribution System (HIRDS) has recently been upgraded from Version 3 to Version 4. HIRDS Version 4 includes a series of new climate projection models of which the RCP8.5 (2081–2090 time period) model has generally been accepted by local governments and is understood to be preferred by QLDC over HIRDS Version 3 with a 2.1°C temperature increase.

Therefore, in order to provide a more conservative estimate of the flood flows around and within the site, HIRDS Version 4 RCP8.5 (2081–2090) rainfall data has been used in the design.

Mill Creek Flows

Flows in Mill Creek have previously been assessed and described in the previous reporting for the Waterfall Park Access Road and Hotel consents (RM171280 and RM180584). Estimated flood flows utilise the Generalised Extreme Value Flow Estimation Methodology and include an allowance for climate change.

Note that the Mill Creek flow path above Waterfall Park is a wide, flat valley that absorbs runoff from the surrounding catchment areas and delays and moderates the flood response at Waterfall Park. The stormwater runoff from the Northbrook Retirement Village site would be immediate compared to the flood response from the greater Mill Creek catchment. Therefore, peak stormwater runoff to Mill Creek would typically occur multiple hours before the peak flood flow from the upper Mill Creek catchment. The stormwater and flood peak flows would not be coincident.

Mill Creek Tributary Flows

The tributary to Mill Creek runs through the Northbrook development site (refer to Figure 5.1 below) and discharges to the southeast corner of the site. It is estimated that the tributary has an estimated spring flow of up to 10L/s, but site observation suggests a typical flow of closer to 0–2L/s.

6.1.2 Existing Stormwater and Flood Flow Pathways

Figure 6.1 below shows the existing flow paths within the local catchment around the Northbrook Retirement Village.

The topography of the land means that the site drains towards the spring–fed tributary and Mill Creek via natural overland flow paths. There is no existing stormwater infrastructure on the Northbrook Retirement Village site.

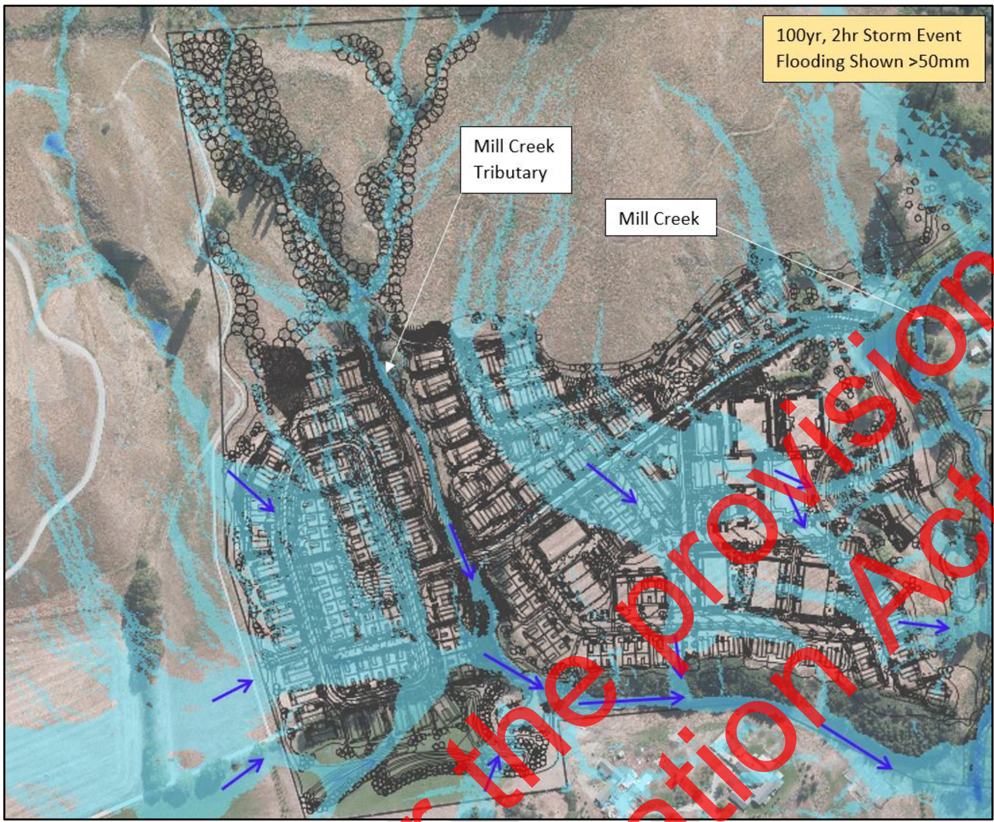

Figure 6.1: Existing Stormwater Flood Flow Pathways – 100yr 2hr Storm Event Flooding Shown 50mm

6.2 Proposed Stormwater Management Concept

The proposed stormwater management concept for the Northbrook Retirement Village provides for collection of stormwater runoff from roofs, roads, and open space which is conveyed in a pipe network system, which discharges into a “treatment pond” before being discharged to a “treatment swale”. Within the site, there are three separate sub–catchments, which are separated by the spring–fed tributary and the medical/childcare centre.

The stormwater system components are described in the following sections as well as shown in Figure 6.2 below.

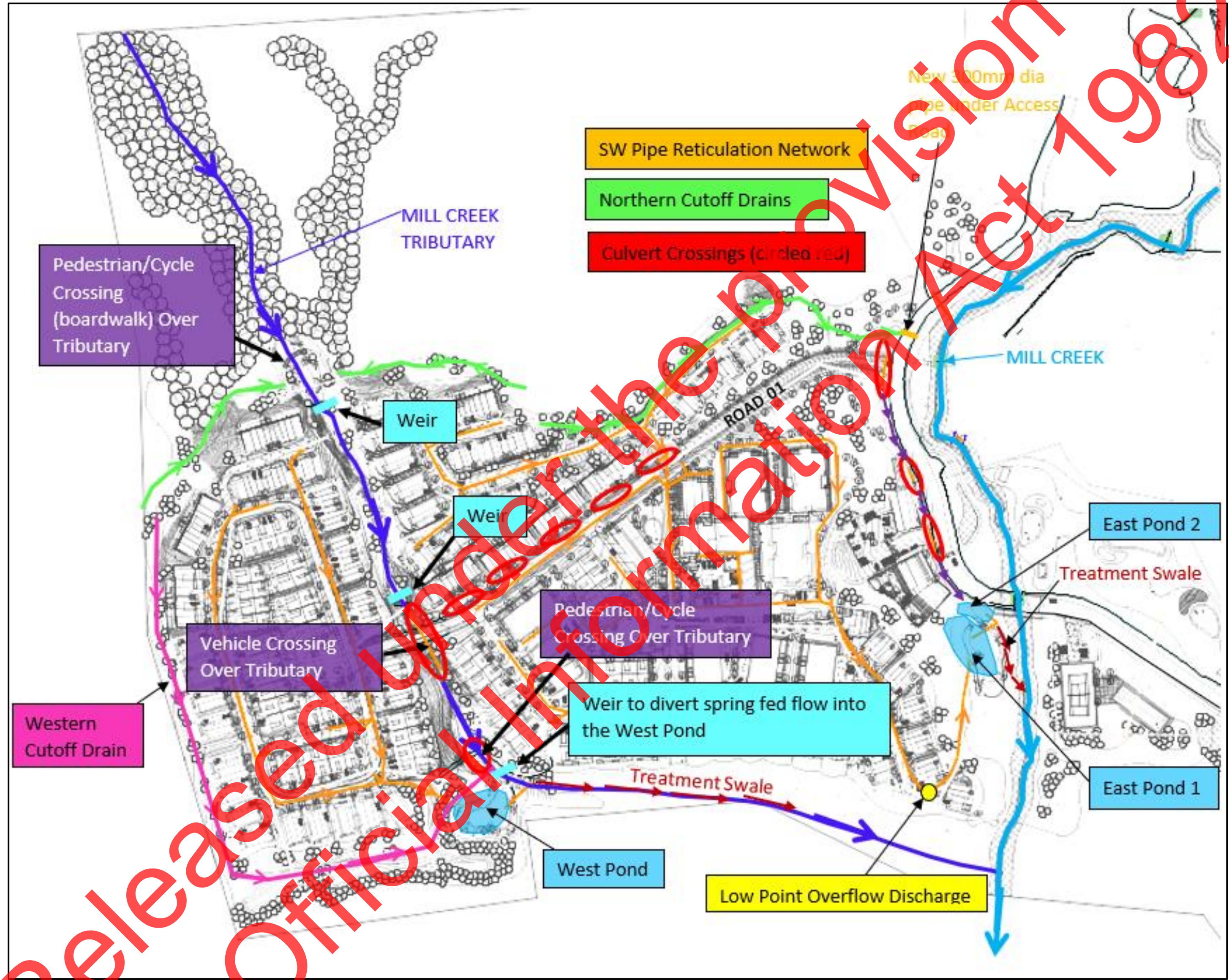

Figure 6.2: Stormwater Management Concept

6.2.1 Primary Pipe Conveyance System

Stormwater runoff from roofs and roads are collected and discharged directly into the conventional stormwater conveyance pipe network installed in the road reserve (pipe network sized to carry the 20yr Average Recurrence Interval (ARI) peak flow with no surcharging of manholes).

6.2.2 Secondary Overland Flow Paths

The roads are designed to convey flows over and above the pipe network capacity for large storm events and direct overland flow into inlet sumps which discharge to the pipe conveyance system.

During major events and/or very high intensity rainfall events, overland flow from garden and recreation areas would be intercepted by the roads and hence enter sumps in the centre of the road.

6.2.3 Low Point Overflow Discharge

In the event of a large or extreme rainfall event or blockage, an overflow point is located near the main pipe discharge locations to the East treatment ponds. At this point, the flood water would pond up until it overtops the kerb and flows over the hill. It is noted that the estimated ponding level is a maximum of 50–100mm and the estimated overflow rates are less than 100L/s and only occur during major storm events or in the event of a blockage.

6.2.4 Cut–off Drains

The cut–off drains are designed to intercept the clean water flood flows from the hillside catchments to the north and west and discharge flows into the Mill Creek tributary or the table water drain on the north side of the main Northbrook access. The flow from the hillsides is relatively small. Flows from the north are estimated at a peak of 0.02–0.075m^3/s for the 100yr ARI and flows from the west are approximately 1.2m^3/s peak for the 100yr ARI.

A maintenance regime would be formed as part of the detailed design in order to ensure debris flows from upstream do not reduce the capacity of the drains.

6.2.5 Culvert Crossings

There are a total of five main culvert crossings along the Waterfall Park Access Road swale and the Mill Creek tributary. Additionally, there are a series of driveway crossings and a road crossing over the table drain in the Northbrook Retirement Village main drive (Road 01).

- The culvert diameter for the culvert under the main Northbrook accessway road (Road 01), collecting flow from the Mill Creek tributary, is estimated to be approximately 750mm diameter in order to convey the 0.75–0.9m^3/s peak flow in the tributary.
- There is also a culvert for a footpath crossing over the tributary, located downstream of the accessway road crossing, mentioned above.

- There are also three culvert crossings located directly adjacent to the main Waterfall Park Access Road. These culvert crossings are connected via a swale used for conveyance of the 100yr ARI flow from the western hillside, Northbrook Access Road, and nearby buildings and carparks.
- Lastly, there are a series of culvert crossings over the northern table drain of Road 01. These are designed to allow for driveway access to villas adjacent to Road 01 and access to the northern villas via the feeder road.

6.2.6 Stormwater Treatment System

utline

The proposed treatment regime is to utilise mud–tanks within the primary pipe collection and conveyance network that would discharge to a treatment pond to capture the finer suspended sediment and further reduce the typical urban contaminant loads followed by a grassed or vegetated treatment swale for polishing the flow from the pond. For the frequent minor rainfall events, the infiltration of water to ground in the treatment pond and the swales would minimise the discharge of stormwater from the additional impervious surfaces to Mill Creek.

It is noted that the highest proportion of contaminant load compared to runoff in the remainder of the storm (by volume) is highest in the initial runoff from the surface (first flush).

The first flush is generally characterised by a peak in some pollutant loads (such as sediments and metals generated from road and urbanised development) immediately prior to the peak in flow volumes. The smaller storms happen most frequently and therefore transport most of the contaminant load. "Best practice for water quality improvement therefore promotes the capture and treatment of the first flush, where practicable, as this is often more practical and cost effective than treating flow volumes from the entire storm event (Auckland City Guideline Document 01 (GD01))."

Treatment Ponds

The first flush is defined as the first 12.5–25mm of direct rainfall runoff from the impervious areas by the Christchurch City Council *"Advanced Analysis"* method provided in the *"Waterways, Wetlands and Drainage Guideline."* The stormwater design allows for hard surface areas to be directed to and captured by the pipe network conveyance system for discharge into the treatment ponds.

Two treatment pond areas (East and West) are proposed to service each of the two sub–catchments of the site. The site is expected to have a low pollutant loading and additionally utilises a combination of first flush pond treatment plus polishing in a swale system (see below sections). Therefore, the minimum pond volume to achieve first flush treatment is based on the first 12.5mm of rainfall runoff from the impervious surfaces as shown in the table below. Additional volume above the 12.5mm rainfall runoff volume requirement will be beneficial. Pond volumes will be confirmed at detailed design stage.

	Western Pond Minimum Volume	Eastern Pond 1 Minimum Volume	Eastern Pond 2 Minimum Volume
First Flush Volume (First 12.5mm from Impervious Areas)	$258m^3$	$516m^3$	$63m^3$

The eastern treatment ponds would be dry ponds, thus allowing the volume to be utilised entirely for the stormwater contaminant runoff. For aesthetic reasons, it is proposed to maintain a wet pond appearance for the western pond, which would be fed from a small inlet pipe from the spring flow in the Mill Creek tributary.

<u>Treatment Swales</u>

Discharge from the treatment ponds is via a small diameter pipe into a treatment swale sized according to GD01. In larger events, the pond bank is designed to convey overflow to discharge directly into the treatment swale.

	Western Swale	Eastern Swale
Water Quality Storm Flow	$0.05m^3/s$	$0.12m^3/s$
Water Quality Storm Swale Water Depth	0.1m	<0.1m
Comment	The lower reach of the spring–fed tributary swale is highly vegetated. In order to avoid use of machinery in the area, the existing cross section and vegetated extent of the flow path has been assessed as being able to provide sufficient treatment for secondary flows.	Both Eastern Ponds 1 and 2 discharge into the same treatment swale, which eventually discharges into Mill Creek.

Erosion protection measures for the pond overflow and swale discharge locations would be developed as part of the detailed design.

6.3 Stormwater Contaminant Loading Assessment

The volumes of traffic generated by the Northbrook Retirement Village would be relatively low (assessed at 1,075 vehicles per day, two-way), and therefore contaminant loadings would also be relatively low.

Table 6.1 below shows the primary contaminants that are anticipated to be present in the stormwater generated from the Northbrook Retirement Village and the associated assessed risk for the development.

Table 6.1: Stormwater Contaminant oading Assessment

Contaminant	Description	Assessed Risk
Suspended Solids (TSS)	▪ TSS is the primary potential contaminant. ▪ Mudtanks in the primary conveyance network would capture a large proportion of the particulate load. ▪ The treatment ponds have been sized to deal to the first flush volumes when the largest proportion of the sediment load happens. ▪ Lastly, the treatment swales provide time for finer particles to settle out of the stormwater flow before reaching the downstream Mill Creek environment.	Low
Hydrocarbons	▪ The primary sources of hydrocarbons are typically generated from vehicle exhausts and engine oil leaks and are generally only considered a concern in high traffic areas (>10,000 vehicles per day) (Auckland Unitary Plan – Technical Report 2013/035, August 2013). ▪ Hydrocarbons have been found to bind to sediments such that removal of total suspended solids is also considered effective at removal of total petroleum hydrocarbons.	Low
Heavy Metals	▪ Lead, zinc, and copper metal contaminants are typically associated with road runoff. ▪ The stormwater that is generated from the building roofs is anticipated to be free of contamination, as modern roofing materials are designed to limit heavy metal loading. ▪ Heavy metal particulates would bind to suspended sediments, which would be treated by removal. ▪ Additionally, vegetation in the swales would absorb dissolved metal particles.	Low
Nutrients (Nitrogen and Phosphorus)	▪ The land is currently used as pasture which has a nutrient loading onto the surrounding land. ▪ As part of the development, the nutrient loading would only be affected by gardening activities which can be managed through the use of materials in the overall maintenance regime. ▪ Nitrogen and Phosphorus are not generated by vehicle activities and therefore not impacted by the increased vehicle traffic. ▪ Furthermore, the treatment swales and ponds provide an opportunity for nutrients to be reabsorbed by plants. ▪ Additional information regarding the effects of phosphorus and nitrogen are found in the Ecology Report (Ryder, May 2020) and Groundwater Assessment (JH Rekker Consulting, May 2020).	Low

Therefore, the overall stormwater contaminant loading to Mill Creek has been assessed as low and the proposed treatment regime has been developed to further reduce risk of contamination.

During the construction period there would be an increased risk of erosion and sedimentation, increased suspended solids load and increased risk of a hydrocarbon spill. An Earthworks Management Plan and Erosion and Sedimentation Management Plan would be developed for the construction period to mitigate potential risks created by construction.

6.4 Pre– versus Post–development Flow Paths Comparison

As part of the design, it is important to consider the pre– and post–development flow discharges and compare flow paths. Figure 6.3 below shows an example of the pre– and post–development flow paths for the critical 100yr ARI – 2hr duration storm. The "critical" duration storm event is the storm duration that results in the maximum peak flow. As part of the design process a range of design storm rainfall hyetographs (rainfall depth versus time graphs) for durations from 0.5hr to 24hr were analysed. From Figure 6.3, the main flow path through the Northbrook Retirement Village site is the Mill Creek tributary flow in both the pre- and post–development scenarios. There are also more minor flows from the surrounding hill catchments to the north and east which discharge into the tributary.

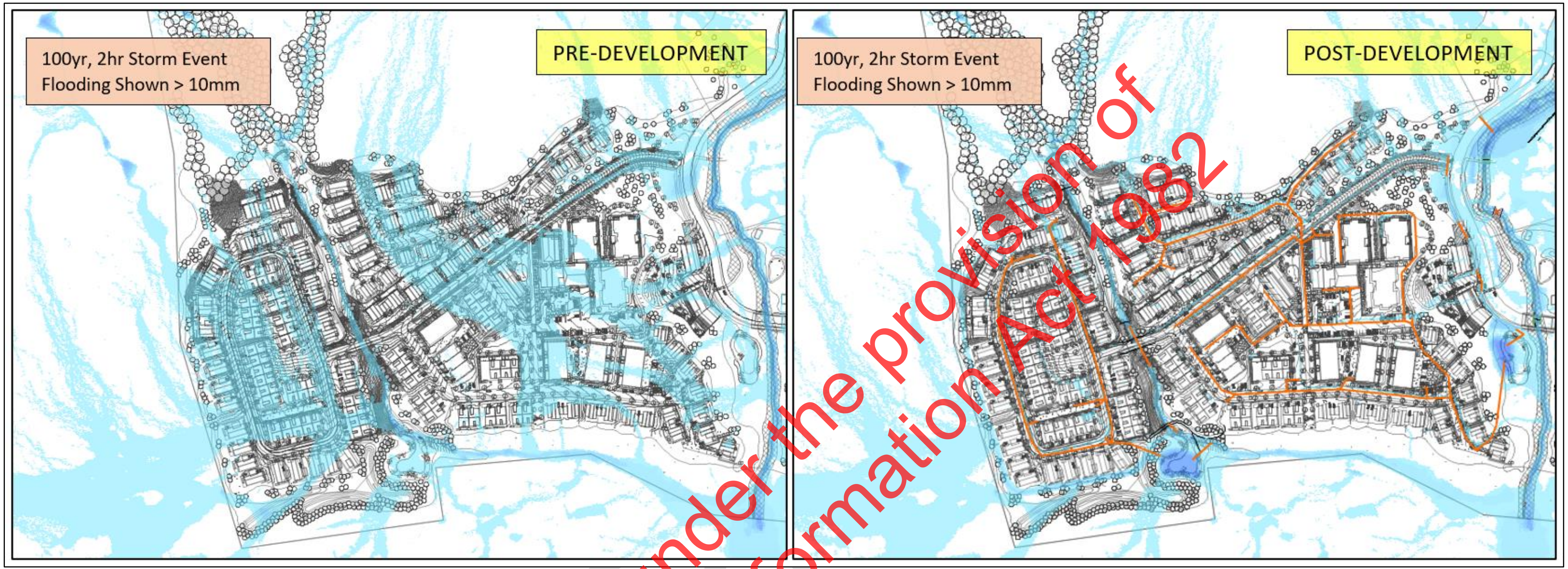

Figure 6.3: Stormwater Flood Flow Pathways

As part of the design, selected points in the model were used to record maximum estimated flood levels along the Mill Creek tributary for the pre– and post–development stormwater runoff conditions. Additionally, the peak pre- and post-development flow have been estimated from the model results at a flow measure line located at 10m south of the site boundary to check the combined effect of the Western Pond discharges to the Mill Creek tributary and Eastern Pond discharges to Mill Creek. The results of this assessment are described below. For the selected point and flow measure locations refer to Figure 6.4.

Additional to the required 20yr and 100yr ARI storms, the 2yr ARI storm event has also been considered to represent a typical moderate flood flow pattern that would be seen more frequently. Refer to the flood level and flow results in Table 6.1 below.

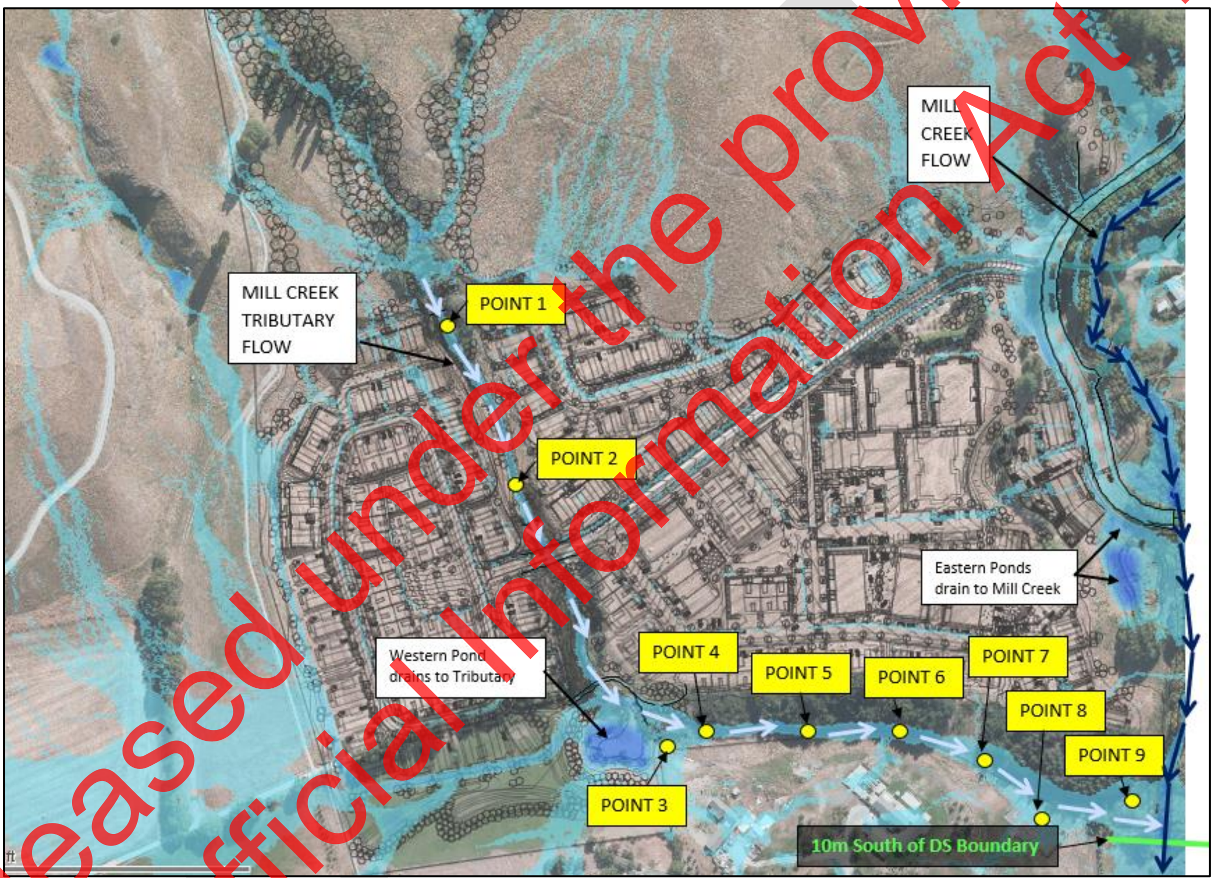

Figure 6.4: Preliminary Selected Result Measurement Locations

The "typical" dry weather flow in the main Mill Creek channel, before the design rainfall storm starts was assumed to be 0.35m^3/s (350 L/s). When a storm event occurs, runoff from the local hillside catchments to the north and west of the Northbrook Retirement Village, the steep hillside catchment along Mill Creek downstream of the waterfall feature at the northern boundary of the Waterfall Park development site, and the retirement village development area generate a small increase in flow to Mill Creek around the time of the peak of the rainfall.

However, this "typical" flow is not the same as the peak flood flow in Mill Creek. The peak flow from the wider Mill Creek catchment above the waterfall feature arrives at the

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Northbrook Retirement Village development site many hours after any local runoff flows. Therefore, the Mill Creek peak flood flows of 10.4m3/s (100yr ARI), 8.5m3/s (20yr ARI), and 4.4m3/s (2yr ARI) occur after the local catchment flows.

In terms of flows, Table 6.1 shows the site runoff flows measured 10m downstream of the site boundary. The location of the flow measure line therefore represents a combination of the flows from the tributary and the main Mill Creek channel. Note that flows are only representative of the local catchment flows, rather than the Mill Creek peak flood flows.

For minor and moderate storm events, in the pre-development condition there is minimal runoff from the local catchments around the retirement village site and along Mill Creek downstream of the waterfall. As a result, for minor and moderate storm events, the assumed typical dry-weather flow in Mill Creek ("typical" flow) persists until the flood flow begins to arrive from the major Mill Creek catchment above the waterfall.

In the post-development situation, the flows are affected by the increased impervious surfaces of the Northbrook Retirement Village, the Access Road, and effect of the Eastern and Western Ponds which have some detention benefit. In the 2yr ARI pre-development situation, there is negligible flow from the local hill catchments. Therefore, the 2yr ARI pre-development flow at the site boundary consists of mainly the "typical" flow in the main Mill Creek channel.

Table 6.1: Preliminary Results – Flood evels and Discharge Flows

	Pre–development	Post–development	Difference (Post Minus Pre)
100yr, 2hr			
Point 1	361.94m	361.94m	0m
Point 2	357.45m	357.43m	–0.02m
Point 3	350.70m	350.69m	–0.01m
Point 4	349.18m	349.11m	–0.07m
Point 5	346.70m	346.67m	–0.03m
Point 6	346.06m	346.05m	–0.01m
Point 7	344.86m	344.80m	–0.06m
Point 8	344.02m	344.00m	–0.02m
Point 9	342.84m	342.81m	–0.03m
Flow at 10m South of DS Boundary (excluding Mill Creek peak flood flow)	6.3m³/s	4.0m³/s	–2.3m³/s
20yr, 6hr			
Point 1	361.80m	361.81m	+0.01m
Point 2	357.26m	357.32m	+0.06m
Point 3	350.50m	350.56m	+0.06m
Point 4	348.96m	348.91m	–0.05m
Point 5	346.52m	346.52m	0m
Point 6	345.88m	345.87m	–0.01m
Point 7	344.72m	344.71m	–0.01m
Point 8	343.94m	343.93m	–0.01m
Point 9	342.76m	342.75m	–0.01m
Flow at 10m South of DS Boundary (excluding Mill Creek peak flood flow)	2.5m³/s	1.7m³/s	–0.8m³/s
2yr, 6hr			
Flow at 10m South of DS Boundary (excluding Mill Creek peak flood flow)	0.39m³/s	0.53m³/s	+0.14m³/s

Overall, the results show flood levels are relatively unchanged in the 20yr and 100yr ARI events and are coupled with decreases in peak discharge flows. These decreases in peak discharge flows are a result of the effect of the detention areas of the Northbrook Eastern and Western Ponds as well as the positioning of the Access Road.

For the 2yr ARI event, there is an increase in peak flow from the pre-development to the post-development scenario due to the increased impervious area compared to minimal runoff from the rolling hill and plateau catchments in the pre–development situation as described above. The impervious areas prevent the absorption of rain in the underlying ground and therefore the runoff from the impervious areas is immediate. There is stormwater detention provided in the Eastern and Western Ponds, but the size of the pond outlets allow a modest unrestricted flow. The small increase in post-development flow of +0.14m^3/s (140 L/s) for the 2yr ARI design storm is considered to have minimal practical effect in Mill Creek downstream of the site compared to the peak Mill Creek flood flow of 4.4m^3/s (4,400 L/s) that would occur hours after the local stormwater flows have discharged.

6.5 perations and Maintenance

It is proposed that the operations and maintenance regime for the retirement village would include routinely monitoring the condition of the cutoff swales, culvert crossings, and treatment ponds and swales. Routine operations surveillance would include inspections of the stormwater and flood management structures after major storm events and annual inspections would monitor the condition and capacity of the swales and assess the deposition of sediment in the ponds against the minimum treatment volume required. Where trigger conditions occur, such as elevated sediment levels, maintenance requirements would be flagged in the course of the inspections and corrective action would be planned and implemented to reinstate the required state.

6.6 Statutory Assessment

6.6.1 Regional Plan: Water for Otago

The tributary to Mill Creek runs through the Northbrook Retirement Village site. As part of the development, it is proposed to construct culvert crossings to allow for vehicle and pedestrian access. It is also proposed to discharge treated stormwater from the development into the Mill Creek tributary via a pond and swale system. The western pond is proposed to be a wet bottom pond fed via the spring flow from the tributary.

The Regional Plan: Water for Otago (RPW) defines a "river" as "a continually or intermittently flowing body of fresh water; and includes a stream and modified watercourse; but does not include any artificial watercourse (including an irrigation canal, water supply race, canal for the supply of water for electricity power generation, and farm drainage canal)." Therefore, although the Mill Creek tributary flows intermittently, it is defined as a "river" under the RPW.

The proposed design would have minimal, if any, effect on the stability of the tributary channel and flood capacity. From a water quality perspective, stormwater discharged from the site would undergo a treatment in sumps, swales and ponds to ensure the water quality in Mill Creek is not affected.

Pursuant to the RPW, consent is required from the Otago Regional Council (ORC) for the following activities:

- Construction of vehicle and pedestrian crossings
  - 1 vehicle crossing – culvert
  - 2 pedestrian and cycle crossings – culvert and boardwalk
- Disturbance of the bed of a river
  - Small weir structures in the channel bed
  - Vehicle crossing construction (Road 01)

- Pedestrian and cycle crossings
- Localised shaping of the channel bed

Additionally, two design elements for the site were assessed as permitted activities under the RPW:

- The discharge of stormwater from the development site to the tributary
- The diversion of water to keep a wet bottom pond fed from the spring flow in the tributary

The activities requiring consent and activities assessed as permitted are discussed in more detail in the below sections.

Vehicle and Pedestrian Crossing Construction

Section 13 of the RPW sets out the rules for land use activities in the bed of a lake or river including construction of bridges and culverts. In relation to the construction of the vehicle and pedestrian crossings as part of the Northbrook Retirement Village, Rule 13.2.1.7 of the RPW states the following (comment is provided on the compliance with each condition):

Rule 13.2.1.7:
The erection or placement of any single span bridge including for pipes over the bed of a lake or river, or any Regionally Significant Wetland, is a permitted activity, providing:

Rule 13.2.1. Conditions	Compliance with Conditions
(a) *The bridge or its erection or placement does not cause any flooding, nor cause any erosion of the bed or banks of the lake or river, or Regionally Significant Wetland, or property damage; and*	Compliant. The pedestrian and vehicle crossings have been designed to ensure that they do not cause flooding, erosion, or property damage. Additionally, the expected velocities in the tributary are expected to be low (<1m/s for the 100yr ARI event).
(b) *No more than 20 metres of bridge occurs on any 250 metre stretch of any lake or river; and*	Compliant. The vehicle and pedestrian crossings are less than 20m.
(c) *There is no reduction in the flood conveyance of the lake, river or Regionally Significant Wetland; and*	Compliant. The crossings are to be designed to ensure there is no reduction in flood conveyance for a 100yr ARI event.
(d) *The bridge soffit is no lower than the top of the higher river bank; and*	Non–Compliant. The pedestrian crossings would be below the top of bank level.
(e) *The bridge and its abutments are secured against bed erosion, flood water and debris loading; and*	Compliant. The crossings would be constructed to be secured against erosion, flood water, and debris loading. The pedestrian crossings are to be overtopped in large flood events. The vehicle crossing is to be situated above the 100yr ARI flood level. Should the vehicle crossing culvert become blocked, flood levels would build up on the upstream side until the top

Rule 13.2.1. Conditions	Compliance with Conditions
	of road height is reached, at which point water would flow over the road.
(f) Where the bridge is intended for use by stock, measures are taken to avoid animal waste entering the lake, river or Regionally Significant Wetland; and	Compliant. The crossings are not intended to be used by stock.
(g) If the bridge is situated over or on public land, then public access over the public land is maintained.	Compliant. The crossings are not situated on or over public land.

The development proposal does not comply with regard to Rule 13.2.1.7 (d), and therefore consent is required for a discretionary activity.

Any boardwalks would be built as a permitted activity under Rule 13.2.1.7A and would be designed to not cause flooding or erosion.

Disturbance of the River ed

From Section 8.2 of the RPW, the issues to be addressed specific to "disturbance" of the bed and margins of a "river," being the Mill Creek tributary are as follows:

Changes in the nature of the flow of water and sediment caused by activities in, on, under or over the bed or margin of a lake or river, can adversely affect:

a. The stability and function of existing structures;

b. The bedform of the lake or river;

c. Bed and bank stability; and

d. Flood carrying capacity.

In relation to the implementation of small weir structures to create ponding areas in the Mill Creek tributary, the design meets the following requirements:

1. There must be no adverse effects due to flood flows on property downstream and no adverse effects on adjacent land as a result of the proposed works.
2. The proposed mitigation measures are based on observations of the current waterway flow regime and are therefore consistent with the waterway's future use.
3. The waterway is designed to confine the design flood flows that could affect buildings proposed on the site.

Rules 13.5.1.1 and 13.5.1.3 of the RPW refer to disturbance of the bed of a river. The proposed works will not comply as “the time necessary to carry out and complete the whole of the work within the wetted bed of the lake or river” is estimated to exceed 10 hours in duration, and therefore a resource consent is required. The other conditions in Rules 13.5.1.1 and 13.5.1.3 including limiting sedimentation and erosion during construction would be included in the earthworks management plan and erosion and sediment control plan prepared prior to construction.

Stormwater Discharge

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 6.2 below lists each of these conditions and specifies how compliance with these conditions is achieved.

Table 6.2: Compliance with Rule 12. .1.8:

Rule 12. .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) The first flush interception ponds and treatment swales provided for the removal of suspended solids.
(b) The discharge does not contain any human sewage; and	The stormwater is predominantly from a natural grassed catchment that would include road and roof runoff and therefore 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*	The stormwater discharge would not give rise to these effects after reasonable mixing.

Rule 12. .1.8 Conditions	Compliance with Conditions
(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 conclusion of the stormwater discharge assessment of effects, see below, demonstrates compliance with the permitted activity rules for RPW.

The objective for stormwater management and effects mitigation planning has been to collect stormwater that falls on roofs, roads, or travels towards the road from open space areas and direct runoff to treatment ponds and swales for the removal of potential contaminants and discharge the collected stormwater to Mill Creek, in compliance with ORC rules and the QLDC COP 2020.

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

Diversion of Water

The diversion of water from the spring–fed tributary is considered to be a permitted activity under Rule 12.3.2.1 of the RPW. In this case, the estimated size of the catchment is less than 50ha and the dam/diversion would be a low level weir with a small diameter culvert to take a portion of the natural flow and discharge it into the proposed stormwater pond, from which the water is again discharged to the tributary approximately 35m downstream of the take location.

6.6.2 QLDC Land Development and Subdivision Code Practice

The stormwater infrastructure has been designed to meet the requirements of the QLDC COP 2020. The QLDC COP 2020 contains requirements for mitigating the adverse stormwater effects due to land development for urban use.

Design Capacity

The design capacity required for the primary and secondary flow paths is specified in Clause 4.5.3.2 of the QLDC COP 2020. The primary flow path is to be designed for a 20yr ARI without surface flooding. Where there is no secondary flow path, the primary system is to be designed for a 100yr ARI worst case flow without surface flooding.

The reticulation network is designed to confine and convey stormwater for a 20yr ARI event without significant surface flooding. The roads have capacity to convey flow in excess of the 20yr ARI event up to the 100yr ARI flow as a secondary flow path.

Downstream Flow Mitigation

To prevent significant adverse effects, the stormwater management system is required to address flows up to the 100yr ARI storm frequency to pre–development flow rates at the site boundary (Clauses 4.2.4 and 4.2.7).

The estimated peak flow at the southern boundary of the site is less in the 20yr and 100yr ARI post–development scenario than the pre–development scenario.

The 2yr ARI event is more representative of the everyday type flows that would occur. For the 2yr ARI event, there is a small increase in flow from the Mill Creek tributary (about 140L/s) due to the higher impervious area due to development. The existing tributary channel is well vegetated and portions upstream of the pond are proposed to be more heavily vegetated as part of the proposed development landscaping plans. Therefore, no additional erosion risk is assessed as a likely outcome. It is assessed that the increase in flow will have minimal, if any, effect. Note that the peak flood flow for Mill Creek is in the order of 4,400L/s and will occur several hours after the local stormwater flows. An increase of 140L/s for the stormwater flows is unlikely to adversely impact Mill Creek or the downstream environment.

Stormwater Quality

Clause 4.2.8 of the QLDC COP 2020 specifies that stormwater treatment devices can be required to avoid adverse effects on downstream water quality. The focus on the management to preserve receiving water quality is becoming an increasingly important focus for QLDC. An adequate treatment system has been proposed with the focus on intercepting suspended solids in order to reduce a contamination risk to Mill Creek.

Building Freeboard Levels

The freeboard requirements from the QLDC COP have been adopted as the minimum freeboard specification for the retirement village. Clause 4.3.5.2 is copied below:

> The minimum freeboard height additional to the computed top water flood level of the 1% AEP design storm should be as follows or as specified in the district or regional plan:

Freeboard	Minimum height
Habitable dwellings (including attached garages)	0.5 m
Commercial and industrial buildings	0.3 m
Non-habitable residential buildings and detached garages	0.2 m

> The minimum freeboard shall be measured from the top water level to the building platform level or the underside of the floor joists or underside of the floor slab, whichever is applicable.

Please also refer to Figure 6.5 below which summarises the freeboard requirements for the site as per the QLDC COP which are met as part of the design.

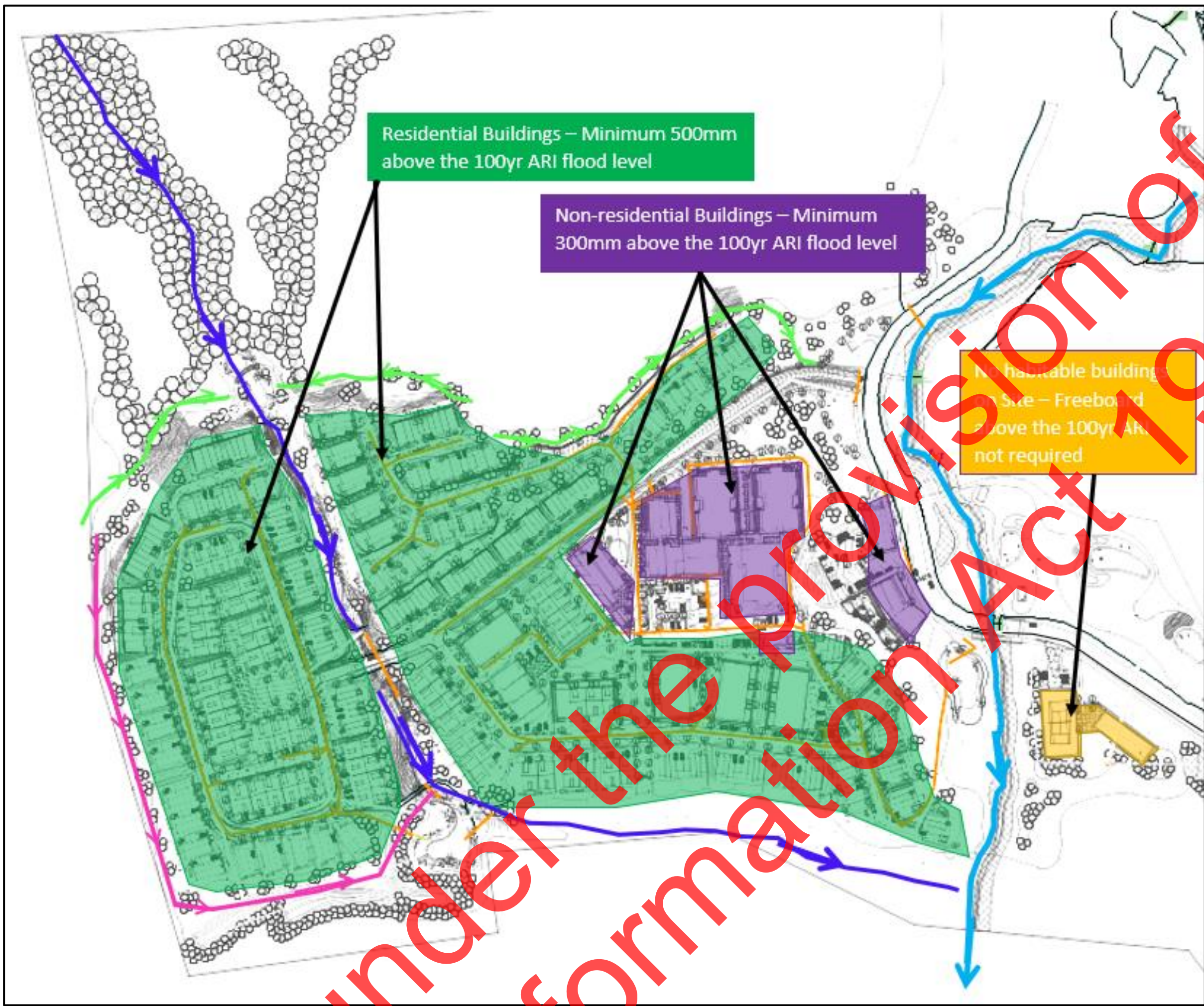

Figure 6.5: Freeboard Re uirements

6. Recreation Area Flood Assessment

The development proposal also includes an allowance for mounding and a recreation area located on the eastern bank of Mill Creek. There are no buildings proposed for this area (only playing fields) and therefore the minimum freeboard requirements in the QLDC COP 2020 do not apply. Based on an initial assessment of the Mill Creek flow patterns, the tennis court and bowling green area is situated above the 100yr ARI flood level.

Golf holes are also proposed on the northern side of the Waterfall Park Access Road. These involve minor reshaping (cut-fill balanced) of the existing ground levels only and will not affect any flow paths or detention volume allowances.

6.8 Mobility Scooter Parking and us Stop Areas Flood Conveyance

As part of the Waterfall Park Access Road design, stormwater runoff from the road and flood waters in the vicinity of the mobility scooter parking and bus stop area are collected and conveyed via a swale system located on the north and south sides of the Access Road. The mobility scooter parking and bus stop will not affect the flood conveyance of these swales and any stormwater runoff from increased impervious area from the parking area or bus stop to the surrounding grassed areas would be negligible. Figure 6.6 below shows how the mobility scooter parking area does not affect the Access Road swale system. Refer to Paterson Pitts drawings for additional information.

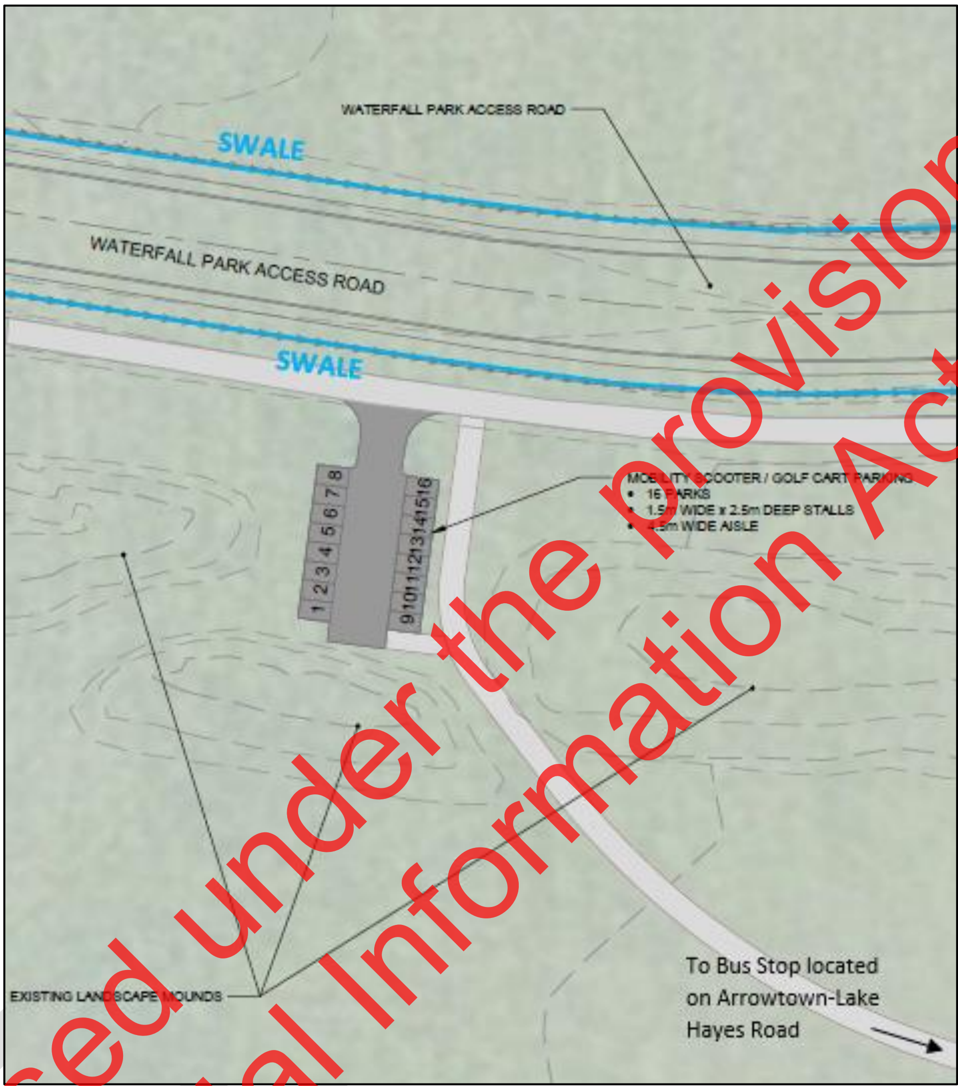

Figure 6.6: Mobility Scooter Parking Area Flood Conveyance

APPENDI 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

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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: 1:5000
Designed by:		A3	
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

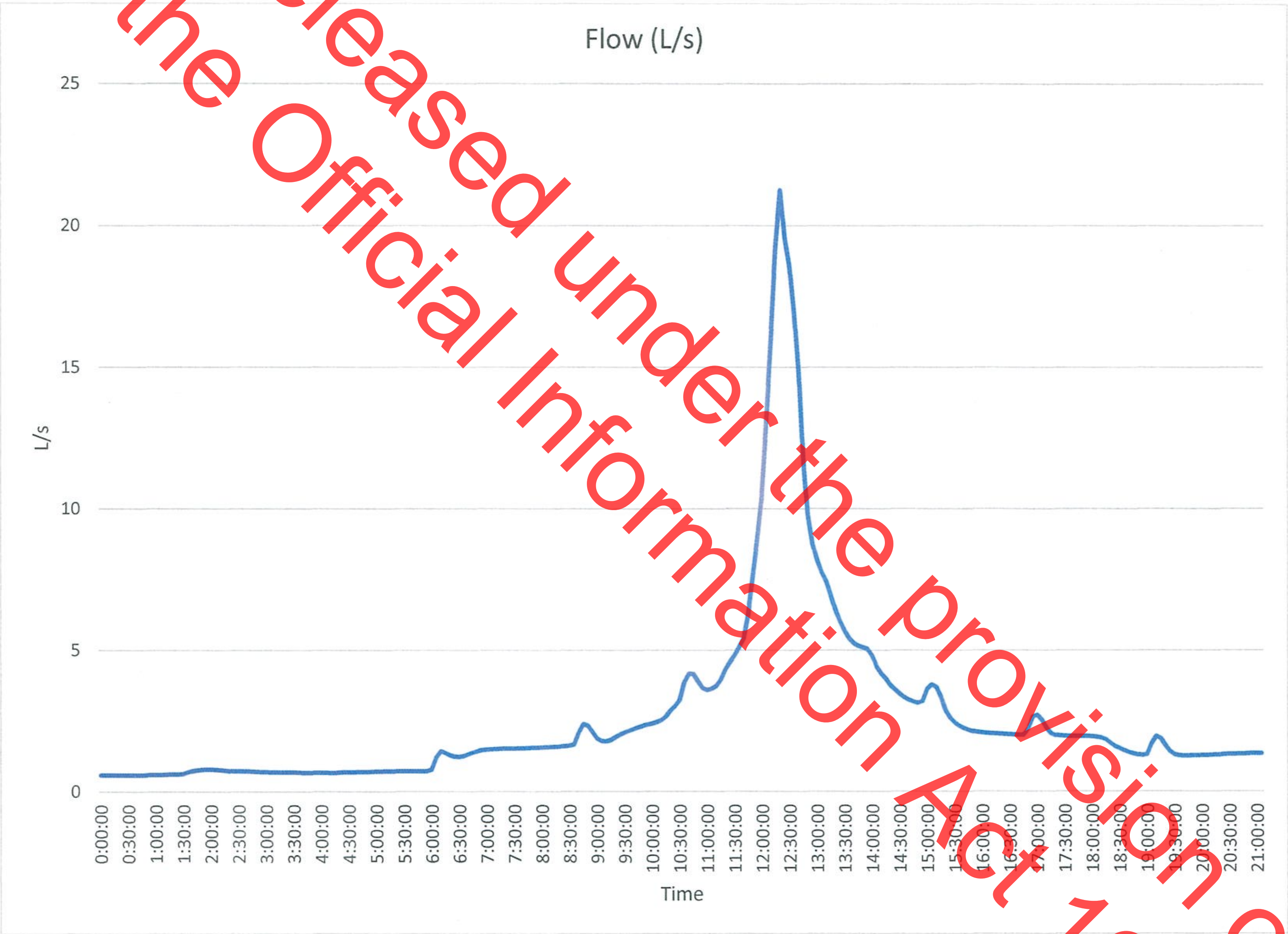


Figure 5: Current peak wet weather inflow to Lake Hayes #2 PS

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

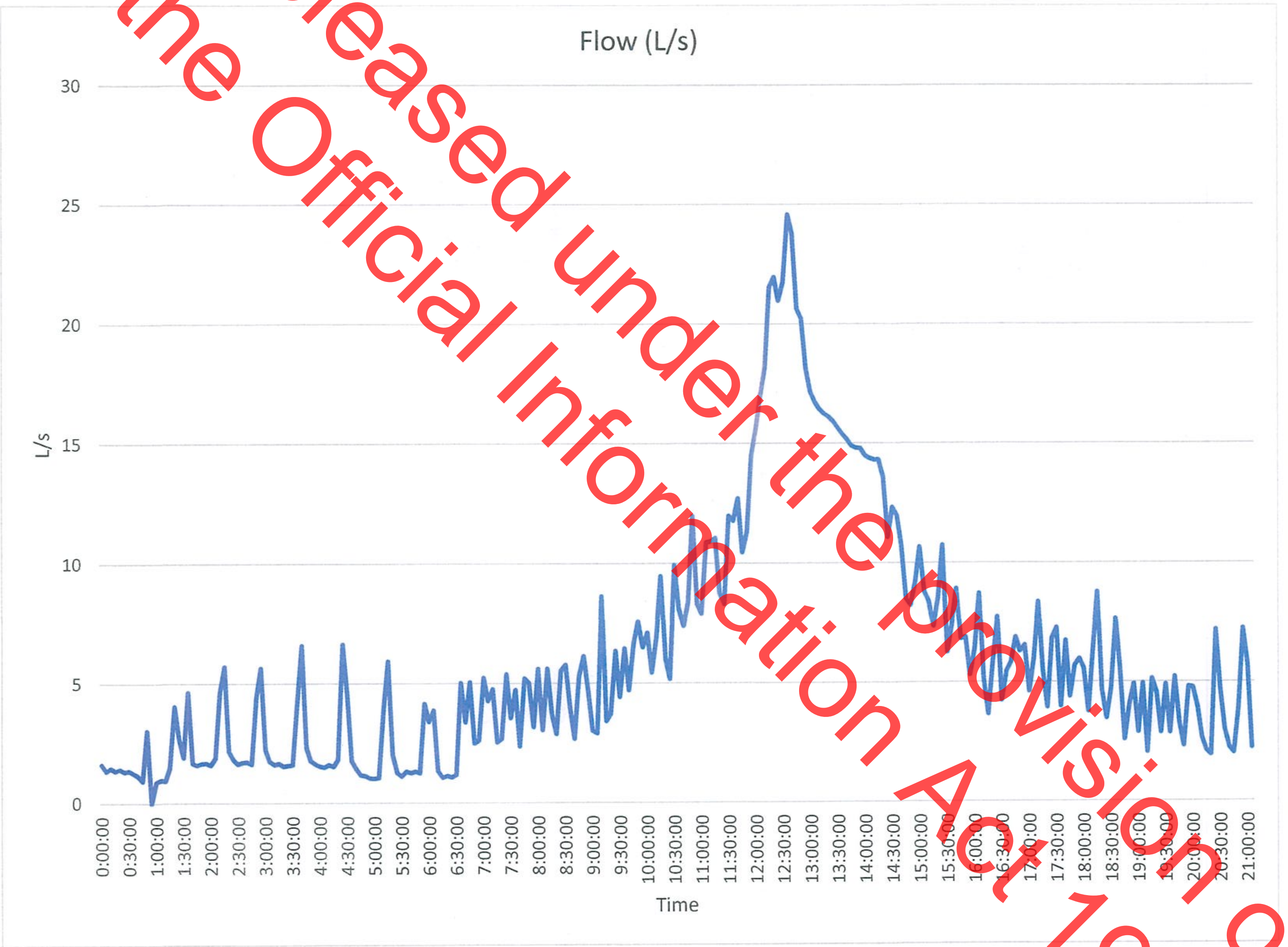


Figure 8: Current peak wet weather inflow to Bendemeer PS

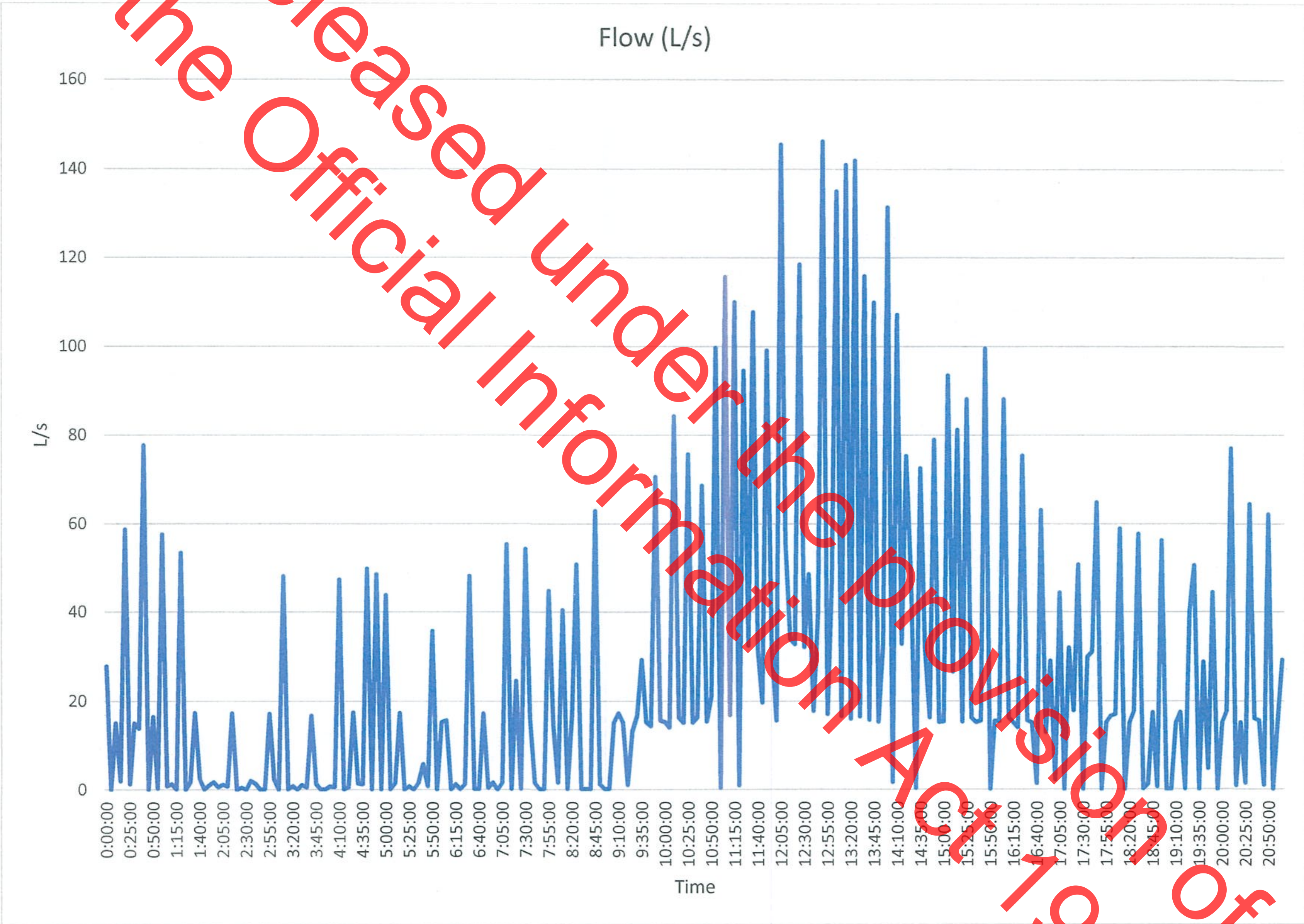


Figure 9: Peak wet weather inflow to Bendemeer PS using 2028 Growth Model (No Waterfall Park Development)

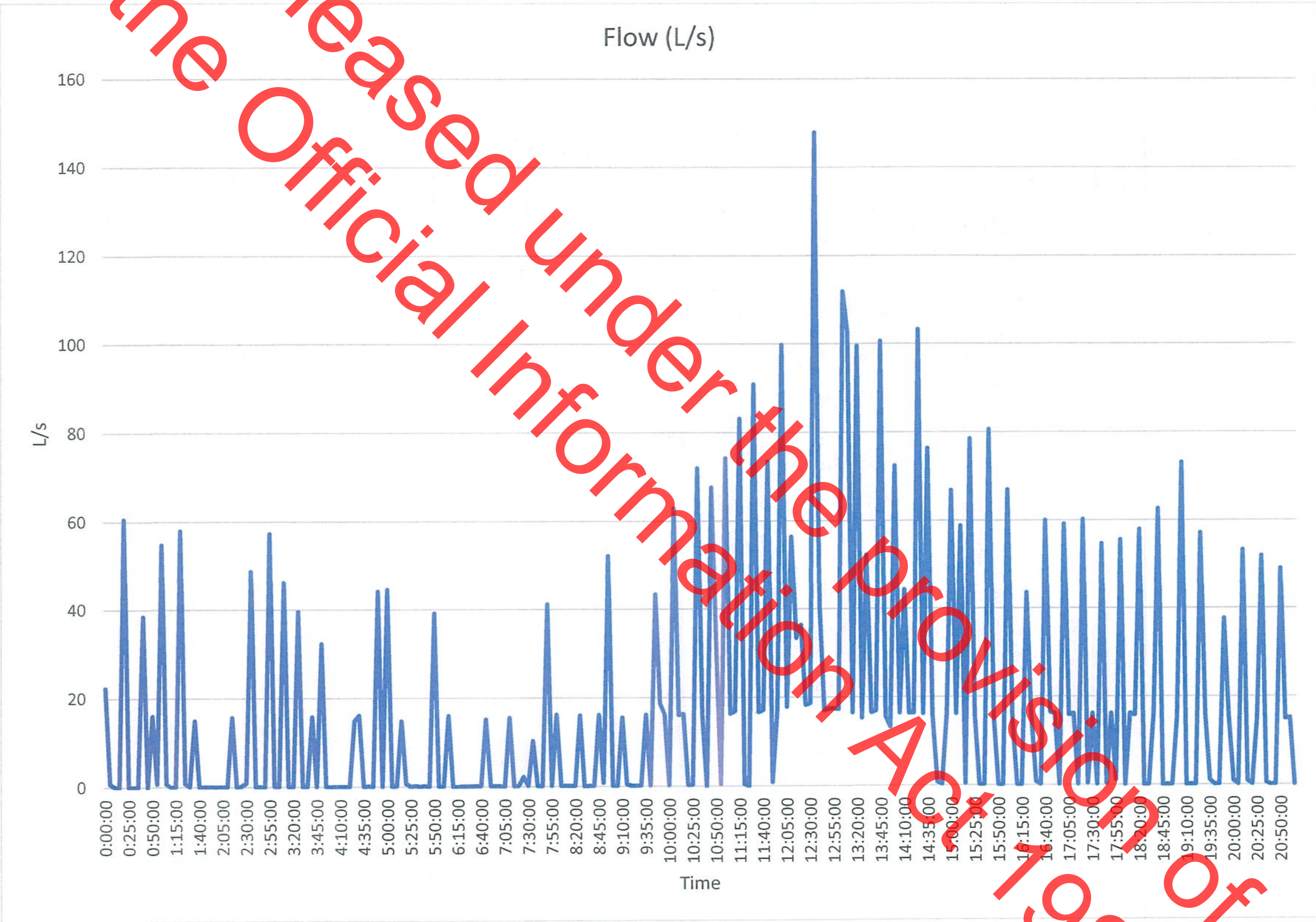
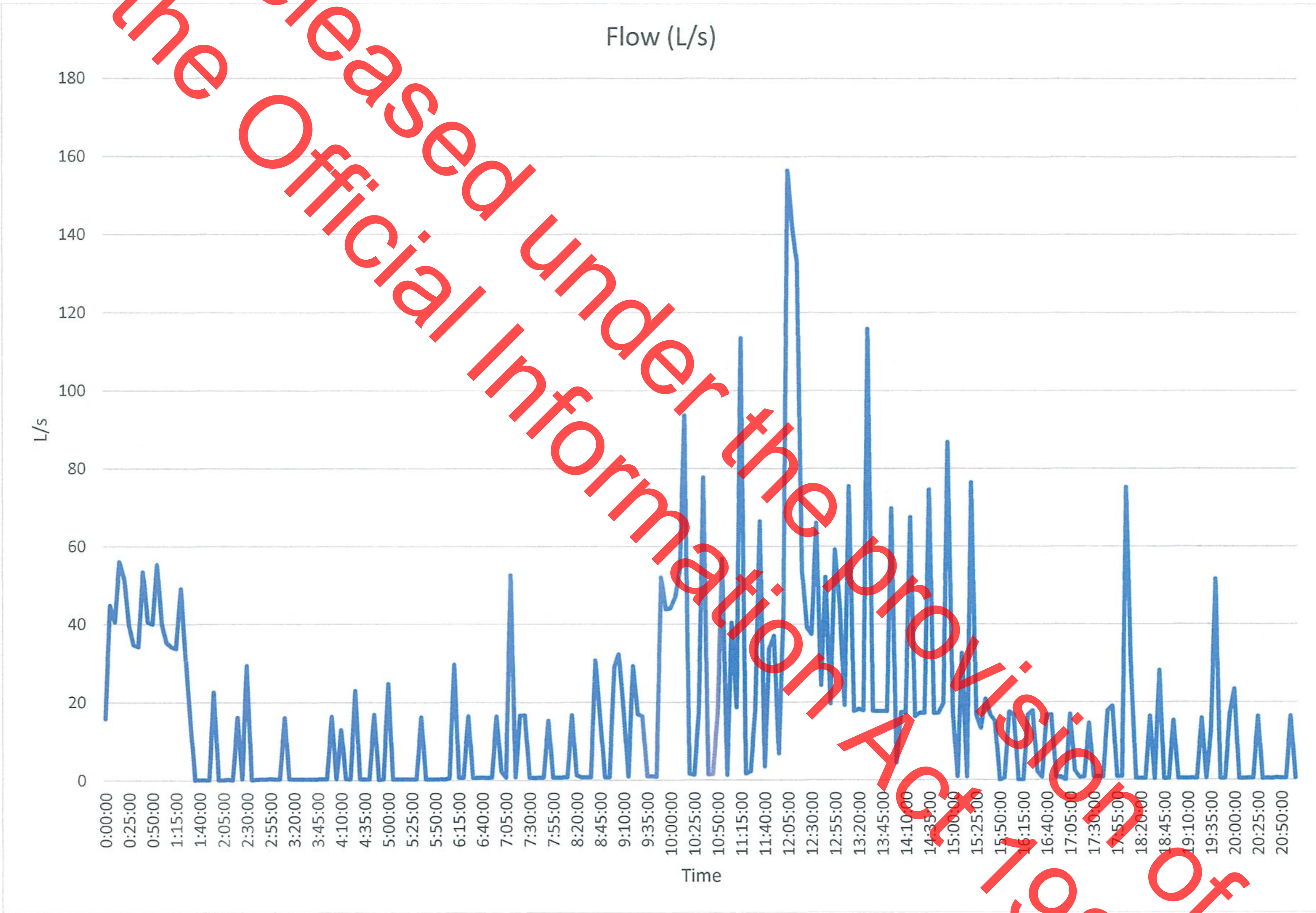


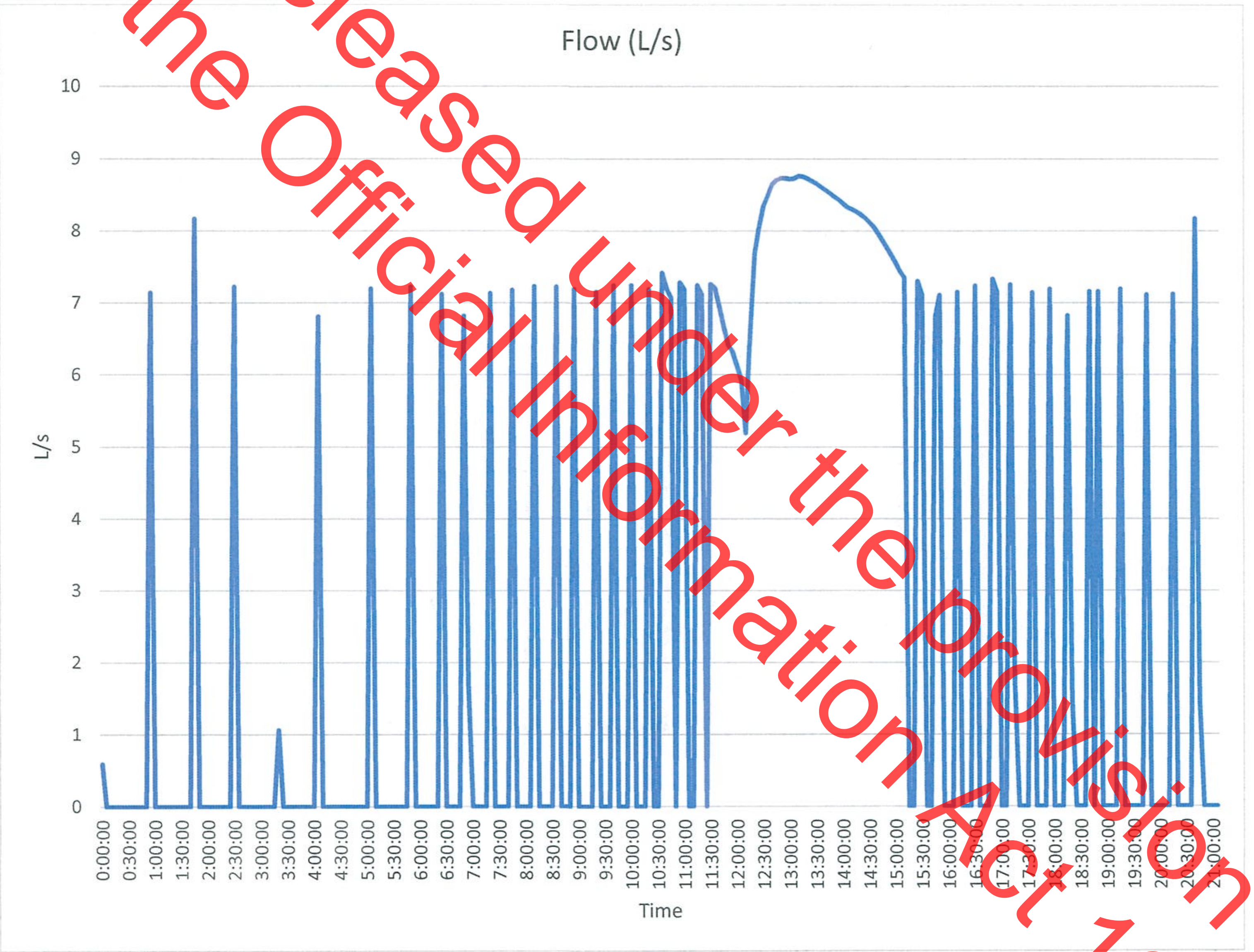
Figure 10: Peak wet weather inflow to Bendemeer PS using 2028 Growth Model, including Waterfall Park Development



Appendix C

Outflows from the Lake Hayes Pump Stations

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



Beca

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

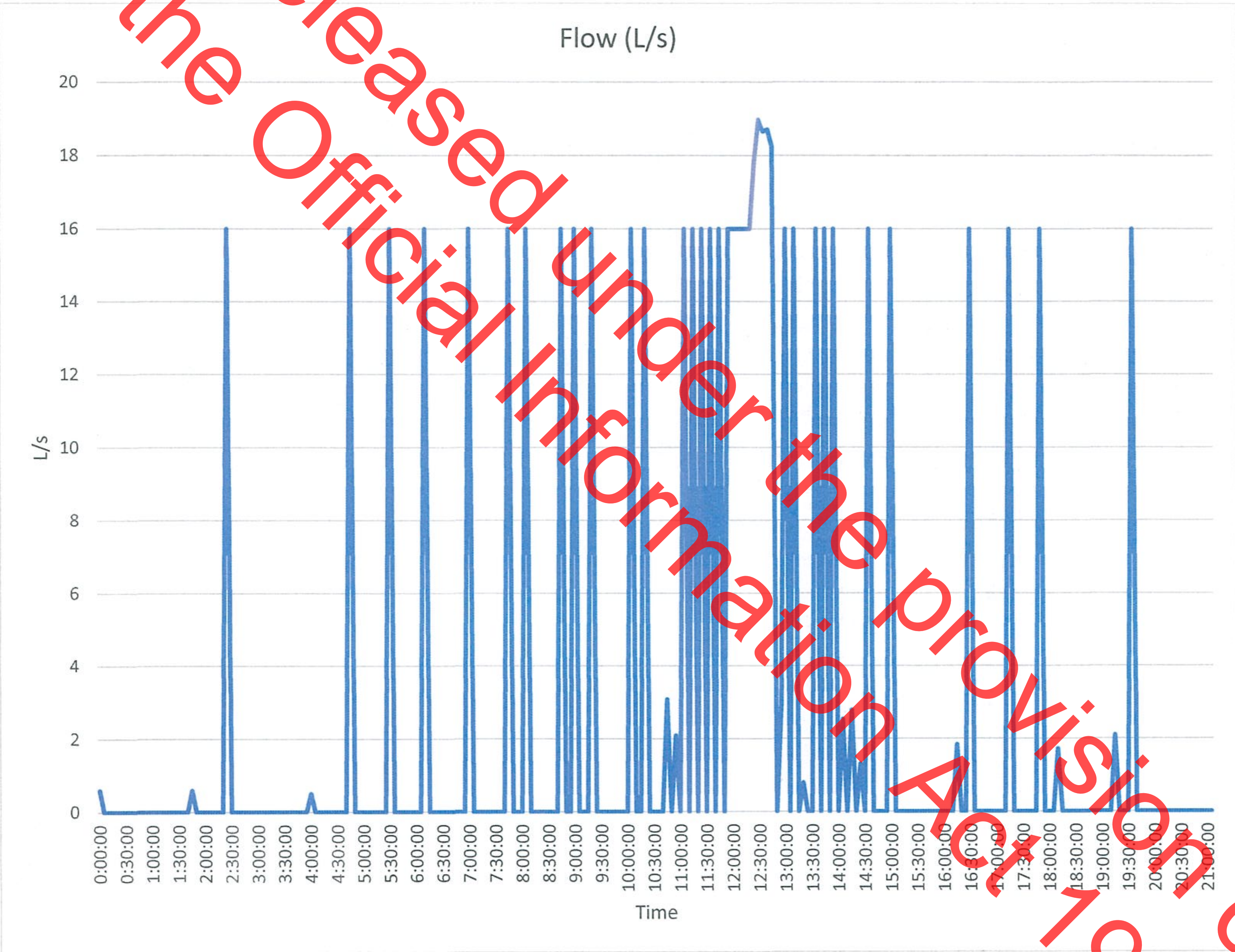


Figure 14: Current peak wet weather outflow from Lake Hayes #2 PS

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

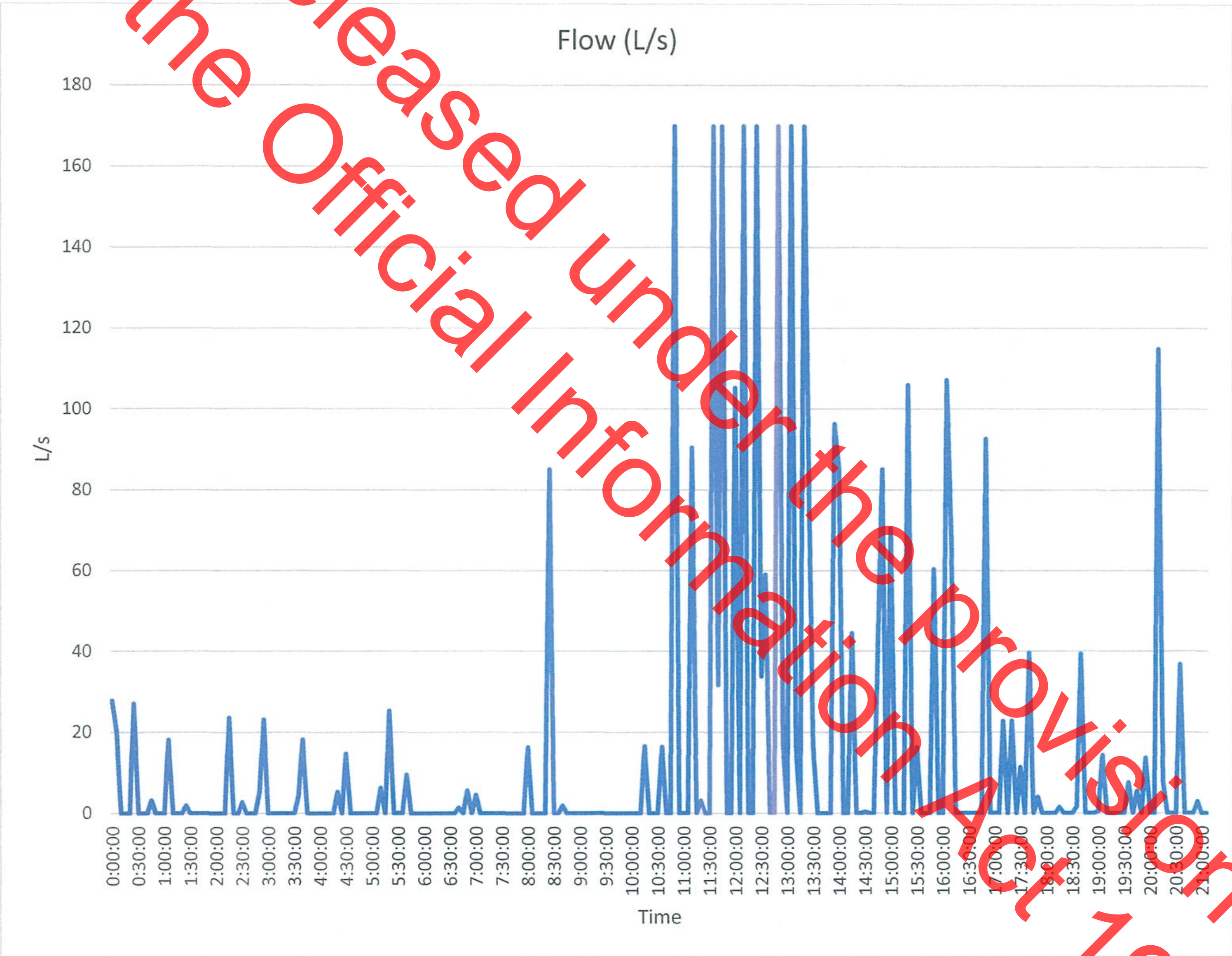


Figure 18: Peak wet weather outflow from Bendemeer PS using 2028 Growth Model (No Waterfall Park Development)

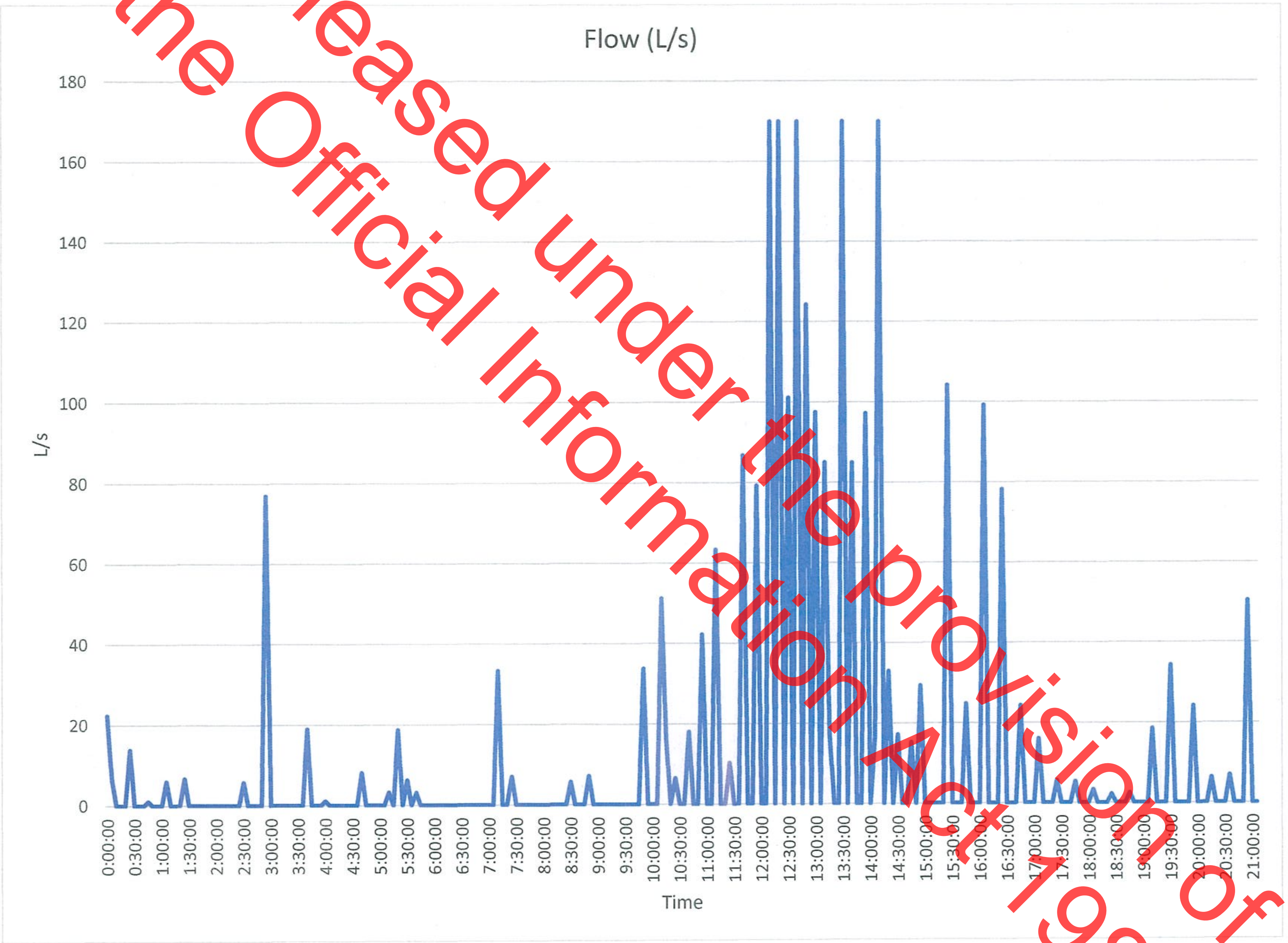
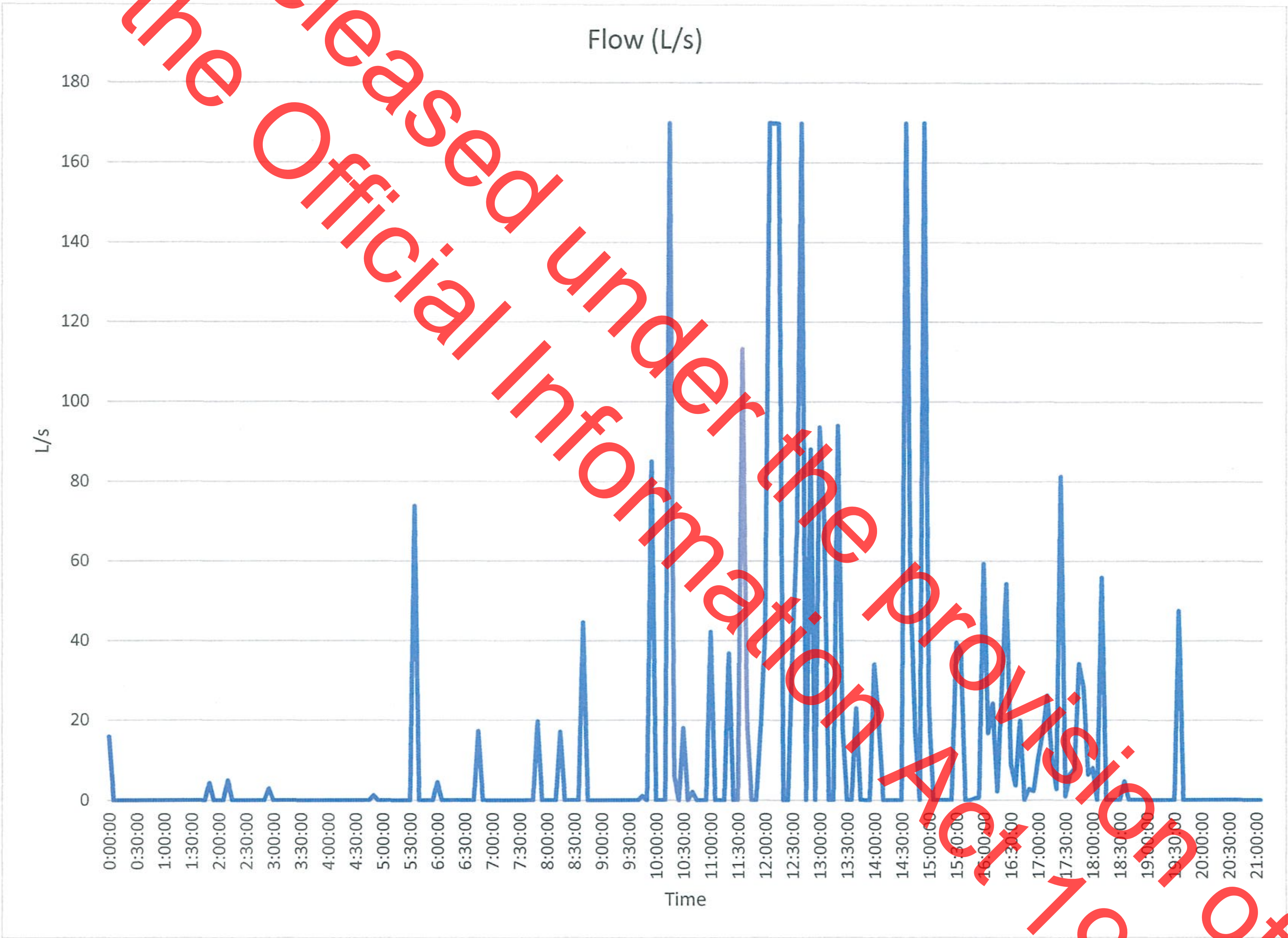


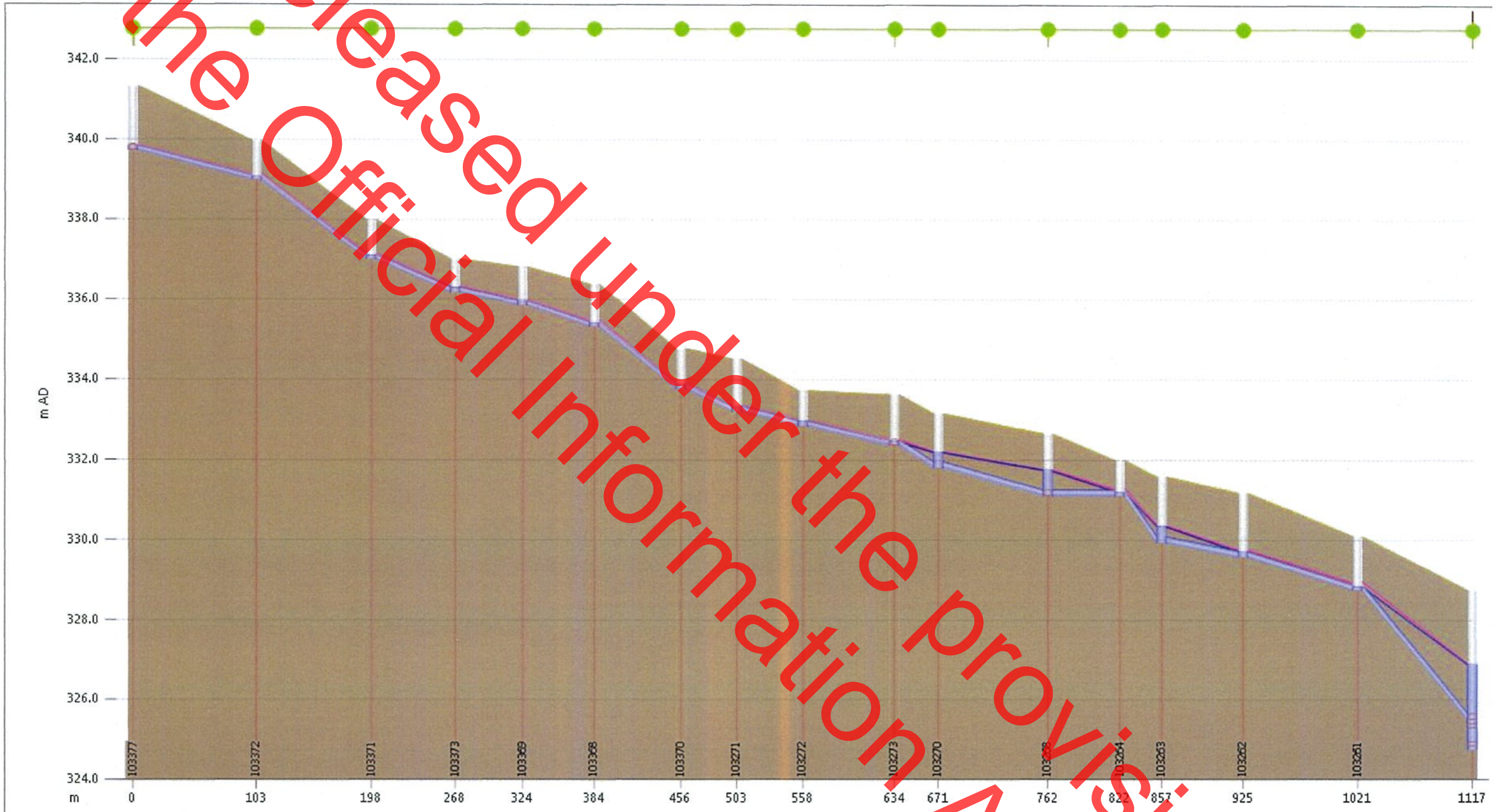
Figure 19: Peak wet weather outflow from Bendemeer PS using 2028 Growth Model, and Including Waterfall Park Development



Appendix D

Long Sections

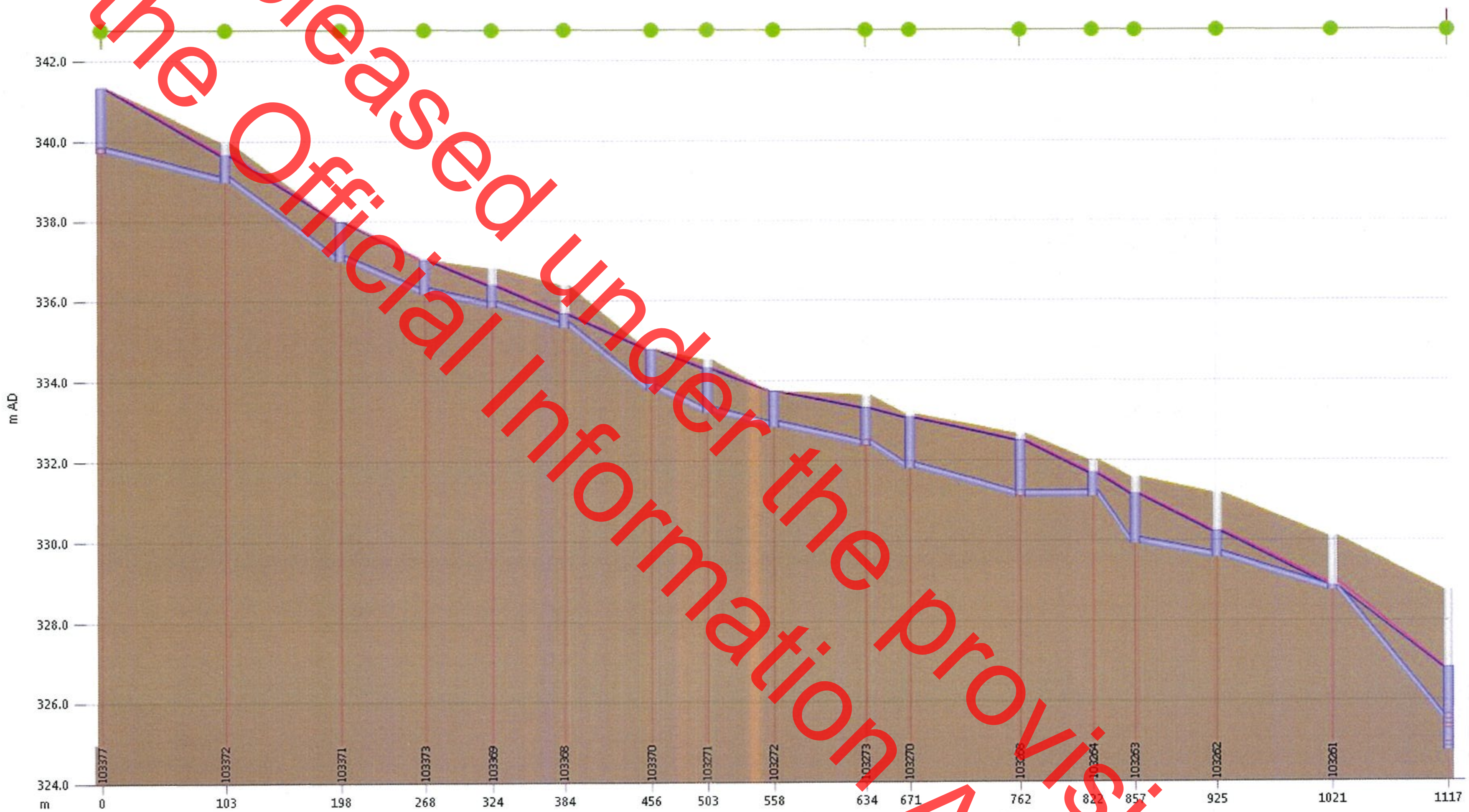
Figure 20: Long Section Upstream of Lake Hayes #1 PS without Development



Link	width (mm)	us inv (m AD)	ds inv (m AD)
103377.1	150	339.720	338.990
103372.1	150	338.990	337.010
103371.1	150	337.010	336.190
103373.1	150	336.190	335.860
103369.1	150	335.860	335.340
103368.1	150	335.340	333.780
-	150	333.780	333.220
103271.1	150	333.220	332.850
103272.1	150	332.850	332.370
-	150	-	-
103270.1	150	331.810	331.100
103268.1	150	331.100	331.100
-	150	-	-
103263.1	150	329.930	329.580
103262.1	150	329.580	328.760
103261.1	150	328.760	325.500

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

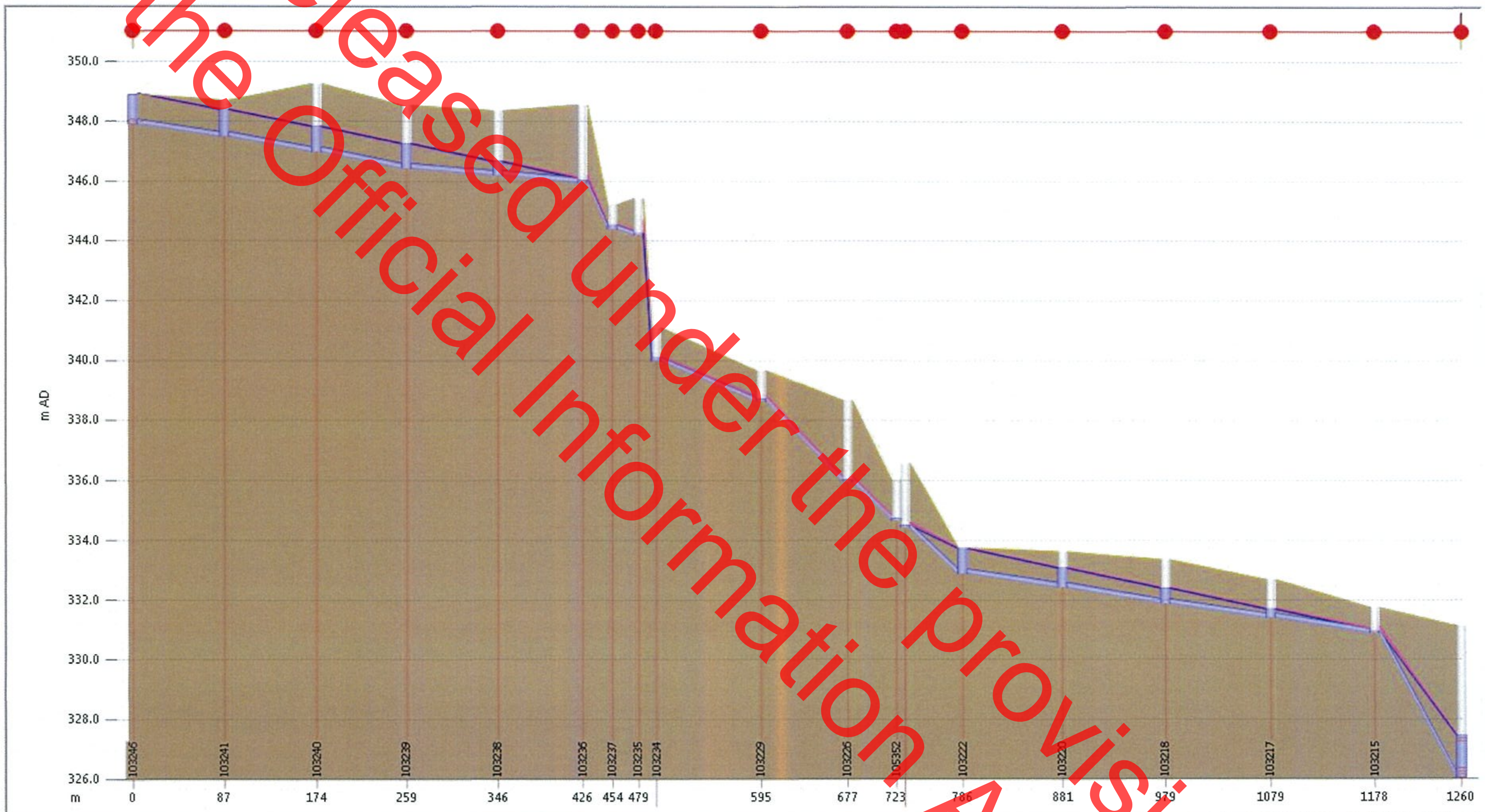
Figure 21: Long Section Upstream of Lake Hayes #1 PS with Maximum Development



Link	103377.1	103372.1	103371.1	103373.1	103369.1	103368.1	-	103271.1	103272.1	-	103270.1	103268.1	-	103263.1	103262.1	103261.1
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

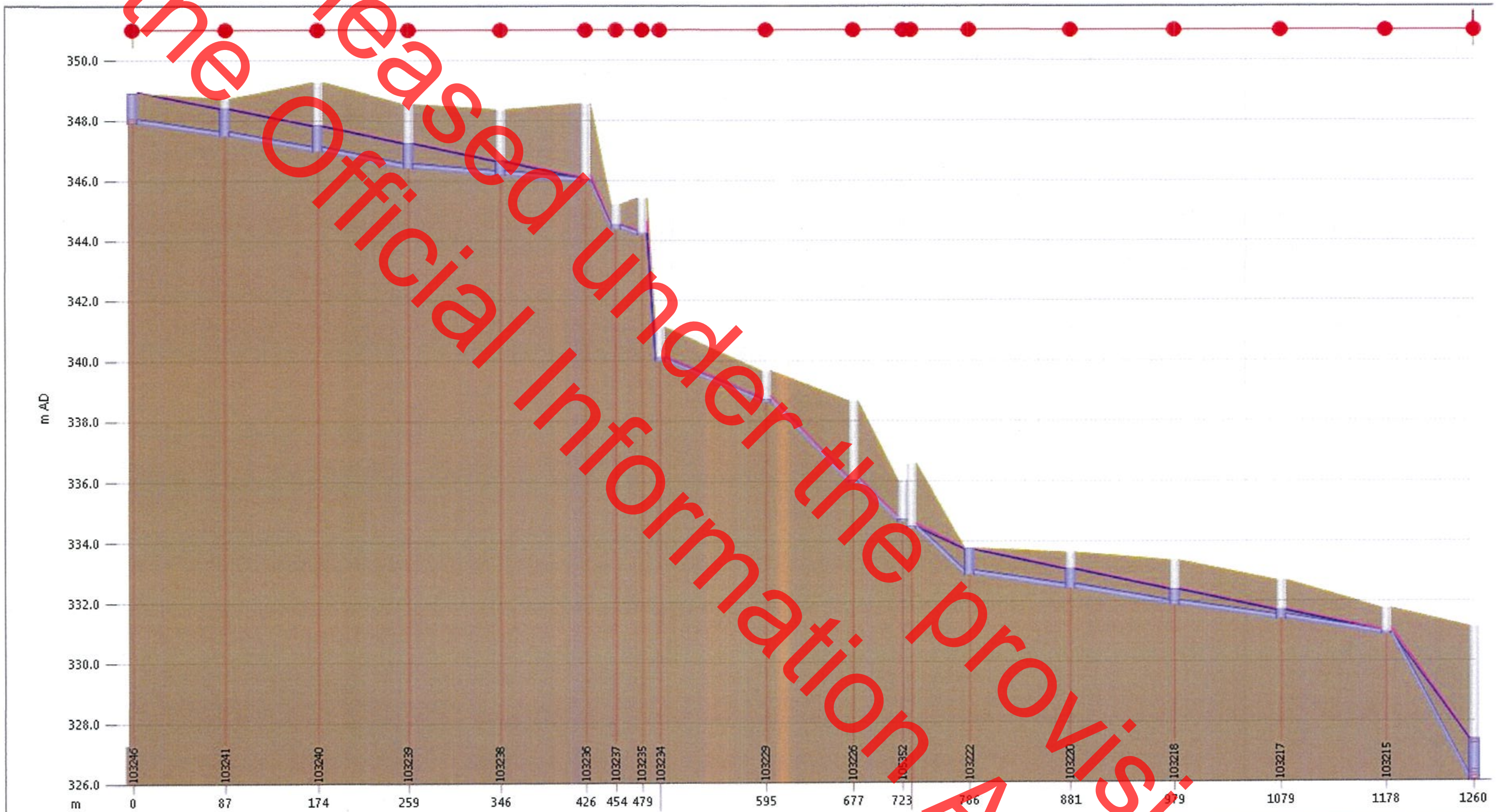


Link	width (mm)	us inv (m AD)	ds inv (m AD)
103246.1	150	347.910	347.500
103241.1	150	347.500	346.990
103240.1	150	346.990	346.440
103239.1	150	346.440	346.200
103238.1	150	346.200	345.976
-	150	-	-
-	150	-	-
-	-	-	-
103234.1	150	340.000	338.650
103229.1	150	338.650	335.950
-	150	-	-
105353.1	150	334.420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

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

Beca

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	-

Beca

APPENDI

Wastewater Modelling Report Addendum

WATERFALL PARK DEVELOPMENT: WASTEWATER NETWORK ASSESSMENT

To:	Richard Powell	Queenstown Lakes District Council (QLDC)
Distribution:	Jayne Richards	Fluent Solutions (FS)
From:	Brian Robinson; Rebecca Ellmers (HAL)	
Subject:	Waterfall Park Development – Wastewater Network Assessment	
Date:	16 January 2019	

1 Introduction

1.1 Objective

The objective of this study is to utilise the existing hydraulic model (Wakatipu Wastewater Model with HAL updates, 2018) of the Queenstown, Arrowtown and Lake Hayes wastewater network to assess the impact of the proposed Waterfall Park development on the wastewater network.

1.2 Background

The Waterfall Park development proposal seeks to discharge a maximum flow rate of 23.4 l/s to the existing network. The initial hydraulic modelling carried out by BECA (Waterfall Park Development Wastewater Modelling, 2018) considered a number of private pump station scenarios at various connection points to the existing network. The development consultant has since requested further assessment of the Waterfall Park development impact.

2 Waterfall Park Development

2.1 Overview

The Waterfall Park development seeks to discharge a maximum PWWF of 23.4 l/s and has considered two potential network connection points as summarized below:

1. Connection to the existing local 150mm network to the south discharging to Lake Hayes #1 Pump Station, and eventually to the Arrowtown-Lake Hayes Pump Station
2. Connection to the existing transmission 300mm gravity/pressure main connecting Norfolk Street Pump Station to the Arrowtown-Lake Hayes Pump Station

The connection point to the existing 150mm network to the south was shown in the assessment undertaken by Beca to result in overflows from the local network upstream of the Lake Hayes #1 pump station. This assessment has focused on the connection point to the existing 300mm gravity/pressure main with a proposed pump rate of 23.4 l/s (i.e. matching expected design flows for the full development.

The location of the development and proposed connection points is shown in Figure 1 below.

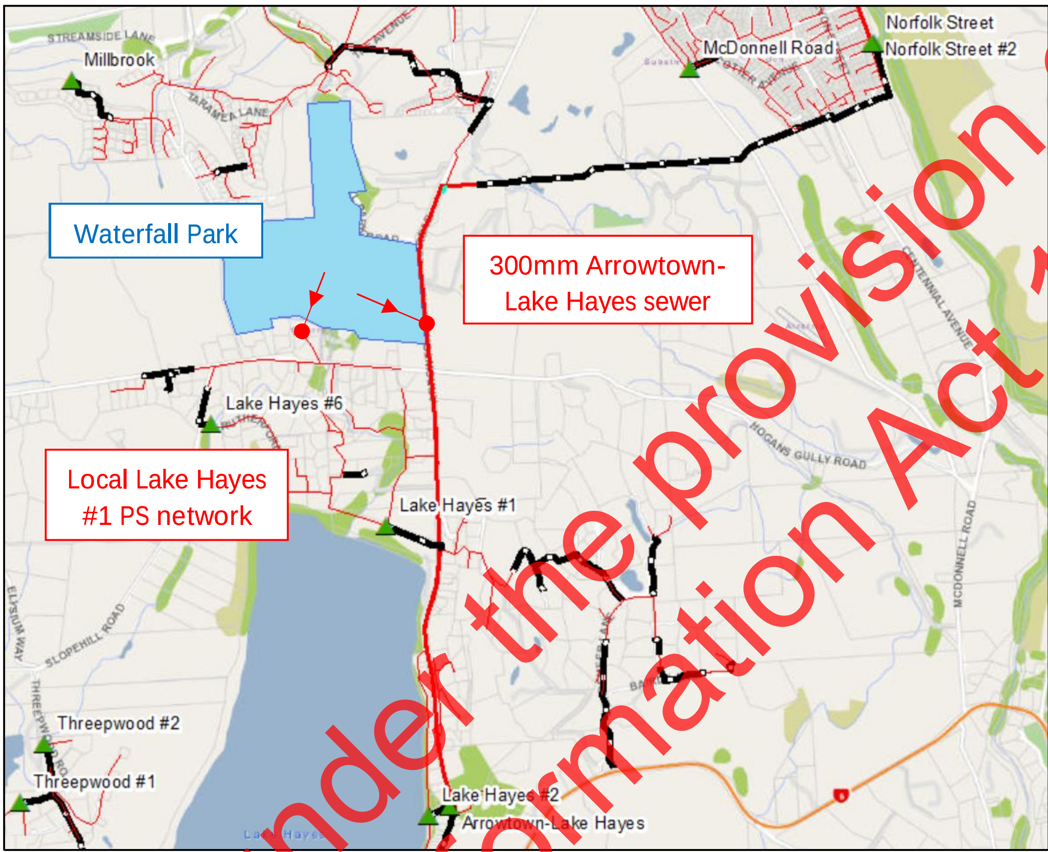


Figure 1: Waterfall Park Development Wastewater Connection

3 Waterfall Park Development Impact

3.1 Proposed Modelling Scenarios

The development consultant Fluent Solutions have since requested further assessment of the Waterfall Park development impact. The initial hydraulic modelling carried out by BECA (Waterfall Park Development Wastewater Modelling, 2018) considered a private pump station with storage and off-peak pumping (assumed to lessen the effect of the development load on the network), with an arbitrary pumped rate of 15 l/s. Fluent Solutions have requested modelling of the maximum proposed development discharge of 23.4 l/s at the Arrowtown-Lake Hayes 300mm connection point (identified as Scenario 3 in the BECA report).

3.2 Scenario 3: Waterfall Park (23.4 l/s) to Arrowtown-Lake Hayes 300mm line

The Wakatipu wastewater model (with 2018 HAL updates included update of pump station capacities) was run under the current (2015) scenario, with and without the proposed Waterfall Park development. The network was assessed against a 5-year ARI design storm to understand the system performance. As shown in the Figure 2 long-section below, the existing network has sufficient capacity in the 300mm Arrowtown-Lake Hayes Wastewater line, discharging to the Arrowtown-Lake Hayes Pump Station.

Figure 2: Existing (2015) Long Section (300mm Arrowtown WW line) – 5 year ARI design storm

The additional peak wet weather flows of 23.4 l/s from the Waterfall Park development were added in to the model, with connection to the 300mm Arrowtown-Lake Hayes wastewater line. As shown in the Figure 3 long-section below, the post-development network has adequate capacity within the 300mm line to receive the full peak wet weather flows from the proposed development.

Figure 3: Post Development (2015) Long Section (300mm Arrowtown WW line) with additional Waterfall Park Flows (23.4 l/s) – 5 year ARI design storm

It should be noted that limited information has been made available to date regarding the levels of this 300mm wastewater pipe, with modelled levels taken from QLDC's GIS which just provides invert and ground levels at the upstream end of the pipe (at the confluence with the Norfolk St and Millbrook rising mains) and at the downstream end (at the Arrowtown-Lake Hayes pump station), with no information provided regarding levels at intermediate points along its length. It is understood that this pipeline, whilst generally operating as a gravity pipe, is designed to operate under pressure if flows exceed the on-grade capacity of the pipeline

3.3 Pump Station Assessment – Current Scenario (2015)

The 300mm Arrowtown-Lake Hayes wastewater line conveys flow from the Norfolk Road Pump Station (maximum capacity 70 l/s) and the Millbrook pump station (maximum capacity 24 l/s) to the Arrowtown-Lake Hayes Pump Station. The modelled inflows and outflows for the Arrowtown-Lake Hayes PS post-development scenario are shown in Figure 4 below.

The Arrowtown-Lake Hayes Pump Station has a maximum capacity of 85 l/s with one pump operating (based on QLDC records). In the post-development scenario (with the 23.4 l/s from Waterfall Park connected), the peak modelled inflow to the pump station is 81 l/s in the 5-year ARI design storm (as shown by the red trace). As shown by the yellow trace, the majority of flows entering the pump station are received from the 300mm line and the Waterfall Park development.

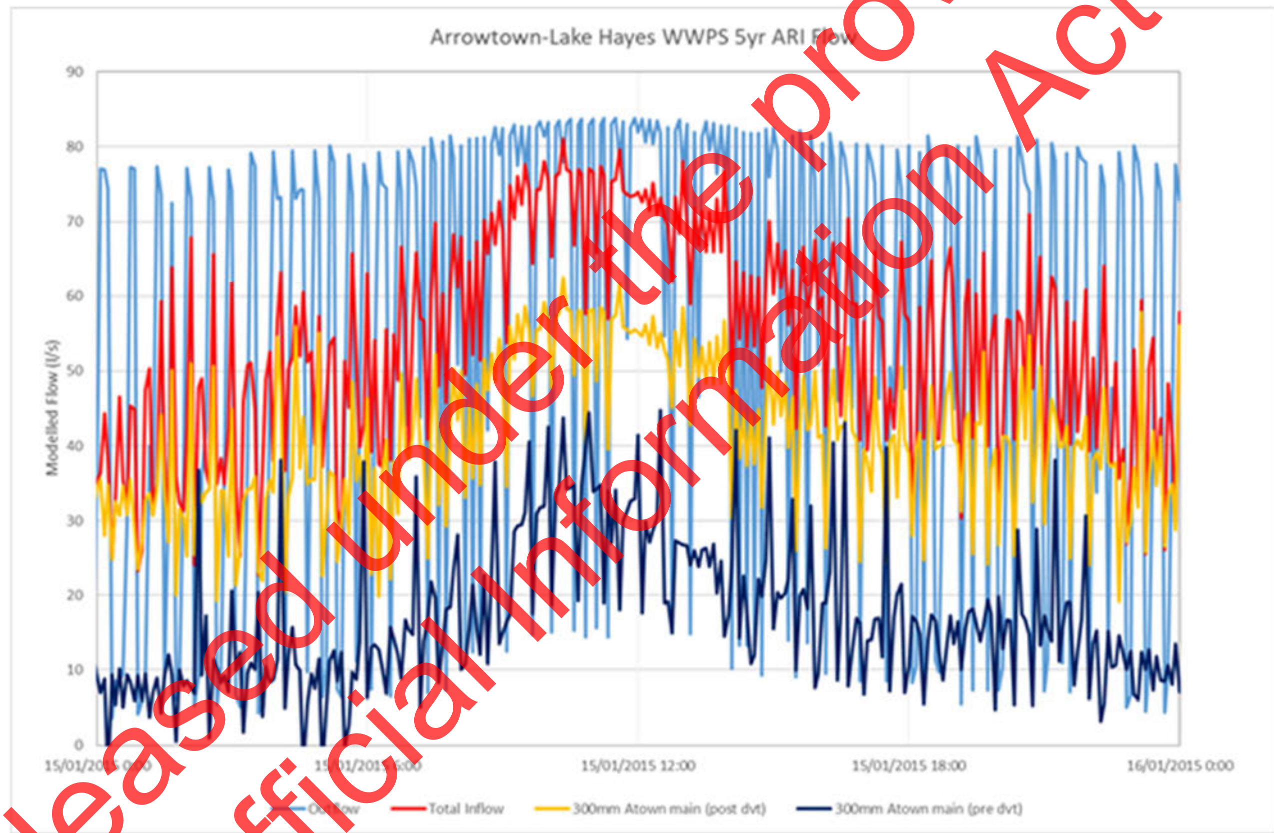


Figure 4: Modelled Arrowtown-Lake Hayes Pump Station flows – 5 year ARI design storm

3.4 Pump Station Assessment – Future Scenario (2055)

Based on a future (2055) population scenario, an assessment was made of the capacities of the relevant pump stations discharging to the Arrowtown-Lake Hayes Pump Station, and can be summarised in the Figure 5 schematic below.

While there is current (2015) capacity in the Arrowtown-Lake Hayes Pump Station for the proposed development, future significant growth in the remainder of the contributing catchment (in addition to the proposed Waterfall Park flow of 23.4 l/s) will likely trigger pump station upgrade requirements.

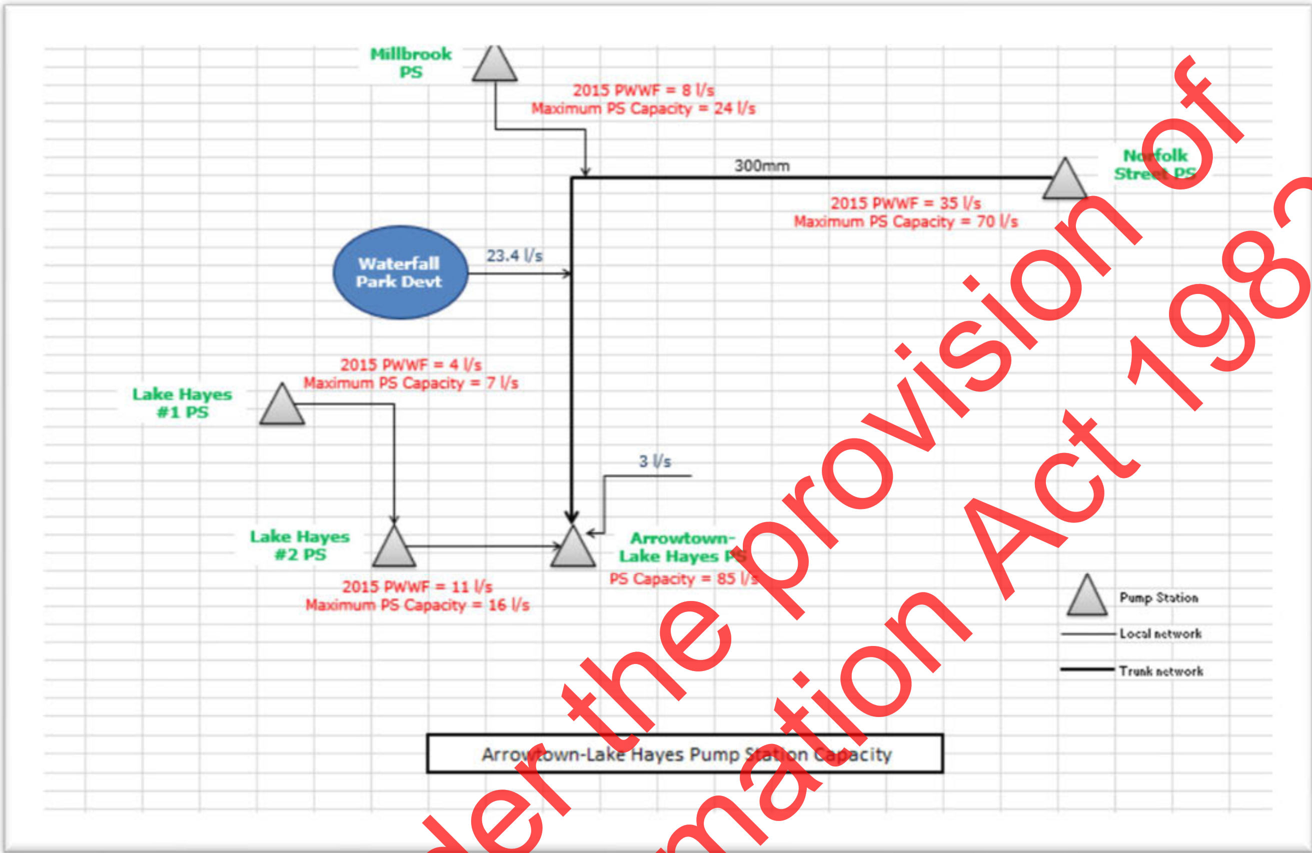


Figure 5: Pump station capacity current (2015) scenario versus theoretical maximum flows

3.5 Pressure at Arrowtown-Lake Hayes 300mm line connection point

In both the current (2015) and future (2055) scenarios, there is sufficient capacity within the 300mm line to receive the additional flows from the Waterfall Park development. Based on the GIS data available, the wastewater line appears to discharges as free flow via gravity (i.e. not pressurized) to the Arrowtown-Lake Hayes Pump Station.

The proposed connection point of the Waterfall Park development to the Arrowtown-Lake Hayes 300mm line has been constructed in the model with an estimated ground and invert level based on existing data. Insufficient level data is available to determine whether there are sections of this pipeline that don't operate under gravity conditions (and hence may operate under pressure), and is recommended as part of the design process for the Waterfall Park development, an assessment is made of actual levels at the proposed connection point to determine whether the pipeline is expected to operate under pressure, and to determine the head that the proposed Waterfall pump station will operate at.

APPENDI C

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

Mason Bros. Building
Level 2, 139 Pakenham
Street West
Wynyard Quarter
Auckland 1010
PO Box 37525, Parnell,
1151
New Zealand

T +64 (0)9 375 2400
mottmac.com

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 ackground

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.

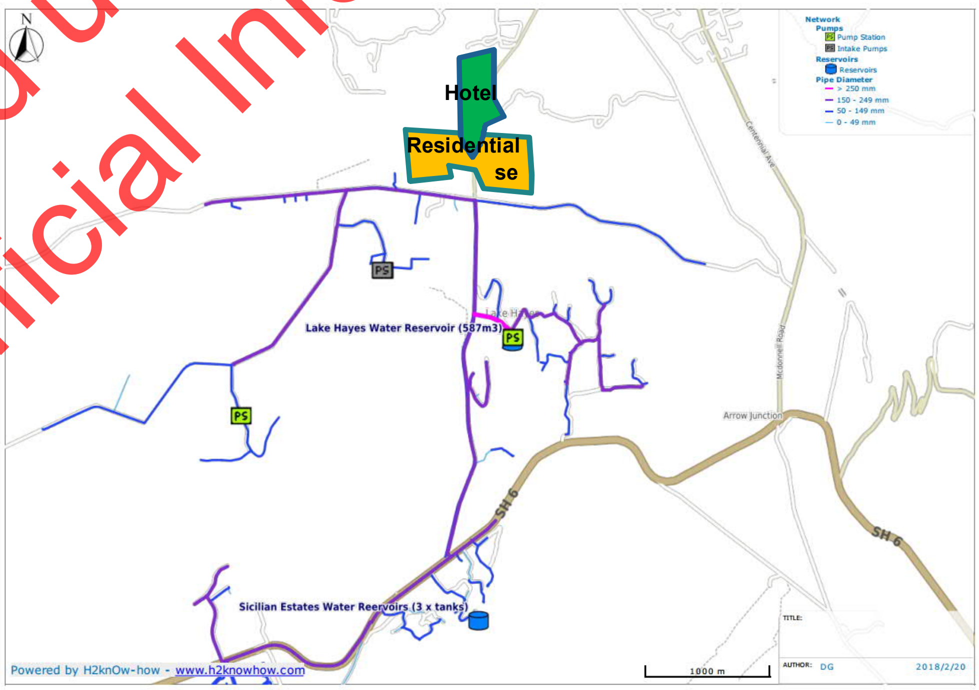


Figure 1 - Proposed Development   ocation

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

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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: R 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: R 36 m)	
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.**

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

Table 2 - ake Hayes Demands

DMA one	**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.

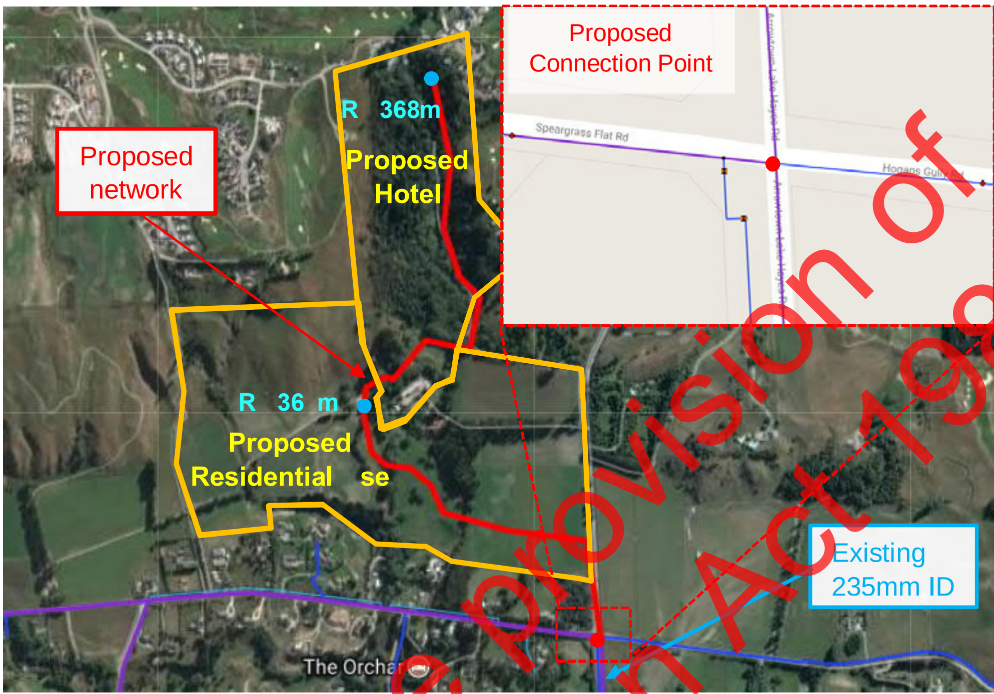

Figure 2 Proposed Development Location Network and Connection Point

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.

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

Scenario	Minimum Pressure (m)	Maximum Pressure (m)	Maximum Head osses (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.

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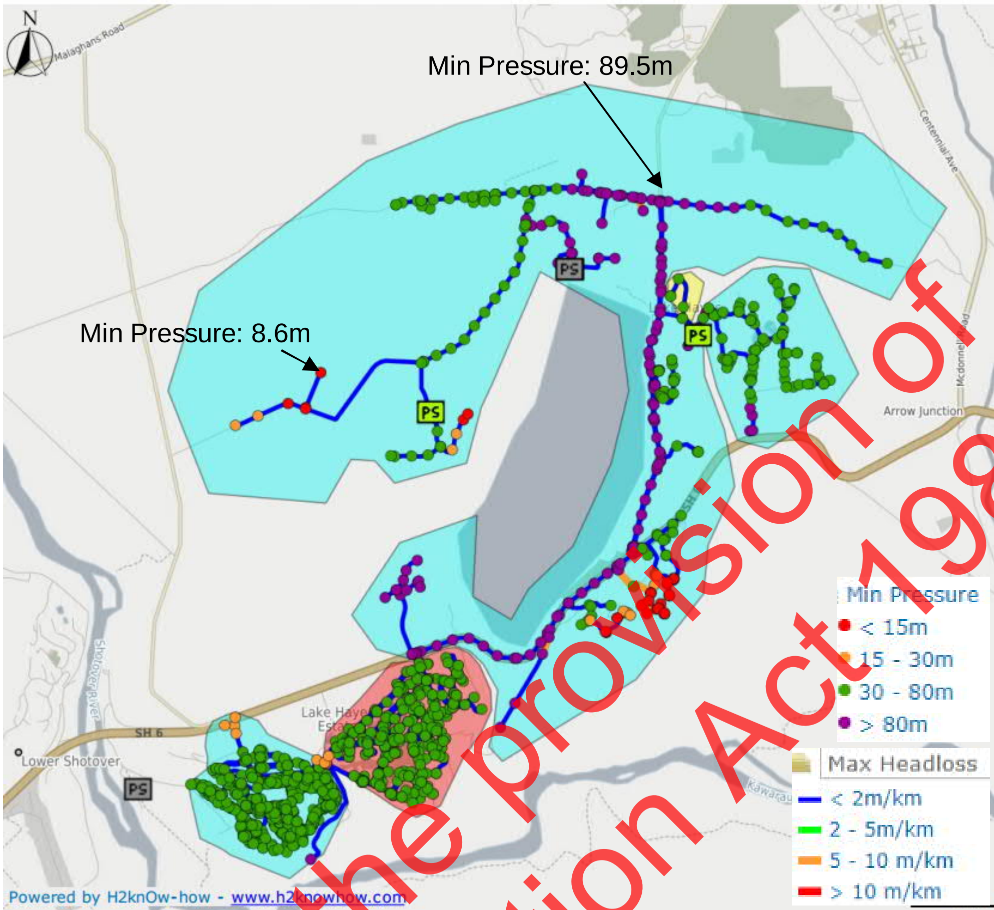

Figure 3 Current Peak Day System Performance Prior Development

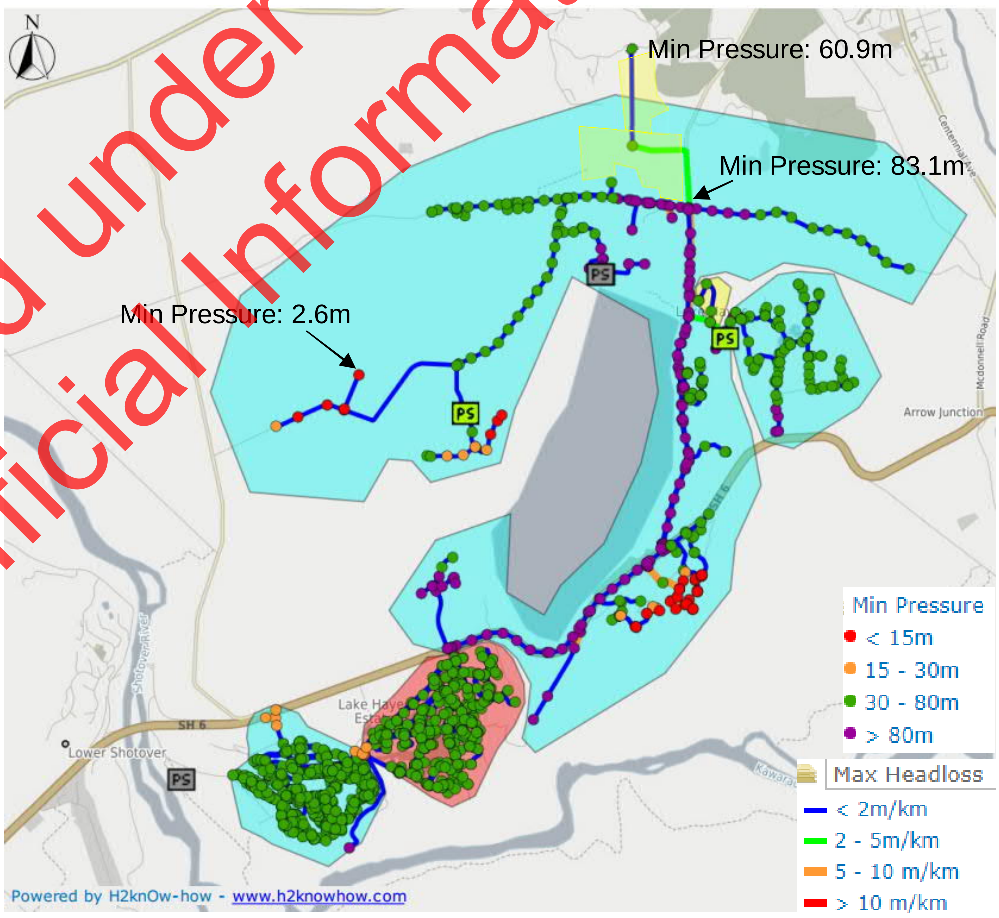

Figure 4 Current Peak Day System Performance - Post Development

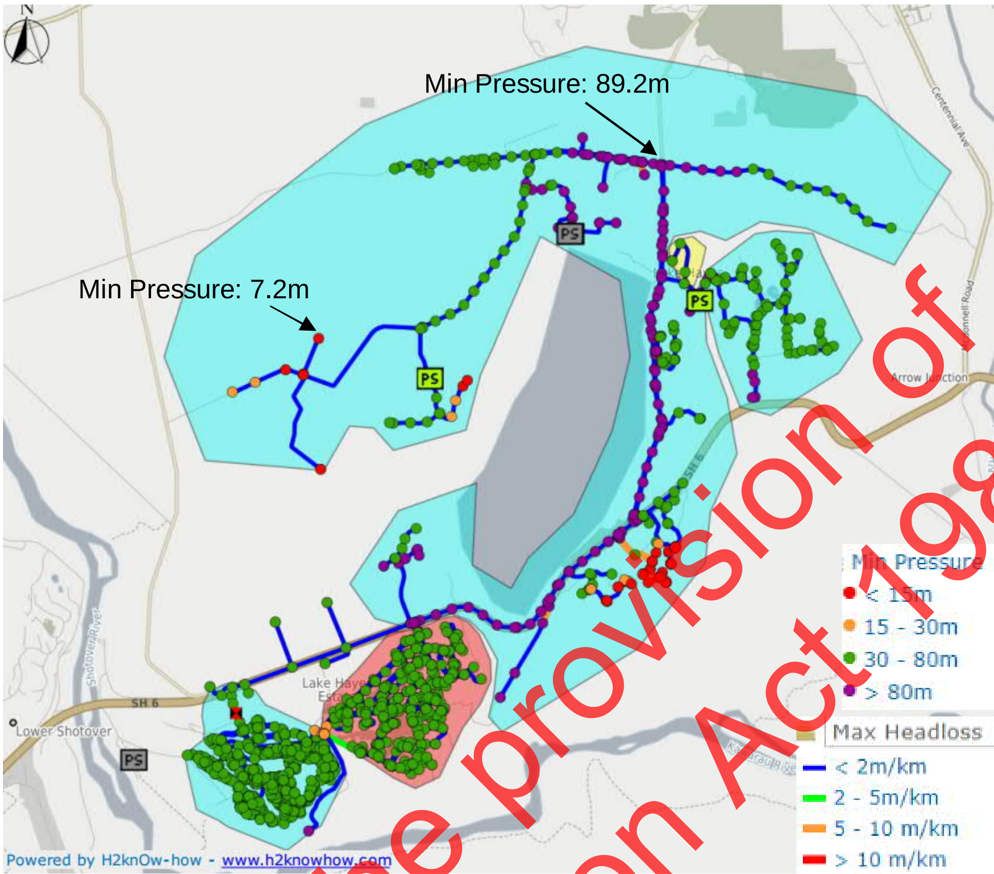

Figure 5 - 2028 Peak Day System Performance - Prior Development

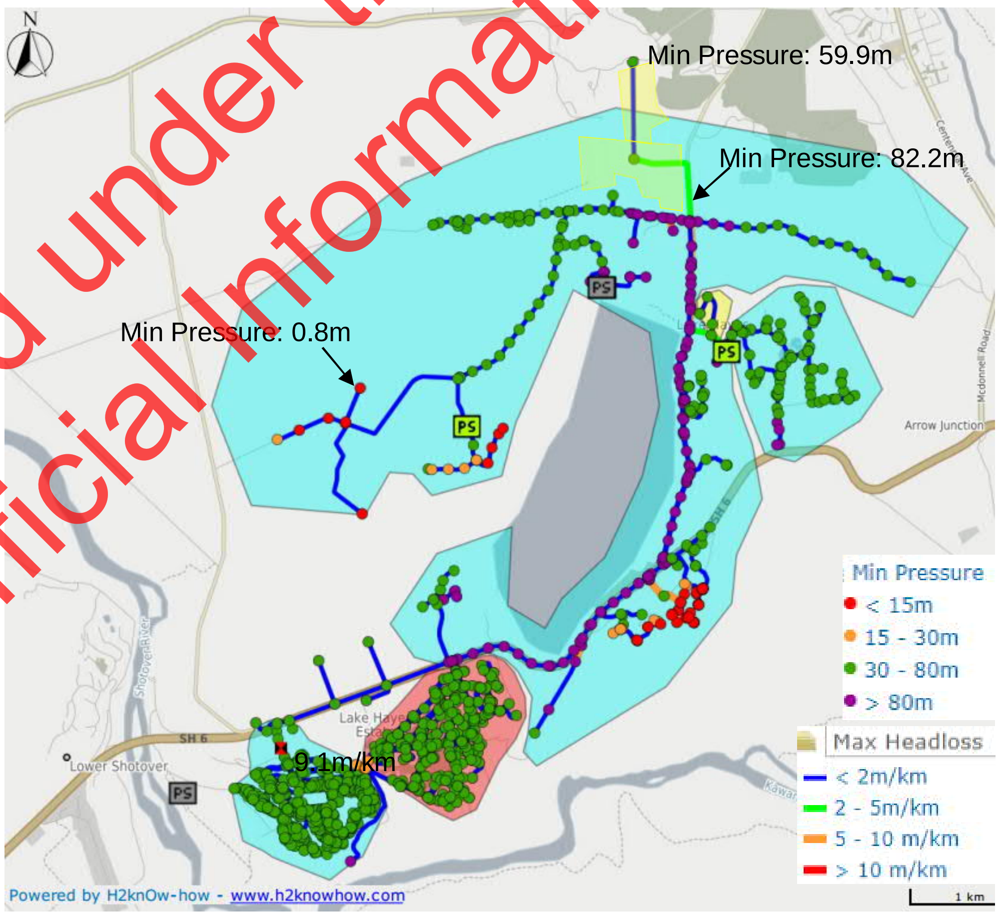

Figure 6 - 2028 Peak Day System Performance - Post Development

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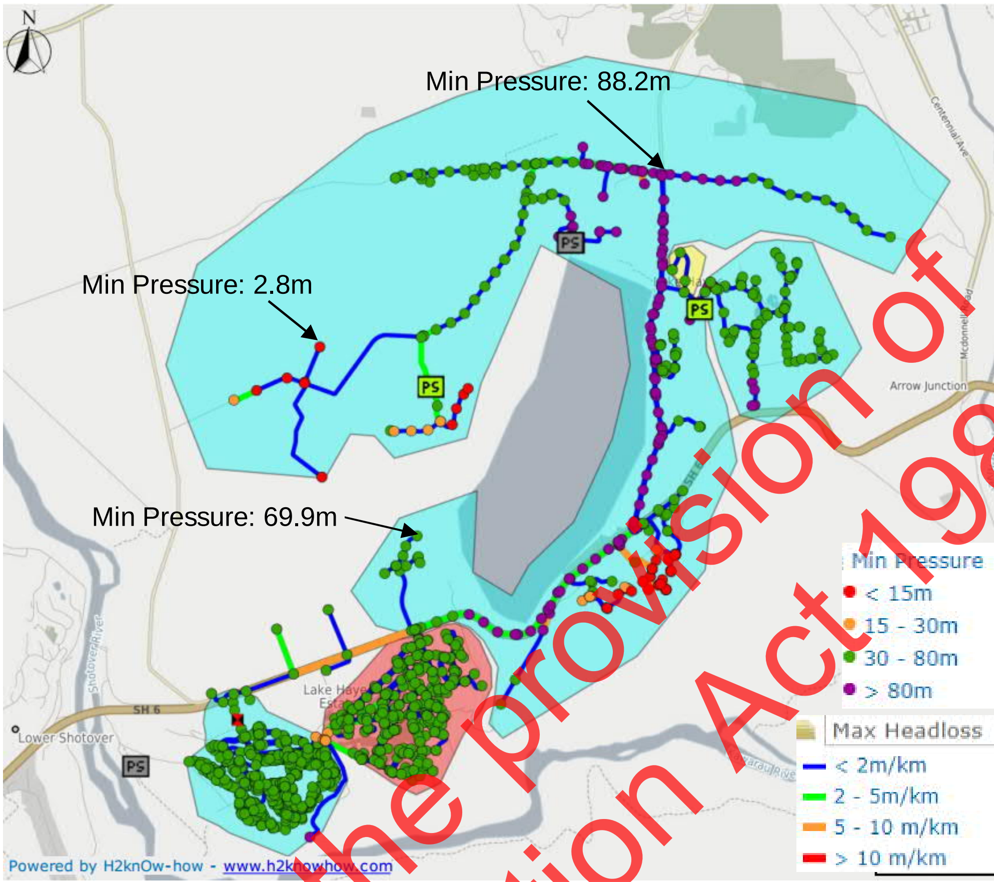

Figure - 2058 Peak Day System Performance - Prior Development

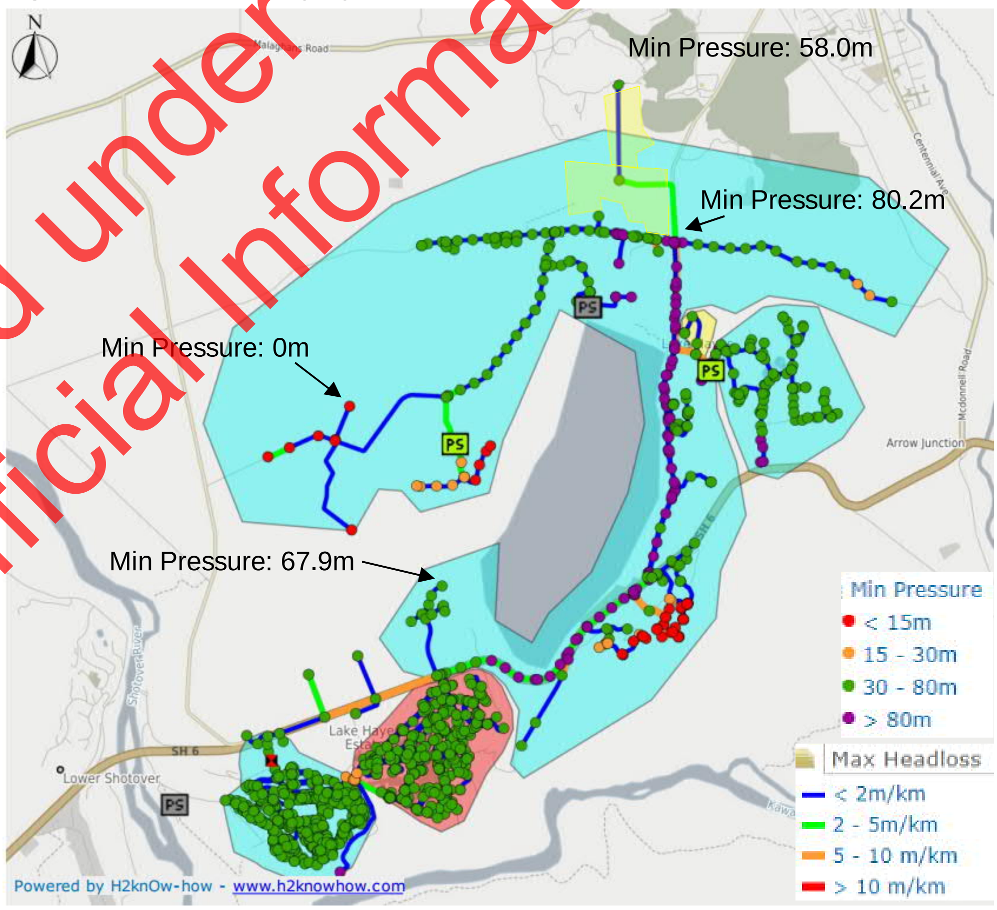

Figure 8 - 2058 Peak Day System Performance - Post Development

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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Diana Galindo
Hydraulic Engineer
s 9(2)(a)

Revision	Date	riginator	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 ayout

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

APPENDI D

Cosgrove's Email Concerning Fire Fighting Requirements

From: Daniel Jessop s 9(2)(a)
Sent: Thursday, 28 November 2019 3:10 PM
To: 'Jayne Richards' s 9(2)(a)
Cc: 'Brady Cosgrove' s 9(2)(a) 'Jakub Macak' s 9(2)(a) 'Lauren Christie' s 9(2)(a); 'Klemens Markiewicz' s 9(2)(a); 'Sam Ballam' s 9(2)(a); 'Martin Robertson' < s 9(2)(a)
Subject: RE: CS19078 Ayrburn Development - Water supply requirements

Jayne,

Thanks for the phone call, to (hopefully) clarify the requirements;

The sprinkler system demand is expected to be no more than 720 L/min @ 450 kPa for the Care Home building and this supply needs to be provided by the incoming mains. The 450 kPa pressure does not need to be achieved when the in-ground hydrants are at full flow, i.e. it can be assumed that the in-ground hydrants are at 0 flow when assessing the water supply pressure for the sprinkler system. Obviously the sprinkler system is at full flow (720 L/min) for this scenario.

With the sprinkler system and hydrants at full flow (i.e. 37 L/s for the Care Home), there needs to be residual pressure of 100 kPa in the incoming mains. Note that the 'FW3' category for non-sprinklered buildings requiring 50 L/s may be more onerous than the sprinkler protected building requirement?

I believe this is what is described in my e-mail below, however apologies if it is not clear.

Also as discussed, yes you can design so that the sprinkler system has 450 kPa pressure with the sprinkler system + hydrants at full flow, however this may have cost implications? It would be a Client decision in our view as it is beyond the minimum requirements of the relevant Standards for complying with the NZ Building Code.

Kind regards,
Daniel Jessop | Senior Engineer (Fire)
Cosgroves Ltd | Level 1, 23-27 Beach St | Queenstown 9300
M s 9(2)(a) | W www.cosgroves.com

From: Daniel Jessop s 9(2)(a)
Sent: Thursday, 28 November 2019 1:10 PM
To: 'Jayne Richards' s 9(2)(a)
Cc: 'Brady Cosgrove' s 9(2)(a); 'Jakub Macak' s 9(2)(a); 'Lauren Christie' s 9(2)(a); 'Klemens Markiewicz' s 9(2)(a); 'Sam Ballam' s 9(2)(a); 'Martin Robertson' s 9(2)(a)
Subject: RE: CS19078 Ayrburn Development - Water supply requirements

Hi Jayne,

Generally your summary aligns with our understanding. For clarity I've prepared some further advice as follows.

In regards to pressure and flows of the sprinkler and hydrant systems the following 'rules' apply, based on the relevant Standards:

a. The sprinkler system water supply needs to achieve the required performance at the design flow + pressure as per e-mail below, however it can be assumed for this case that the hydrants are not being used
b. With the hydrants and sprinkler system at full flow (serving one building), the residual pressure in the mains needs to be 100 kPa.

With respect to the proposed development and the building types, we summarise the requirements as follows:

a. Single level Housing (non-sprinklered) – FW2
b. Active Recreation/Amenities Building (non-sprinklered) – FW3
c. Multi-level Apartments (non-sprinklered) – FW3
d. Care Home (Sprinklered) – FW2
e. Childcare Centre (non-sprinklered) – FW3
f. Maintenance (non-sprinklered) – FW3
g. Medical Centre (non-sprinklered) – FW3

Water supply requirements:
a. FW2 - 25 L/s total (12.5 L/s each from two hydrants)
b. FW3 - 50 L/s total (25 L/s each from two hydrants)

Apologies I didn't come back to you yesterday – I was pretty well buried until late in the day.

Give me a call if you have any questions about the above.

Kind regards,
Daniel Jessop | Senior Engineer (Fire)
Cosgroves Ltd | Level 1, 23-27 Beach St | Queenstown 9300
M s 9(2)(a) | **W** www.cosgroves.com

AUCKLAND | CHRISTCHURCH | QUEENSTOWN

APPENDI E

Hydraulic Calculations

Fluent SOLUTIONS

Title	Hydraulic Calculations to assess maximum headlosses in 235mm ID pipe
Job No.	Q000492
Job Title:	Northbrook Retirement Village
Engineer:	Jayne Richards
Checked:	Anthony Steel
Date:	1/04/2020

Pre-development flows - estimated from Mott MacDonald Headloss/km in Table 6 (to match headloss in m/km)

Pipe Dia (ID)	Flow Q (l/s)	Area A (m^2)	Pipe diameter m	Roughness Coefficient k (mm)	Length L (m)	Kinematic Viscosity ν (10^{-6} m^2/s)	Mean Velocity V (m/s)	Hydraulic diameter D (m)	Reynolds Number Re	Friction Coefficient f	Total Head loss ΔH (m)	m/1000 m	Velocity Head	Description
235 mm	12.7	0.04338	0.235	0.015	1	1.5	0.29	0.235	45867	0.0214	0.00	0.40	0.004368623	**Estimated flow before development- current peak day**
235 mm	16	0.04338	0.235	0.015	1	1.5	0.37	0.235	57785	0.0204	0.00	0.60	0.006933893	**Estimated flow before development- 2028 peak day**
235 mm	22.4	0.04338	0.235	0.015	1	1.5	0.52	0.235	80899	0.0190	0.00	1.10	0.01359043	**Estimated flow before development- 2058 peak day**

Post-development flows - estimated from Mott MacDonald Headloss/km in Table 6 (to match headloss in m/km)

Pipe Dia (ID)	Flow Q (l/s)	Area A (m^2)	Pipe diameter m	Roughness Coefficient k (mm)	Length L (m)	Kinematic Viscosity ν (10^{-6} m^2/s)	Mean Velocity V (m/s)	Hydraulic diameter D (m)	Reynolds Number Re	Friction Coefficient f	Total Head loss ΔH (m)	m/1000 m	Velocity Head	Description
235 mm	57	0.04338	0.235	0.015	1	1.5	1.31	0.235	205859	0.0160	0.01	6.00	0.088000853	**Estimated flow post development- current peak day**
235 mm	60	0.04338	0.235	0.015	1	1.5	1.38	0.235	216694	0.0159	0.01	6.59	0.09750787	**Estimated flow post development- 2028 peak day**
235 mm	65.8	0.04338	0.235	0.015	1	1.5	1.52	0.235	237641	0.0156	0.01	7.81	0.117270548	**Estimated flow post development- 2058 peak day**

Approximate flows allocated to Waterfall Park development in Mott Macdonal Model:

Current peak day: 44.3 L/s
2028 peak day: 44 L/s
2058 peak day: 43.4 L/s

New flow (Waterfall Park Hotel plus Northbrook Retirement Village):

Current peak day: 31.3 L/s
2028 peak day: 31.3 L/s
2058 peak day: 31.3 L/s

New post-development flows (Mott Macdonald Pre-Development Flows plus Waterfall Park Hotel plus Northbrook Retirement Village):

Current peak day: 44 L/s
2028 peak day: 47.3 L/s
2058 peak day: 53.7 L/s

New post-development flows (Mott Macdonald Pre-Development Flows plus Waterfall Park Hotel plus Northbrook Retirement Village):

Pipe Dia (ID)	Flow Q (l/s)	Area A (m^2)	Pipe diameter m	Roughness Coefficient k (mm)	Length L (m)	Kinematic Viscosity ν (10^{-6} m^2/s)	Mean Velocity V (m/s)	Hydraulic diameter D (m)	Reynolds Number Re	Friction Coefficient f	Total Head loss ΔH (m)	m/1000 m	Velocity Head	Description
235 mm	44	0.04338	0.235	0.015	1	1.5	1.01	0.235	158909	0.0167	0.00	3.74	0.052437566	**Estimated flow post development- current peak day**
235 mm	47.3	0.04338	0.235	0.015	1	1.5	1.09	0.235	170827	0.0165	0.00	4.26	0.060598162	**Estimated flow post development- 2028 peak day**
235 mm	53.7	0.04338	0.235	0.015	1	1.5	1.24	0.235	193941	0.0162	0.01	5.38	0.078106242	**Estimated flow post development- 2058 peak day**

Extract from Mott Macdonald Report:

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.