Leeston Industrial – Infrastructure Assessment

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Document Details:

Date: 13 June 2019
Reference: 3-C1831.01
Status: Final

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Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
Version 1	13 June 2019	MdL, MG, LMM	DJ	AH	Final report

Revision Details

Revision	Details

1 Introduction

WSP Opus were engaged by Selwyn District Council (SDC) to complete a three waters infrastructure capacity assessment to support the extension of the Business 2 (Industrial) Zone in the south-east of Leeston township (LEE3 site). The proposed redevelopment of the LEE3 site will include additional land being zoned for industrial use, as indicated in the Proposed District Plan. The following report documents the assessment completed in accordance with the scope of work provided by WSP Opus on 29 January 2019. This scope of work included the following activities:

- Apply the water distribution model to assess the impact of LEE3 site on the network under existing and future growth conditions.
- Apply the wastewater reticulation model to assess the impact of the LEE3 site on the network, under existing and future conditions.
- Assess the impact of wastewater generated at the LEE3 on flow and load to the Leeston WWTP considering projected growth,
- Conduct a stormwater assessment of the existing and future site, and identify requirements for stormwater conveyance and treatment for the post-development site.

2 Background

2.1 Site Description

Figure 2-1 shows the location of the LEE3 site at the eastern end of the Leeston township. The site is comprised of a 10.52 ha area south of High Street and Station Road, and is roughly centred along Volckman Road with an Western boundary along Cunningham Street. The current site consists of a farm equipment supplier, tyre shop and open space used for farming.

Figure 2-1: Location of LEE3 Site

Leeston and the proposed LEE3 site is currently serviced by the following key water infrastructure:

- Bores at Gallipoli Street and Lake Road
- A network of DN100 and 150 mm trunk mains

Wastewater servicing for the existing site is provided by a 150mm diameter sewer on Cunningham Street, which discharges to the Station Street Pump Station (Station Street PS) from where flows are pumped directly to the Leeston wastewater treatment plant (WWTP) located at the southern limit of town.

2.2 Anticipated Growth

SDC completed an area-wide water and wastewater servicing Master Plan which identified several infrastructure upgrades to service anticipated growth. As part of this process, several areas were identified where growth is planned or likely to occur from 2017 through to 2047. Figure 2-2 presents the location of growth areas in addition to the LEE3 site, both in Leeston and the community of Doyleston to the northeast. All growth highlighted in Figure 2-2 is in the area contributing to the Leeston WWTP.

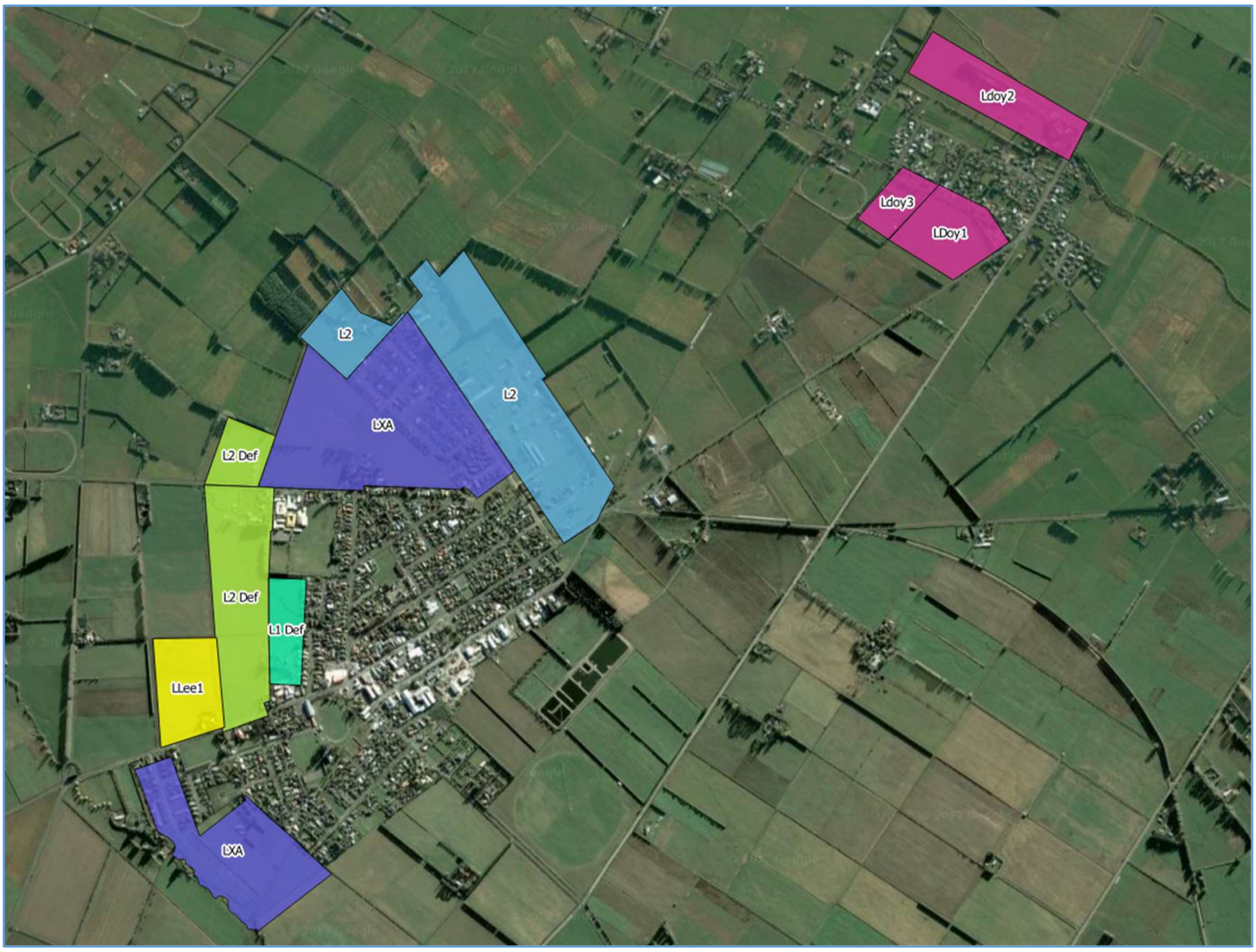


Figure 2-2: Master Planning Growth Areas

Table 2-1 provides details of the anticipated growth areas highlighted in Figure 2-2 based on the 2017 Master Planning projections as provided by SDC.

Table 2-1: Planned Number of Lots per Growth Area

Growth Area ID	Existing Lots (2017 / 2018)	Planned Lots
	Leeston	
LXA (Greenfield north)	142	218
LXA (South)	88	110
L2 (West)	25	299
L2 (East)	0	59
L2 Def (North)	0	47
L2 Def (South)	0	191
L1 Def	0	53
LLee1	0	25
Sub-total		*1,002*
	Doyleston	
LDoy1	0	85
Ldoy2	0	110
Ldoy3	0	85
Sub-total		*215*
Total		*1,217*

The Master Plan hydraulic models (water and wastewater) were applied in this study to assess the suitability of any proposed infrastructure upgrades to service the LEE3 site under existing and future conditions.

2.3 Proposed Infrastructure Upgrades

2.3.1 Water

Water Master planning for 2047 recommended the following upgrades to the current water supply network:

- Decommission Lake Road bore
- Construct two new bores on Leeston-Dunsandel Road
- Upgrade pipes on Leeston-Dunsandel Road, Cunningham Street, Gallipoli Street, Market Street, High St, Manse Road and the trunk main on Leeston Road (between Leeston and Doyleston)

Apart from the pipe upgrade on the Leeston to Doyleston trunk main, there are no other pipe upgrades ear-marked near the LEE3 development.

Figure 2-3 shows the pipe diameters and well sites in the 2047 master planning model.

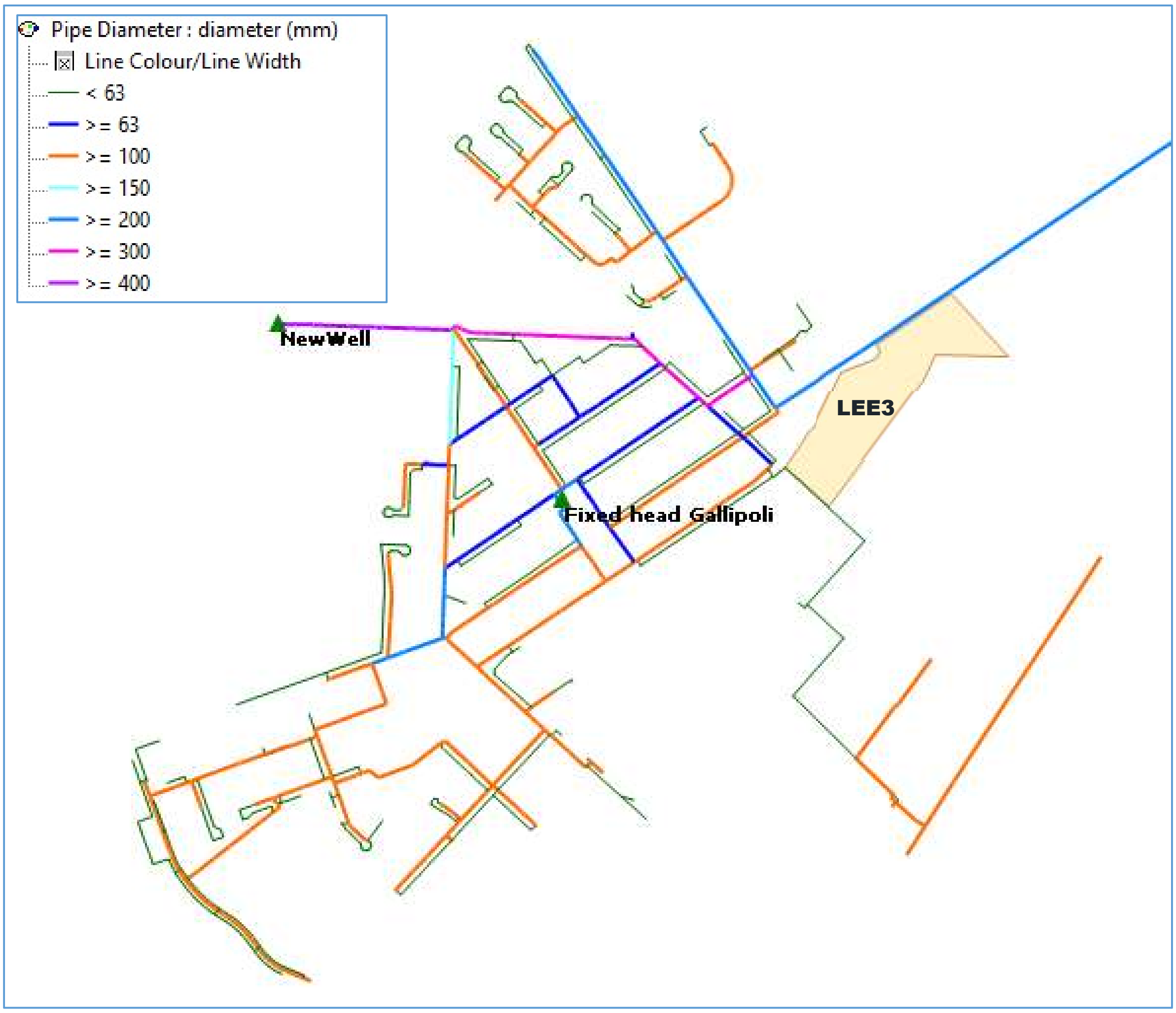


Figure 2-3: Master Planning 2047 Leeston Pipe Network

2.3.2 Wastewater

Wastewater hydraulic modelling previously completed by WSP Opus in 2016 for Leeston recommended that the sewer on Cunningham Street be upsized to 225-300mm diameter pipe and the capacity of the duty pump at the Station Street PS be increased to 65L/s. Furthermore, as part of a separate project, WSP Opus provided options for the Leeston WWTP to accommodate growth in the contributing area. The report '*Leeston WWTP Options Report – Revised Population Options (WSP Opus August 2017 Report)*' found that the existing irrigation area of 28.6 ha will be unable to meet the consented nitrogen standard of 200 kgN/ha + 250 kgN/ha carry off allowance in 2018 but can be managed by increasing the irrigation area to 35.3 ha, which was already available. The report found that based on the growth predictions, the increased area will also become limited for nitrogen application and further capital expenditure to meet consent conditions will be required by 2023/2024. The WSP Opus August 2017 Report included a 20% contribution from existing commercial areas, the report however stated that the 20% on flow for the commercial contribution was conservative.

2.4 Assumptions

WSP Opus provided a detailed list of assumptions that were applied in assessing infrastructure requirements to service the LEE3 in a memorandum dated 4 March 2019. Appendix A contains this memorandum, with key assumptions as follows:

- An average water usage rate of 0.15 L/s/ha was determined to be appropriate to represent demand from the LEE3 site. A peaking factor of 1.8 was applied to this value to represent the peak daily demand, resulting in a design demand of 2.84L/s being applied in the water model.
- For wastewater generation, it was assumed that 100% of water provided to the site is discharge to the collection system. A peaking factor of 5 was also applied to wastewater flows, resulting in a peak design flow of 7.94 L/s being applied in the wastewater model.
- Expected flows to the WWTP are projected to increase from 713 m^3/d (2013) to 1,511 m^3/d (2048). Water Quality of the existing population was estimated on the base of an average production per habitant (60 g/BOD; 70 g TSS; 12 g TKN; 7.5 g NH_3 and 5 g TP). Average contaminant concentrations in wastewater discharged from the LEE3 site are:

Parameter	Concentration Average (mg/l)
BOD	350
TSS	390
NH_3	42
TKN	67
TP	27

- Required Effluent Quality from the WWTP are:

Parameter	Average	Unit
BOD	10	mg/l
TSS	15	mg/l
NH_3	5	mg/l
TN	43.6	kg/d
TN	28.9	mg/l
TP	5.8	kg/d
TP	3.8	mg/l

- For the stormwater assessment, it was assumed the site will consist of a maximum 80% impervious coverage (allowing space for stormwater management and landscaping). On-site stormwater treatment systems will be provided as part of each sites development and will limit flows to the greenfield equivalent. Each site is to have a maximum hard-stand area of 100m^2 discharging to the existing road corridor with a single access point.
- Growth projections for Leeston documented in previous reports completed by WSP Opus were assumed to remain valid for this assessment.

3 Infrastructure Assessment

The following sections provide details of the infrastructure assessment completed to determine the servicing requirements for the LEE3 site.

3.1 Water

3.1.1 Methodology

The water modelling was undertaken using Infoworks WS-Pro (master database = WS Pro (2.5.7) Selwyn Master Planning (converted v. 15) and the updated and verified 2017 peak day model (base network = LEE DOY Peak Day 2017) and the updated 2047 master planning model (base network = LEE DOY Master Planning 2047_Revised Upgrades).

Two scenarios have been run in both the 2017 and 2047 models to check current and future infrastructure capacity and the sensitivity of local pipe capacity to changes to the design flow:

- Scenario 1 – Peak design flow = 10.52 ha * 0.15 L/s/ha * 1.8 = 2.84 L/s
- Scenario 2 – Peak design flow = 10.52 ha * 0.38 L/s/ha * 1.8 = 7.2 L/s

Further to the assumptions outlined in Appendix A:

- Demand category = 10-hour commercial
- Three connection points to the existing network: one connection to existing pipes at the intersection of Station and Cunningham Streets and two connections onto the Leeston – Doyleston trunk main on Leeston Road
- Demand is divided evenly between all connection points.
- Leakage = 197 L/connection/day (2016 Water Balance Report). Given that number of connections is currently unknown, assume a nominal leakage of 0.12 L/s (based on a nominal land parcel size of 2000 m2 and 50 allotments)

The proposed LEE3 development, existing water pipes and assumed connection points are shown in Figure 3-1. Note that connection point 1 and 2 are very likely to be linked by a connecting pipe and this has been modelled as a DN150 pipe.

Figure 3-1: Assumed connection points between existing water network and LEE3

3.1.2 Results

Scenario 1 – Existing Network

Figure 3-2 shows the worst-case node pressures and pipe head-losses in the existing peak day model.

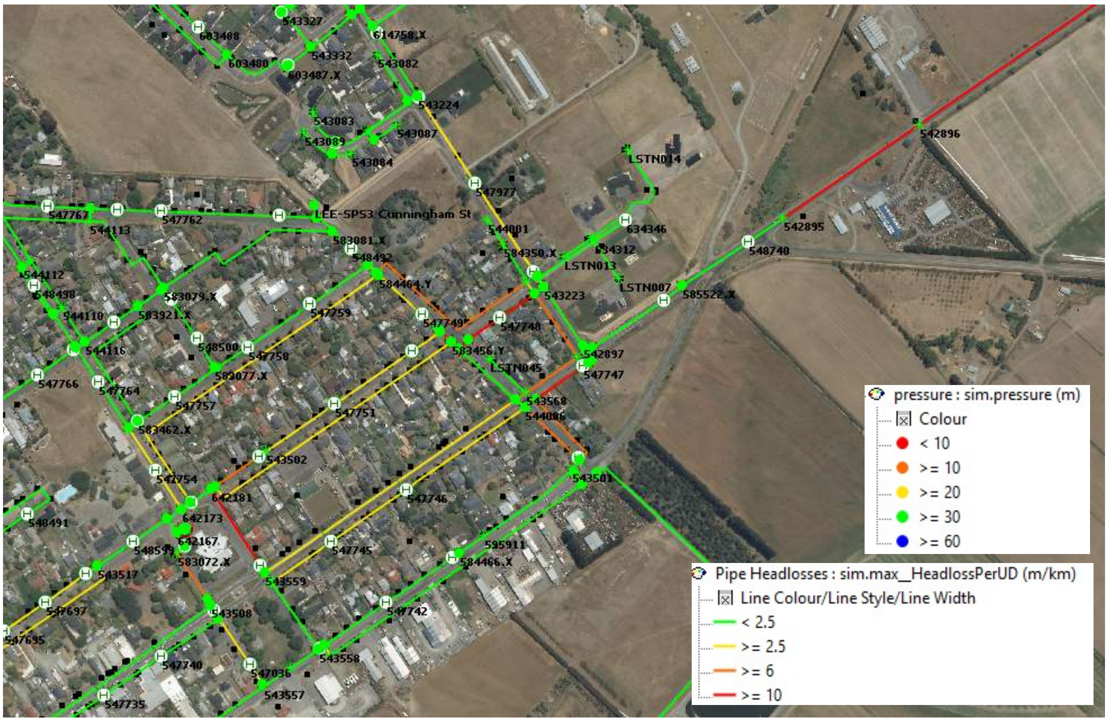


Figure 3-2: Peak day pipe head-losses in the existing network

There are already some existing capacity and velocity issues in pipes near LEE3 with several pipes showing head-losses greater than 2.5 m/km and velocities greater than 1.0 m/s. Pipes highlighted in orange and red have high or very high pipe head-losses. Despite the capacity issues, there are no issues with pressure level of service.

Figure 3-3 shows the pressure drop at connections 1 and 3 resulting from the additional demand at LEE3. Note that the large network pressure drops occur when the Osbourne Park Reservoir is filling and the reservoir float valve is open.

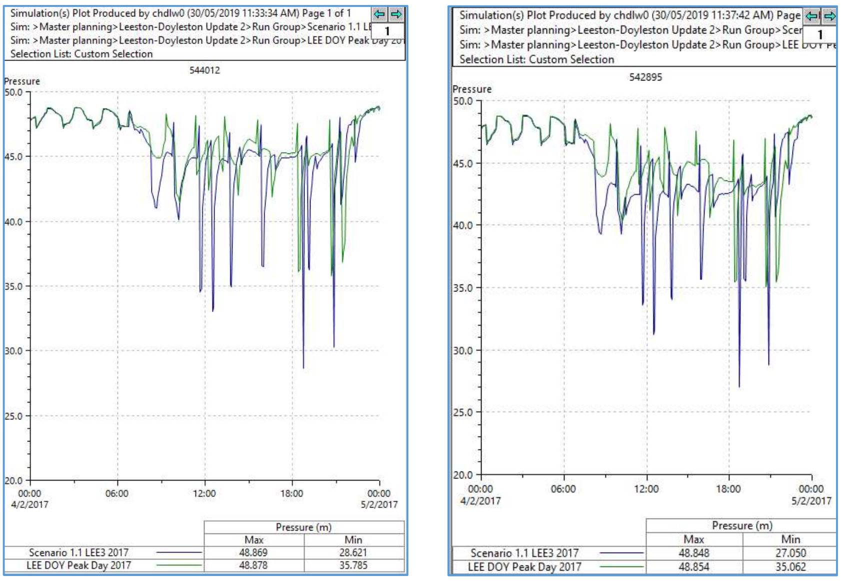


Figure 3-3: Comparison of network pressures at connection point 1 and 3 with and without LEE3

The increased demand at LEE3 further increases pipe head-losses on the Leeston-Doyleston DN75mm trunk main. However, despite pressure drops of up to 8-9m when the Osbourne Park Reservoir is filling, network pressures do not fall below the pressure level of service and reservoir filling is not impacted.

Scenario 2 – Existing Network

Figure 3-4 shows the network pressures when LEE3 demand = 7.2 L/s.

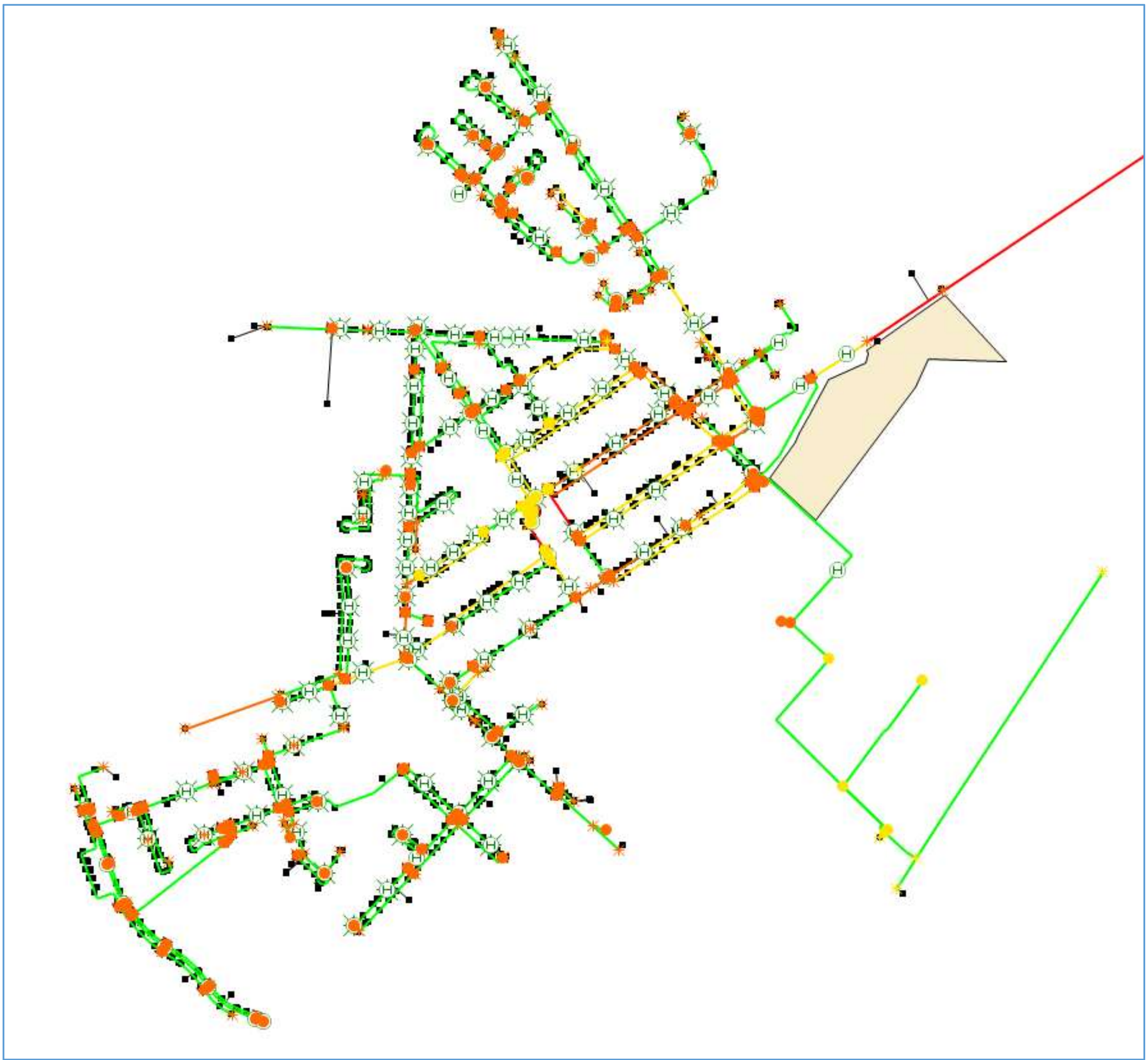


Figure 3-4: Peak day pipe head-losses in the existing network with LEE3 added (Scenario 2 demand)

The existing network cannot support the increased demand. Network upgrades are required to service the LEE3 development if peak demand exceeds approximately 4 L/s.

Scenario 1 – 2047 master planning model with recommended upgrades

Figure 3-5 shows the worst-case node pressures and pipe head-losses when the scenario one - LEE3 demand is added to the 2047 master planning model.

Figure 3-5: Peak day pipe head-losses in the 2047 network (with Scenario 1 demand and upgrades)

The addition of LEE3 does not create any pressure or level of service issues in the 2047 network. No additional upgrades are required to service demand scenario 1 (2.84 L/s).

Scenario 2 – 2047 master planning model with recommended upgrades

The scenario 2 demand does not create any pressure or level of service issues in the 2047 network. No additional upgrades are required to service demand scenario 2 (7.2 L/s). Figure 3-6 shows that there is very little impact on peak hour pressure resulting from addition of LEE3 scenario 2 demand.

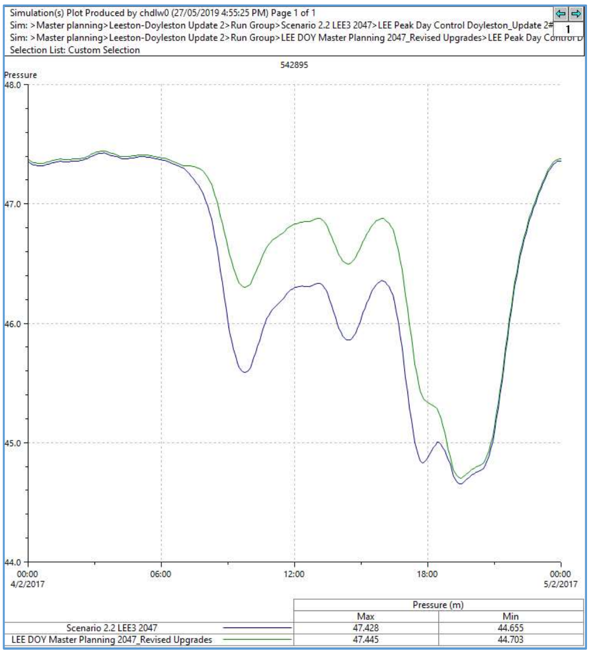


Figure 3-6: Pressure drop at connection 1 resulting from addition of scenario 2 LEE3 demand in 2047 master planning model

3.2 Fire Flow

Fire flow has not been assessed as part of this investigation. The fire flow requirements for each commercial / industrial property should be assessed on a case by case basis according to the New Zealand Fire Service Firefighting Water Supplies Code of Practice.

3.3 Wastewater

3.3.1 Risk Management Approach

The impact of wastewater flows from the LEE3 site was assessed against performance of the network under existing conditions and considering planned growth in Leeston. Impact of flows from LEE3 were assessed by reviewing the extent of surcharged pipes, sewer overflows and pump station run time increases.

System performance was assessed using the modelled system to the 5-year storm event. Figure 3-7 presents the profile of this rainfall event, having a duration of one-hour, peak intensity of 56.3 mm/hr, and total precipitation volume of 27.9mm. No allowance for climate change is included in this rainfall data, this is consistent with previous growth assessment work for SDC.

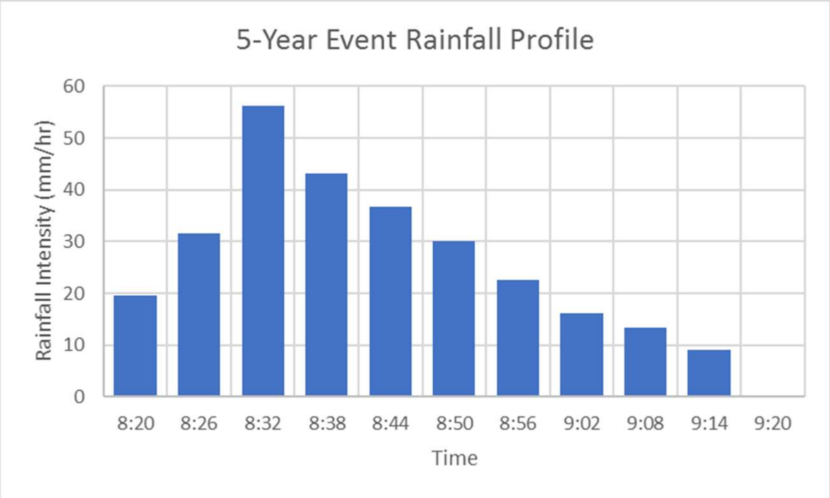


Figure 3-7: 5-Year Event Rainfall Profile

3.3.2 Model Scenarios

Four scenarios were defined in the wastewater collection system model to assess sewer conditions both before and after the inclusion of the LEE3 site. Each scenario and its significance are described below.

Scenario 1: Existing Conditions

This scenario was used to understand the existing capacity in the wastewater collection and as a baseline for comparison with post-development conditions. System performance was evaluated under peak flow conditions resulting from the 5-year storm event using the existing conditions model which was calibrated in 2017.

Scenario 2: Future Conditions without LEE3

This scenario was used to evaluate future performance of the existing wastewater collection system prior to the inclusion of the LEE3 site. This approach allowed for the impact of flows from the LEE3 site to be isolated in comparison to a later scenario. System performance was evaluated under peak flow conditions resulting from the 5-year storm event using the future conditions model. High groundwater is also assumed, similar to the levels that occurred in 2014, which increases the baseflow in the network.

Scenario 3: Future Conditions with LEE3
This scenario was used to evaluate future performance of the existing wastewater collection system with the inclusion of the LEE3 site. System performance was evaluated under peak flow conditions resulting from the 5-year storm event using the future conditions model (including high groundwater) with a design flow added to represent the LEE3 site.

Scenario 4: Future Conditions with LEE3 and Proposed System Upgrades
This scenario was used to evaluate future performance of the wastewater collection system with the inclusion of LEE3 site and previously recommended upgrades to the wastewater collection system. The purpose of this scenario was the assess the suitability of the previously proposed upgrades considering updated flows from the LEE3 site. System performance was evaluated under peak flow conditions resulting from the 5-year storm event using the future conditions model (including high groundwater) with a design flow added to represent the LEE3 site.

3.3.3 Analysis Results

The following presents model results of the scenarios described in Section 3.3.2. Appendix B contains sewer maximum hydraulic grade line (HGL) profiles along Cunningham Street for all modelled scenarios. It is noted that performance of the Station Street PS rising main was not considered as part of this assessment.

Scenario 1: Existing Conditions
Model results for this scenario indicate there are existing capacity issues in the sewer that is planned to receive flows from the LEE3 site. Sewer surcharging is predicted on Cunningham Street from the Station Street PS to Selwyn Street. Backwater effects are predicted in sewers discharging to the Cunningham Street sewer, with surcharging on Pennington Street, Selwyn Street and High Street and upstream sewers on Cunningham Street. The peak simulated flow to the Station Street PS is 26.1 L/s.

Scenario 2: Future Conditions without LEE3
Model results for this scenario indicate that existing sewer surcharging resulting from the 5-year event in the sewer on Cunningham Street is exacerbated by anticipated growth. Surcharging is predicted to extend further north on Cunningham Street to include sewers up to Pennington Street. The peak simulated flow to the Station Street PS is 29.7 L/s, note this peak flow is restricted by the pipe capacity.

Scenario 3: Future Conditions with LEE3
Model simulations for future conditions with the inclusion of the LEE3 site predicted similar results to the previous scenario. The model predicted sewer surcharging on Cunningham Street from the Station Street PS to Pennington Street. The peak simulated flow to Station Street PS is 33.3 L/s, note this peak flow is restricted by the pipe capacity.

Scenario 4: Future Conditions with LEE3 and Proposed System Upgrades
Model results for this scenario indicate that previously recommended upgrades on Cunningham Street have adequate capacity to convey flows from the 5-year event, under future conditions with the inclusion of flows from the LEE3 site. Peak simulated flows to Station Street PS are 67.1 L/s.

3.4 Wastewater Treatment

The findings of the WSP Opus August 2017 Report form the basis of the current WWTP evaluation. The wastewater associated with the LEE3 site was added to the 2017 data and the growth of the existing commercial areas was varied to ensure that the growth of these areas are also included.

3.4.1 Wastewater Treatment Plant Loads and Flows

Wastewater Flow

The population data used in the WSP Opus August 2017 Report was the same as those supplied by SDC on 8 March 2019 as Leeston Growth. The WSP Opus August 2017 WSP Report therefore served as the basis for estimating the flows from the communities of Leeston, Southbridge and Doyleston for this evaluation. The wastewater flow from the LEE3 site was added to the 2017 flow data for this evaluation.

The wastewater flow to the Leeston WWTP was calculated using the population growth data for the communities it serves. The flow prediction assumes that for each additional person connected, the average flow increases by 200 L/hd/d, which includes usages and infiltration. Figure 3-8 shows the effect of population growth on the wastewater flows.

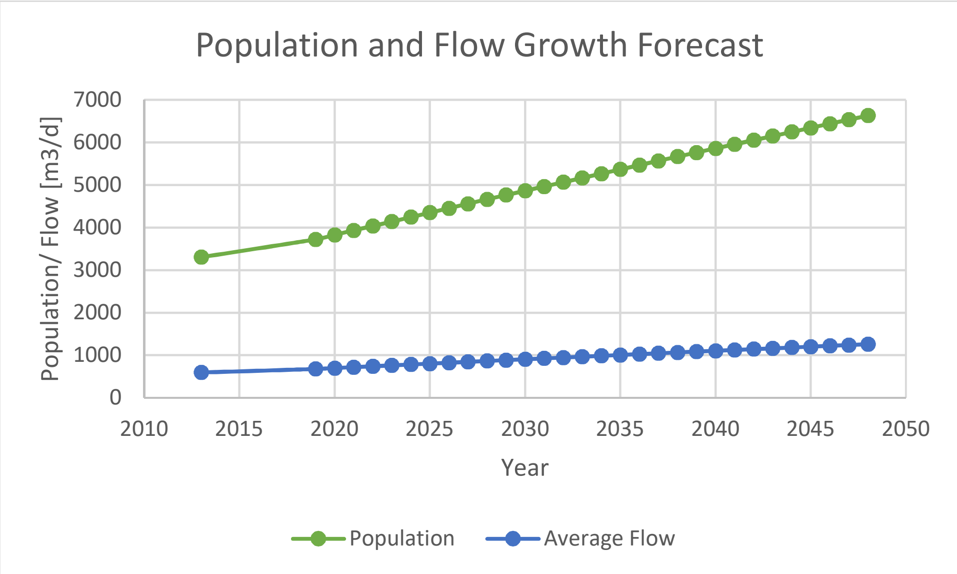


Figure 3-8 Population growth and wastewater flow forecast

In the previous report a 20% commercial flow contribution was included in the evaluation to account for growth of the existing commercial areas. While allowance must be made for growth in these areas, the 20% flow contribution was considered conservative. As such the existing commercial growth in this evaluation was based rather on growth in population. In this evaluation a 0%, 5% and 10% increase in population was included to account for growth of the existing commercial areas.

As detailed information about the nature and type of industries planned for Business 2 Zone is not currently known, the wastewater flow was estimated using the Christchurch City Council (CCC) Infrastructure Design Standard (IDS). The IDS provides guidance on the average design wastewater flow from commercial developments. The IDS states that where the type of industrial zoning is not known, average wastewater flow should be assumed as if for Industrial Heavy (IH) zoning, which is equivalent to 0.38 L/s/ha. However, based on the water demand statistics for industrial areas throughout the Canterbury Region (see *Table 3-1*) average demand varied from 0.02 to 0.11 L/s/ha, which includes IH zones. Therefore, the average design wastewater flow for Industrial General (IG) zoning of 0.15 L/s/ha in the CCC IDS is deemed more appropriate for the proposed extension of the Business 2 Zone. Based on the total area of 10.58 ha of the proposed industrial zone, the average wastewater flow to the WWTP was taken as 1.59 L/s or 137 m^3/d.

Table 3-1 Water demand statistics for industrial areas throughout Canterbury Region

Area	Zone Land Use	No Customers	Area (ha)	Customer Density (Customers/ha)	Total Average Demand (L/s) *	Average Demand per Area (L/s/ha)
Rolleston						
Izone Drive	Business 2	91	44	2.1	2.7	0.06
Christchurch City						
All zones	Industrial General	1627	847	1.9	62	0.07
All zones	Industrial Heavy	1085	1122	1.0	67	0.06
All zones	Industrial Park	8	128	0.1	2.8	0.02
Timaru City						
Washdyke Industrial Area	Industry Heavy	51	150	0.3	16.9	0.11
Canterbury Average Demand Per Area (L/s/ha)						0.07

Wastewater Load
The domestic load on the Leeston WWTP is based on the unit per capita loading values used in the previous WSP Opus report (August 2017, reference 3-38981.01), shown in Table 3-2, which is typical for domestic wastewater.

Table 3-2 Unit per capita loading assumed for domestic wastewater from Leeston

Parameter	Unit Per Capita Loading
BOD	60 g/hd
TSS	70 g/hd
TKN	10 g/hd
NH_3	7.5 g/hd
TP	5 g/hd

For the LEE3 site, the concentration of the wastewater from the site was calculated using figures in Table 3-2, assuming an average population equivalent flow of 140 L/hd/d. This value is lower than the 180 – 200 l/hd/d normally used, as it is assumes the population equivalent flow from the LEE3 site will be lower, disregarding flow normally associated with bathing and laundry. Using the above assumptions, the wastewater concentration from the LEE3 site was calculated (see Table 3-3). The values are deemed conservative and can be adjusted when more information becomes available on the nature and type of industry planned for the LEE3 site.

Table 3-3 Estimated wastewater concentrations from the LEE3 site.

Parameter	Concentration
BOD	429 mg/L
TSS	500 mg/L
TKN	86 mg/L
NH_3	54 mg/L
TP	36 mg/L

3.4.2 Future Irrigation Loading

In the WSP Opus August 2017 Report it was shown that nitrogen was the limiting factor for the Leeston WWTP and the irrigation field, therefore only total nitrogen load application was considered in this evaluation. For the LEE3 site redevelopment the additional nitrogen load was estimated at 9.8 kg N/d. The domestic and the existing commercial areas nitrogen load was estimated from the flow forecast (Figure 3-2) and the unit per capita loading of 10 gN/hd (see Table 3-2).

The figures below provide the effect of total nitrogen load to the Leeston WWTP and the effect of load on the irrigation areas. Allowance is included for hay removal based on typical performance of 250 kg/ha/yr removed, so allowing a maximum nitrogen load of 450 kg/ha/yr. This allows a daily average load of 43.5 kg/d over 35.3 ha (extended irrigation field area).

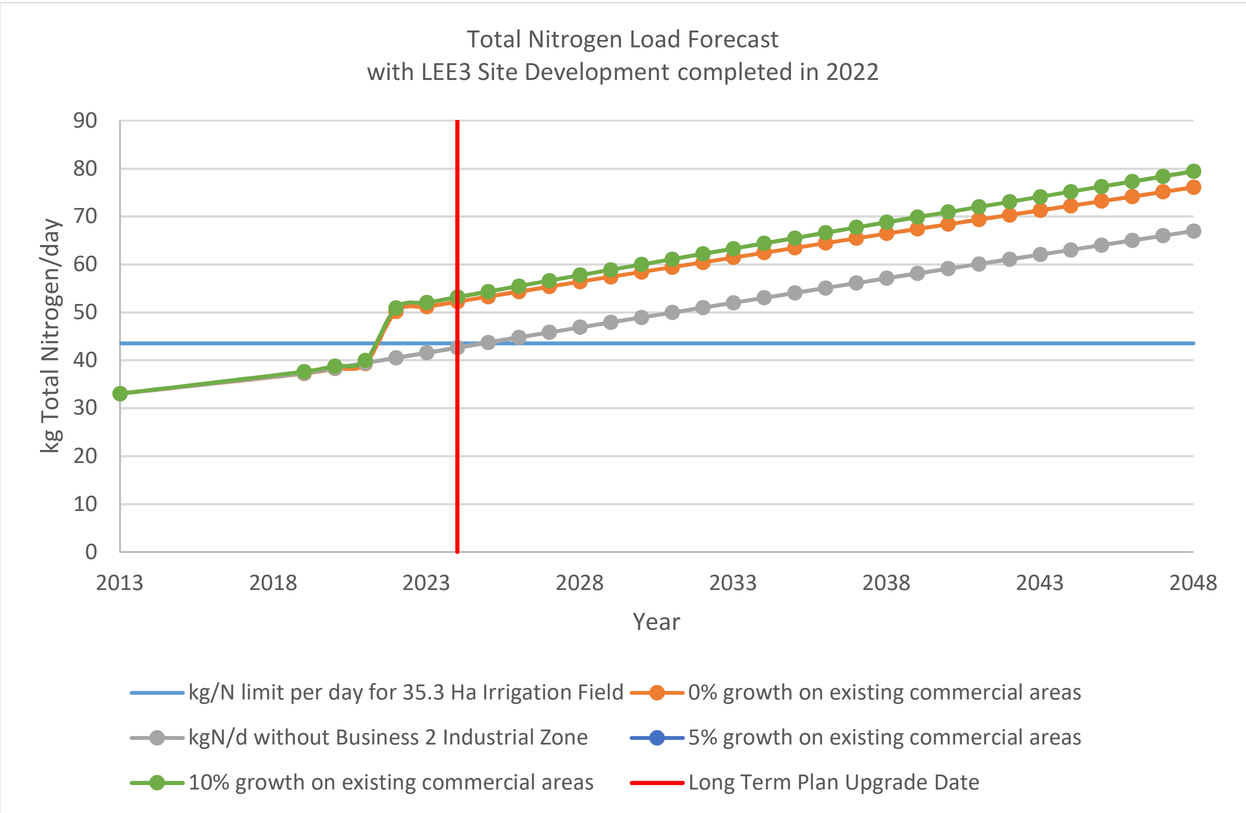


Figure 3-9 Total nitrogen load forecast with LEE3 site development completed in 2022

Figure 3-9 shows the impact of the LEE3 site redevelopment would have on the Leeston WWTP and irrigation field if the wastewater flows from the site would come online in 2022. Growth from the existing commercial areas was varied from 0% to 10%. The initial growth estimate that was used in the August 2017 report is also shown (kgN/d without Business 2 Zone), along with the recommended upgrade date of 2024. From Figure 3-9 the LEE3 site development will have a substantial impact on the total nitrogen load. The same year the wastewater flows from the site comes online (2022), the maximum allowable nitrogen load will be exceeded.

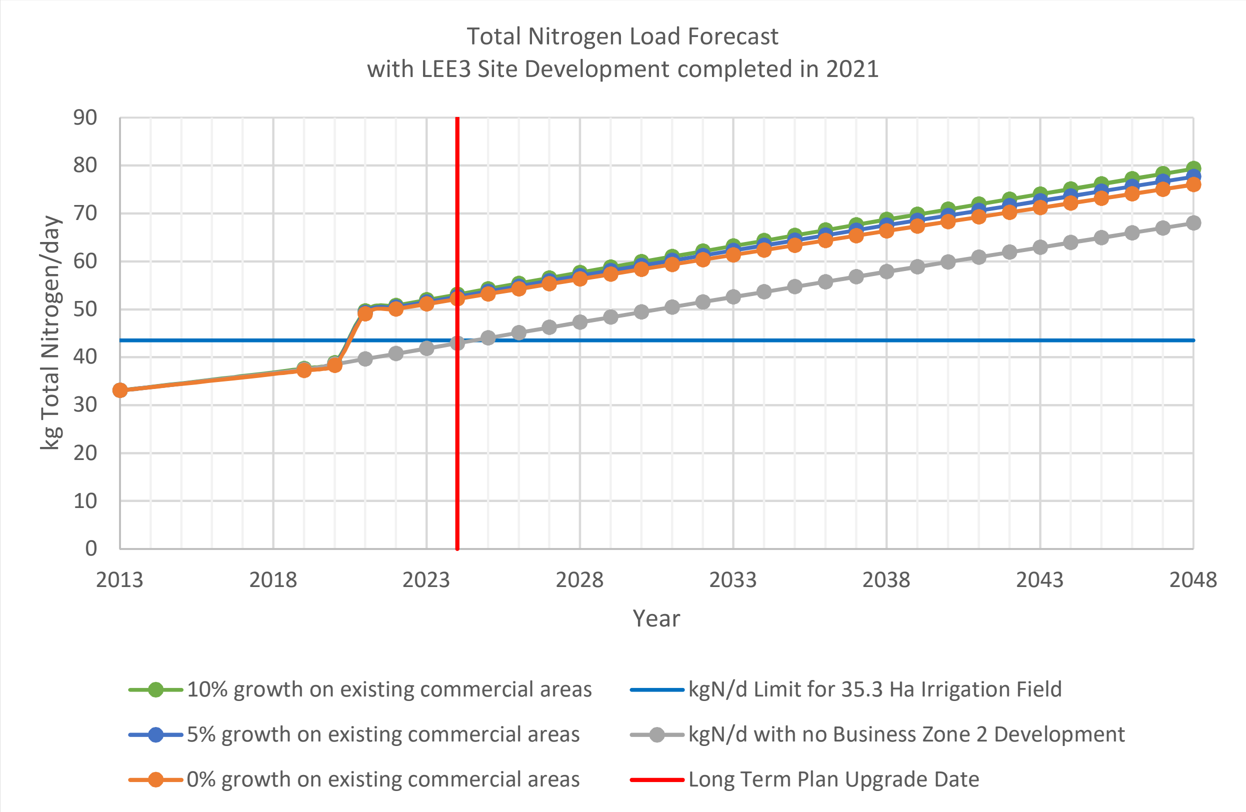


Figure 3-10 Total nitrogen load forecast with LEE3 site development completed in 2021

In Figure 3-10, the completion date of the LEE3 site was assumed to be one year earlier i.e. 2021. As expected, the year in which the maximum allowable nitrogen load is exceeded moves one year earlier. Figure 3-9 and Figure 3-10 shows that the impact of the existing commercial areas is marginal when compared to the impact the LEE3 site redevelopment will have on the Leeston WWPT and irrigation field.

In Figure 3-11 the impact of protracted completion/development of the LEE3 site redevelopment is shown. To illustrate this, it was assumed that the first half of the wastewater flow (69 m^3/d) and associated load (4.9 kg N/d) comes online in 2022, with the remainder in 2023 (137 m^3/d at 9.8 kg N/d).

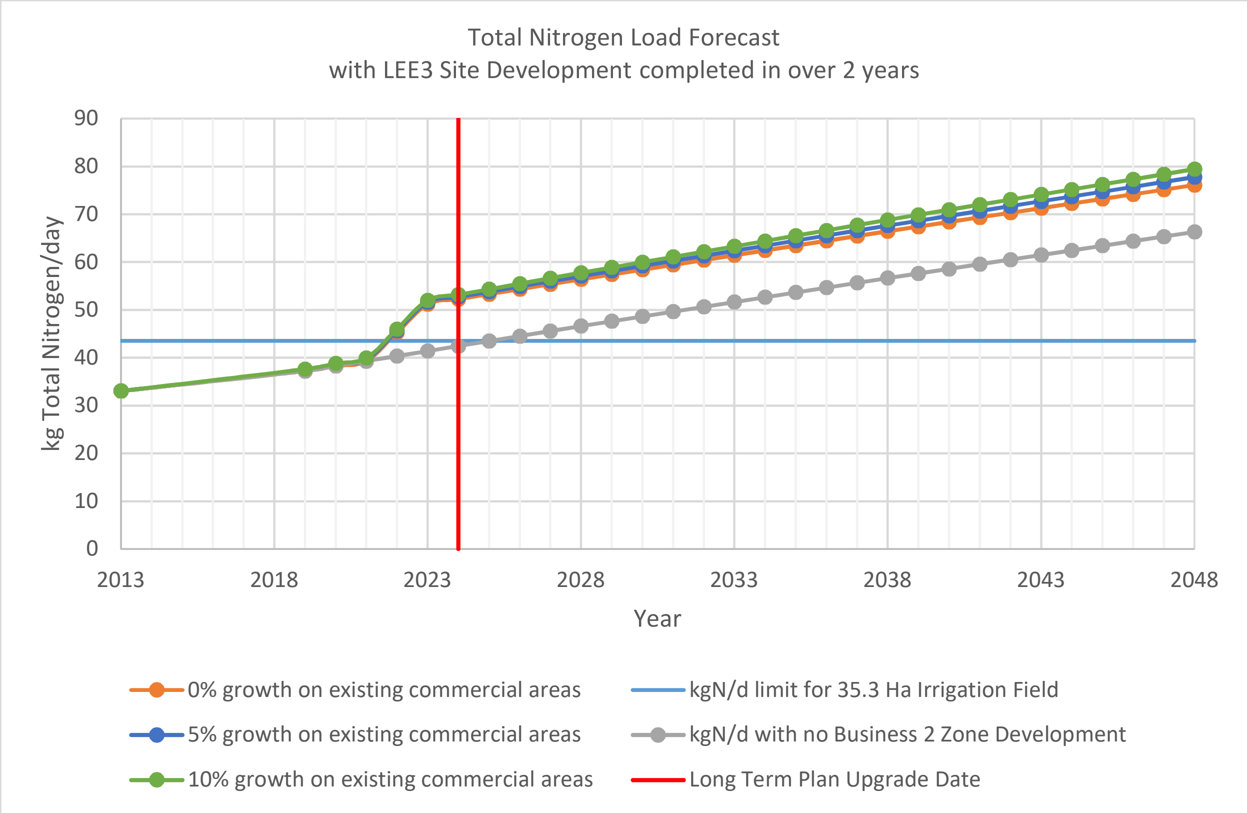


Figure 3-11 Total nitrogen load forecast with LEE3 site development completed over 2 years.

From Figure 3-11 it can be seen that even if the site is developed over a two-year period, the maximum allowable nitrogen load is still exceeded in the first year flow is generated from the LEE3 site i.e. 2022.

From Figures 3-9 to 3-11 it is evident that the 35.3 ha irrigation field will exceed its capacity when the LEE3 site wastewater flows are received at the Leeston WWTP. It is therefore recommended that the plant upgrade is planned to align with the LEE3 site development. If the LEE3 site development is planned for after 2024, the recommendation of the August 2017 report remains, and the plant upgrade should be planned for 2023/2024 to maintain compliance with the consent.

3.5 Stormwater

3.5.1 Hydrological Setting

The site is located over shallow to moderately deep poorly drained clay / silt with high groundwater present. The area is prone to waterlogging during winter months with prolonged rainfall and has limited surface drainage when saturated.

The land falls in a south east direction towards Lake Ellesmere and is relatively level (typically 1:400 longitudinal slope). Lake Ellesmere is the main receiving environment downstream.

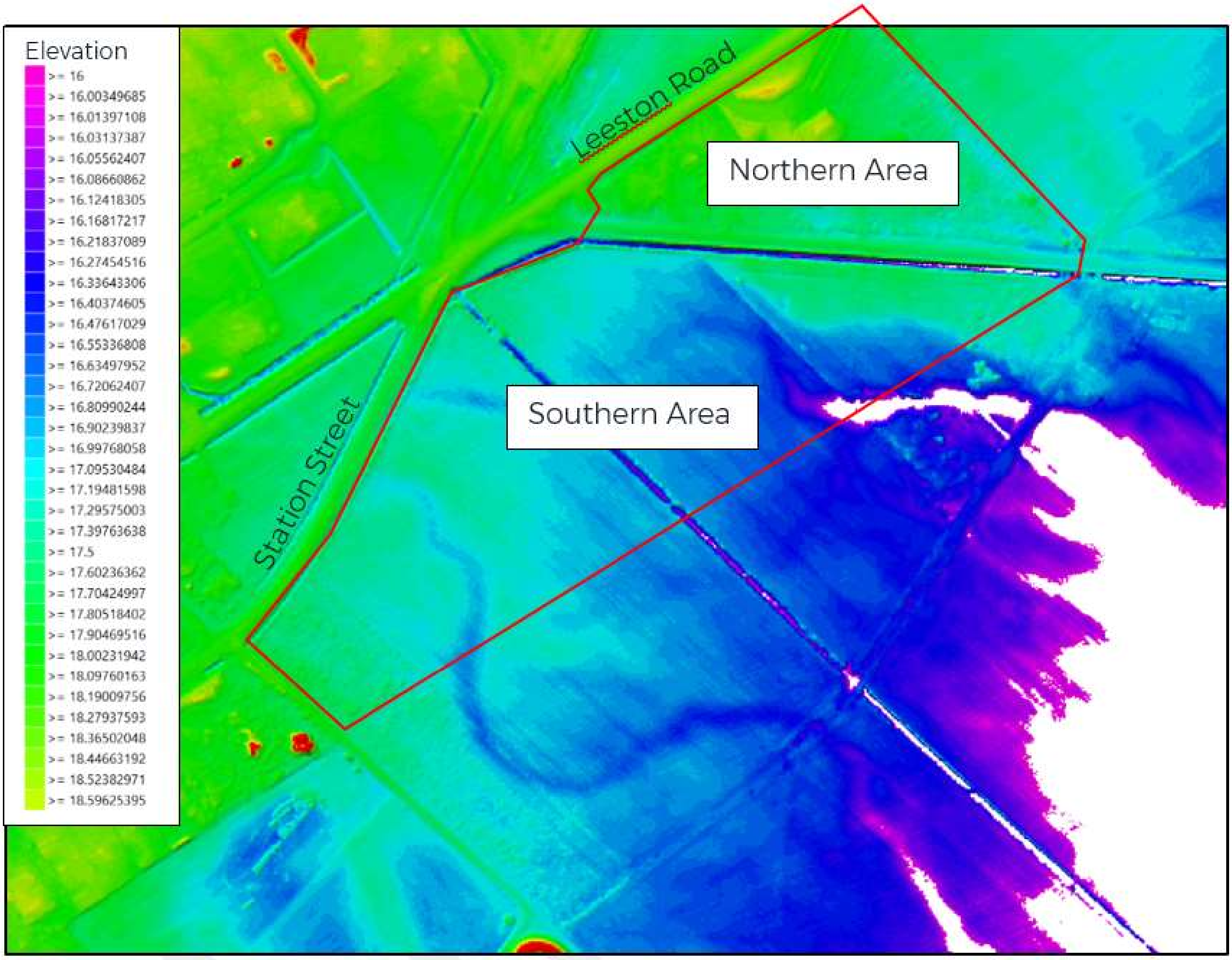


Figure 3-12 Surface elevations and site boundary (red).

3.5.2 Flood Risk

This report specifically excludes flood risk. This will be considered separately to this report by SDC.

3.5.3 Likely Development

The site is proposed to be re-zoned as industrial land use. The development is expected to be largely impervious in nature with large roof areas, access roads, carparking, yard areas with limited landscaping or greenspace.

SDC's current approach to stormwater management is that each area of development provides its own on-site stormwater management system (as opposed to SDC providing a catchment scale system).

Given the potential range of land uses within this industrial area, this allows for the treatment systems to be targeted to the specific land use of each site and would provide less restriction on the activities undertaken on the site.

Figure 3-13 Overview of the proposed development area (magenta boundary) contours shown in red.

Part of the proposed development area is already used for industrial activities but could be redeveloped or intensified in the future.

3.5.4 Hydraulic Neutrality

Run-off rates are expected to vary considerably, subject to antecedent rainfall and groundwater elevations. In terms of highest run-off, this is likely to occur during late autumn, through winter and into spring when groundwater levels are elevated, and soil moisture is higher / evapotranspiration losses are lower.

During significant rainfall events that result in localised or downstream flooding, the run-off rates from the area are expected to be relatively high due to limited soakage. Hence the required volume for attenuation of flows will likely be lower than would be required if there to be located over free draining soils further inland. However, these systems will still require a reasonable land footprint due to the limited fall of the land.

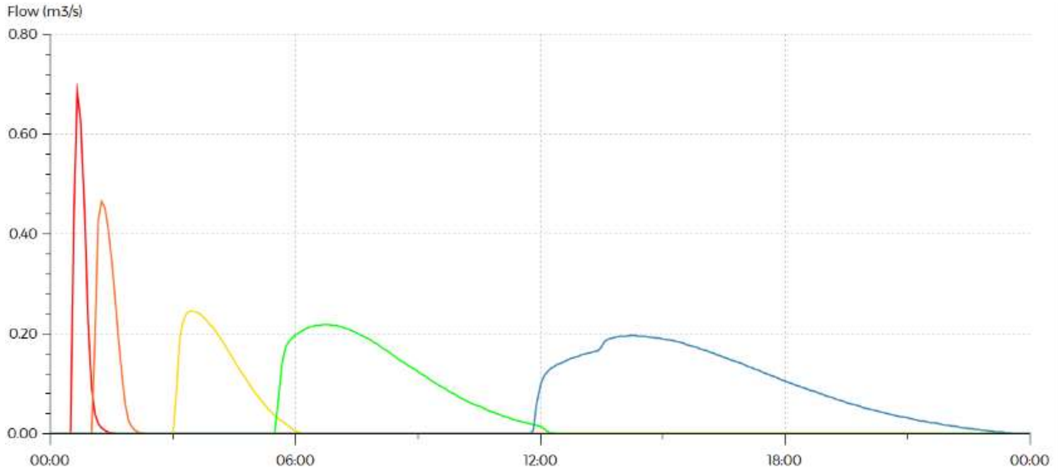
To assess the likely greenfield discharge rates a hydrological model of the site was developed. The run-off volume and routing model utilises the Horton model. Initial losses were calculated based on an average slope of 0.003 (m/m) in accordance with the Wallingford Procedure for pervious surface.

Run-off volume was calculated using the following parameters:

- Initial infiltration rate = 25mm/hr
- Terminal infiltration rate = 1mm/hr
- Time to saturation = 2 hours

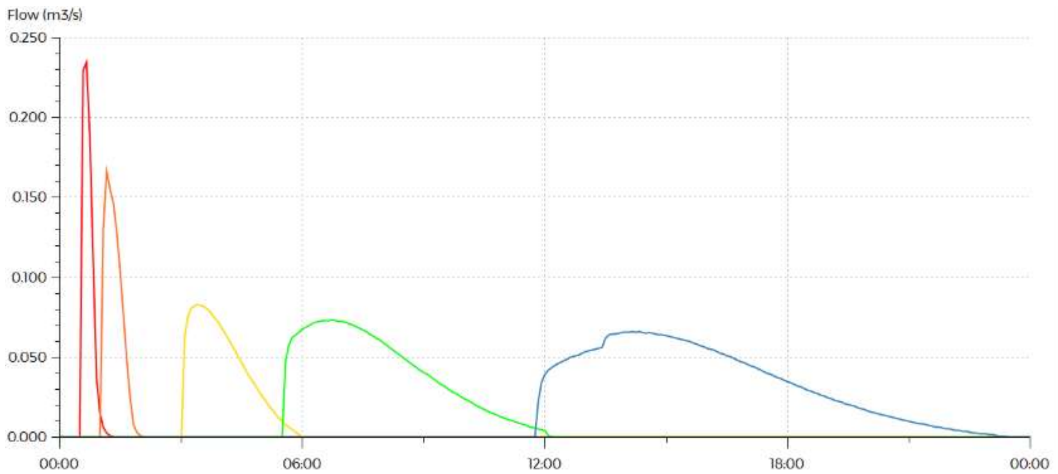
The run-off was then routed using a linear reservoir model with n = 0.15.

The 2% AEP rainfall design values were based on HIRDS V4 with the SDC Design Rainfall Leeston scaling factor applied. The 2% AEP rainfall was then translated to a hyetograph using a parabolic fit in-line with the HIRDSV4 normalised hyetographs as outlined in the HIRDS V4 Technical Report.



	Flow	
	Min	Max
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 1hr	0.000	0.685
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 2hr	0.000	0.466
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 6hr	0.000	0.246
...odel group>Runs>Leeston Hyd!>Leeston - 2% AEP 12hr	0.000	0.218
...odel group>Runs>Leeston Hyd!>Leeston - 2% AEP 24hr	0.000	0.196

Figure 3-14 Estimated greenfield run-off for the Southern area



	Flow	
	Min	Max
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 1hr	0.000	0.235
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 2hr	0.000	0.167
...Model group>Runs>Leeston Hyd!>Leeston - 2% AEP 6hr	0.000	0.083
...odel group>Runs>Leeston Hyd!>Leeston - 2% AEP 12hr	0.000	0.073
...odel group>Runs>Leeston Hyd!>Leeston - 2% AEP 24hr	0.000	0.066

Figure 3-15 Estimated greenfield run-off for the Northern area

The hydrological parameters used should be reviewed in terms of the existing Leeston hydraulic flood model for appropriateness but should provide an acceptable approximation of greenfield runoff.

The following figures show the increase in runoff post-development compared with the greenfield runoff.

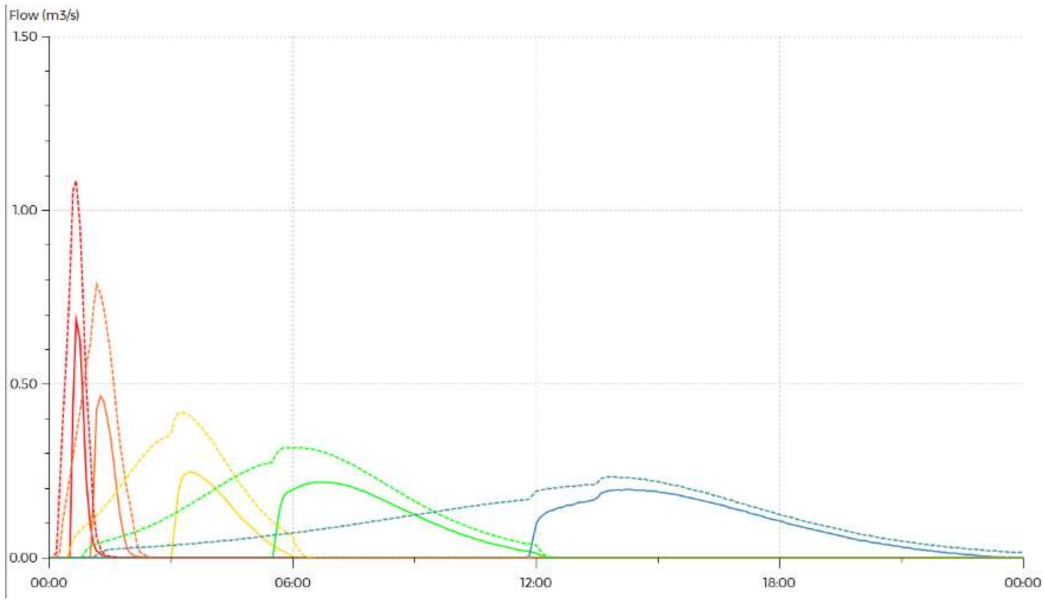


Figure 3-16 Estimated greenfield flow compared with post-development flow (Southern area)

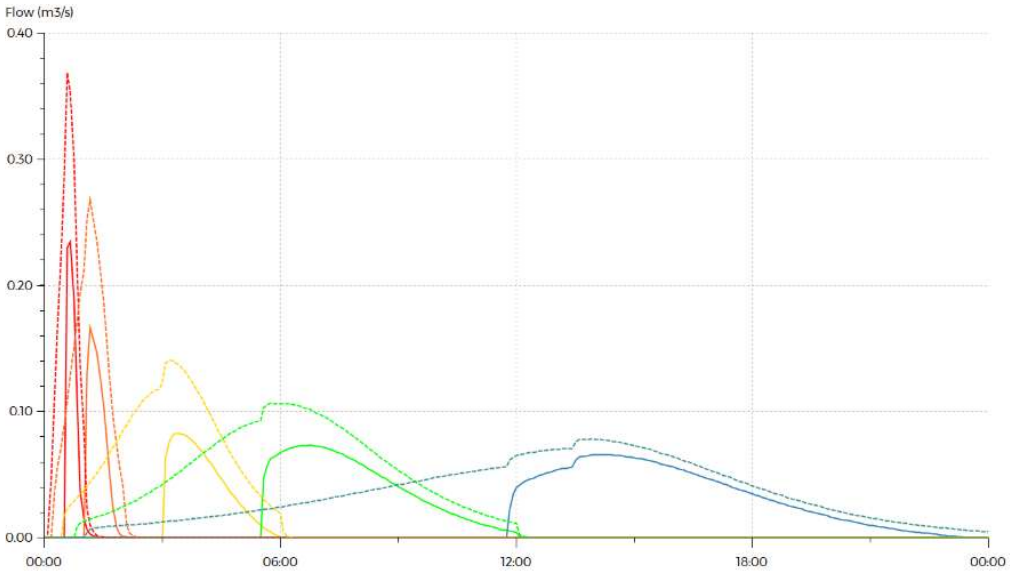


Figure 3-17 Estimated greenfield flow compared with post-development flow (Northern area)

Figure 3-16 and Figure 3-17 show that the developed site doesn't significantly increase peak discharges but does significantly increase the volume of stormwater discharged. This is because soakage losses have been significantly reduced in the developed site.

When the assumed post-development flow is overlaid with the estimated greenfield flow, the following is apparent:

- The volume of run-off is significantly increased. Post development, less water is ponding or infiltrating into the site's soils.

- The onset of flow is much earlier as impervious surfaces have minimal losses and respond to rainfall quickly.

- The difference in peak flow is greatest during shorter duration higher intensity events, when the greenfield losses due to surface storage and infiltration are higher relative to the rainfall depth.

- The difference in peak flow is reduced as the duration of storm increases. This is due to saturation of the soil and higher rates for greenfield run-off proportionally.

3.5.5 Conceptual Attenuation Design

To assess mitigation requirements for the expected increase in peak flow from the development, the following conceptual design has been modelled:

- A first flush basin with extended release time. This assumes full capture of a 15 mm water quality (WQ depth) event for slow release which could include soakage to ground (when feasible). These utilise a small diameter orifice at invert level for the slow release of water. For the purpose of the modelling, no soakage was allowed for, as at certain times of the year it may be severely limited.

- Additional flood attenuation for 10% AEP and 2% AEP events with allowance for climate change (RCP 6.0 2089-2100) to reduce the peak discharge to greenfield (or lower). The area provided for attenuation is equivalent to 15% of each lot area.

- WQ volume = 15 mm * site area impervious area (80% of the lot area)

- The invert area = WQ volume * 4 (250 mm depth)

- Top area = site area * 15%

- Basin depth = 0.5m (due to limited fall across the area)

- V-notch outlet 0.25m above the basin invert for events that exceed the WQ volume

The results of the conceptual analysis are presented below.

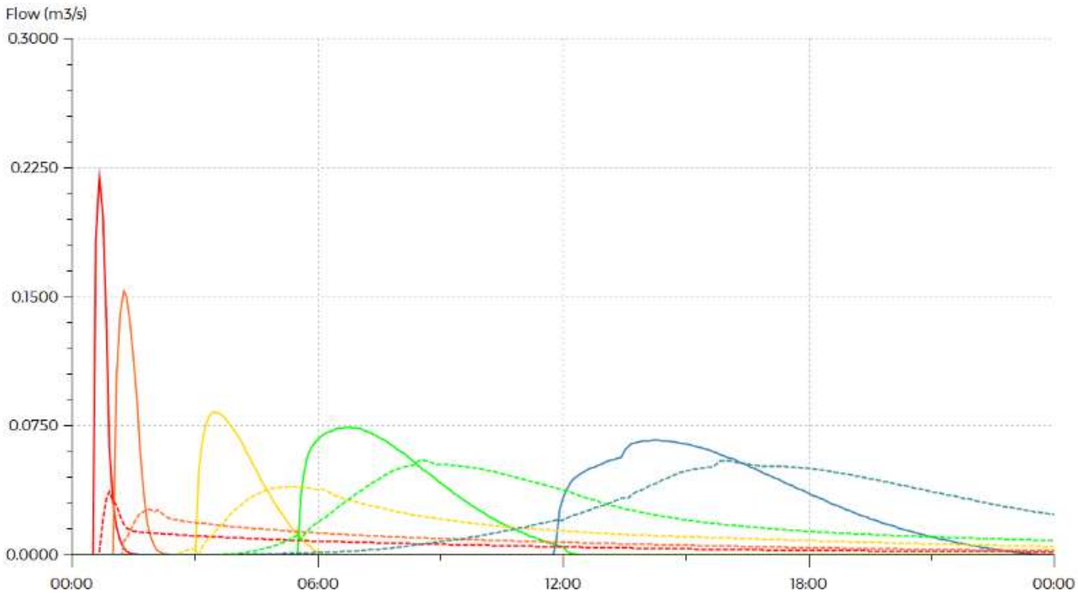


Figure 3-18 2% AEP pre (solid line) and post (dashed line) hydrographs for comparison

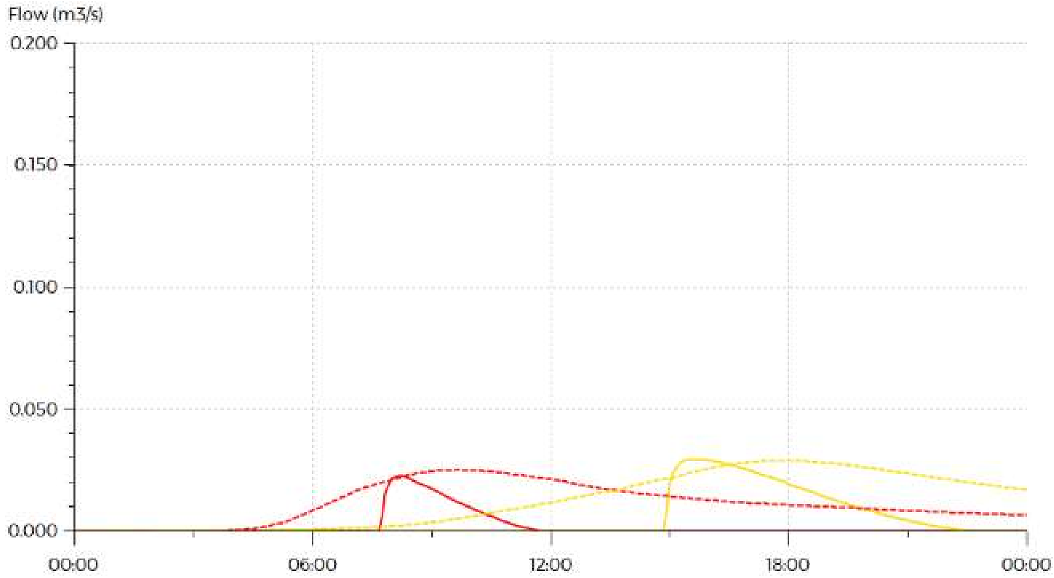


Figure 3-19 10% AEP pre (solid line) and post (dashed line) hydrographs for comparison

Note that only the 12-hour and 24-hour storms were assessed for 10% AEP as these are the worst-case scenarios for basin sizing.

The above results confirm that a shallow stormwater attenuation basin incorporated first flush treatment should be able to achieve hydraulic neutrality using 15% of the lots overall area, assuming a maximum imperviousness of 80%. The conceptual design can be scaled up or down depending on the eventual development lot sizes.

3.5.6 Lake Ellesmere

Whilst the increase in impervious area will reduce the losses to groundwater and thus increase the volume of stormwater discharge, it would be discharged over a longer period, allowing additional losses from the Lake downstream over the extended time period.

Currently all water, whether infiltrating or discharged via surface flow, is expected to end up in Lake Ellesmere. Hence the additional volume is not expected to adversely affect the lake or flooding associated with the lake. The extended release time should also better mimic the more in-direct path that sub-surface water takes to laterally discharge into the existing land drainage.

3.5.7 Proposed Conveyance

Northern Section

The general fall of the land is to the existing roadside drain on Volckman Road as shown on Figure 3-12. The roadside swale appears to have a fall of approximately 1:230 based on SDC LiDAR information and could be maintained for conveyance, as well as further treatment of flows discharged from the site.

At the eastern end of the swale, an inlet structure and culvert would need to be provided to divert the flow into the drain on the opposite side of the road. An intermediate inlet and culvert part way along the swale may also be required at the time of design once better topographical information is available or to keep the swale flow depth to an acceptable level.

Land fall, based on the SDC LiDAR information, appears sufficient to achieve fall at all locations to the existing road side drain.

Any accessways would need to be designed to work with the swale so as to not compromise conveyance.

Southern Section

An existing drain runs through this section of land, as well as two historic flow paths (visible in the LiDAR, see Figure 3-12) which were likely the natural flow path prior to modification of the drainage.

These secondary flow paths would likely be filled or modified by any development of the site. Hence, it is proposed that all flows are directed to swales along the south-eastern boundary. These would collect treated stormwater and overland flow during exceedance events and direct them to the existing drain.

Land fall, based on the SDC LiDAR information, appears sufficient to achieve fall at all locations to the existing road side drain. Whilst the slope is quite flat, conveyance at these slopes is feasible if frictional losses are kept low enough through generous sizing.

With provision of stormwater attenuation systems on-site, the design flow rate for these swales will also be relatively low i.e. 22 L/s/ha.

Figure 3-20 Average slopes and proposed drainage layout

4 Conclusions and Recommendations

The following sections present key findings and recommendations based on the above infrastructure assessment.

4.1 Water

The existing water supply network has marginal capacity to support the addition of the LEE3 commercial / industrial development. Up to 4 L/s flow can be provided to the LEE3 site but this results in significant pressure drops to all Leeston customers and exacerbates existing capacity and velocity issues in the network.

By contrast, the upgraded network (to meet 2047 growth) can easily accommodate the LEE3 development, although if significant water users are based at this site, the water supply capacity of the network should be reassessed. The new well or wells on Leeston-Dunsandel Road and trunk main upgrades on Pound Road and Cunningham Street will be required to service the development. It is not possible to service LEE3 in the interim by simply upgrading local pipes.

4.2 Wastewater

The following conclusions and recommendations were made based on the wastewater reticulation scenarios modelled:

- In response to the 5-year storm event, there are existing sewer capacity issues in the Cunningham Street sewer, which is intended to provide wastewater servicing for the LEE3 site.
- The extent of capacity issues in the Cunningham Street sewer is predicted to increase because of expected population growth in the contributing area. However, model results indicate that flows from the LEE3 site will not have a substantial effect in exacerbating capacity issues in this sewer.
- Previously recommended sewer upgrades are predicted to have sufficient capacity to convey future wastewater flows with the inclusion of the LEE3 site. These recommendations include upsizing the sewer on Cunningham Street from Pennington Street to High Street with 225mm diameter pipe, and from High Street to Station Street PS with 300mm diameter sewer. It was also recommended that the capacity of the Station Street PS be increased to accommodate the anticipated future peak flow of 67.1 L/s.

4.3 Wastewater Treatment

The following conclusions are recommendations were made based on the WWTP capacity assessment completed above.

- A capital scheme improvement should be timed to coincide with the completion of the LEE3 site development or 2024, whichever comes first, to continue to meet the current consent conditions.
- The 100% Activated Sludge Plant proposed in the WSP Opus August 2017 Report will have to increase by approximately 10% to cater for the additional load due the LEE3 site development (based on nitrogen loading).
- The capital cost impact of the LEE3 site development on the Leeston WWTP would need more detailed sizing and an updated cost evaluation.

- Routine monitoring should be undertaken to establish background flow and load data essential for future plant design.
- As a minimum, monthly composite crude sewage sampling and continuous flow monitoring of influent should be commenced to provide suitable data for design of the future treatment plant.

4.4 Stormwater

The stormwater assessment has demonstrated that the site can be serviced in terms of conveyance and mitigating downstream effects, through achieving hydraulic neutrality and providing site specific first flush treatment systems, subject to the following conditions:

- Maximum 80% imperviousness per lot
- Appropriate design and review of on-site systems for correct design and hydraulic assessment
- At least 20% of each lot is set aside for landscaping and stormwater management
- Appropriate primary drainage and secondary flow paths are provided on each lot to convey flows to the stormwater management areas
- Appropriate flow paths are provided to collect stormwater discharges from each lot and direct them to an appropriate discharge point
- Appropriate assessment and management of flood risk and pluvial run-on is undertaken (not part of the scope of this work)
- An easement is secured to protect the existing drain and allow for maintenance access.

5 References

Springer, A. (2017) Leeston WWTP Options Report – Revised Populations Options, August 2017, reference 3-38981.01, Auckland.

Appendix A
Memorandum: Assumed Flows for Leeston Industrial Area

Memorandum

To	Murray England
Copy	Sue Harrison
From	Mark Groves, Mark de Lange and Debbie Weeds
Office	Christchurch Environmental Office
Date	31 May 2019
File	3-C1831.01
Subject	Assumed Flows for Leeston Industrial Area

1 Introduction

The Selwyn District Council (SDC) retained WSP Opus to confirm the three water infrastructure needs to service the extension of the Business 2 (Industrial) Zone in the south-east of Leeston Township. The Council is considering extending the Business (Industrial) Zone in the south-east of Leeston Township to include additional zoned land within the Proposed District Plan. This memorandum outlines assumptions that will applied in assessing infrastructure needs for the proposed industrial development for each of the three waters assessments. Figure 1 shows the boundary of the proposed industrial zone, which has a total area of 10.58Ha.

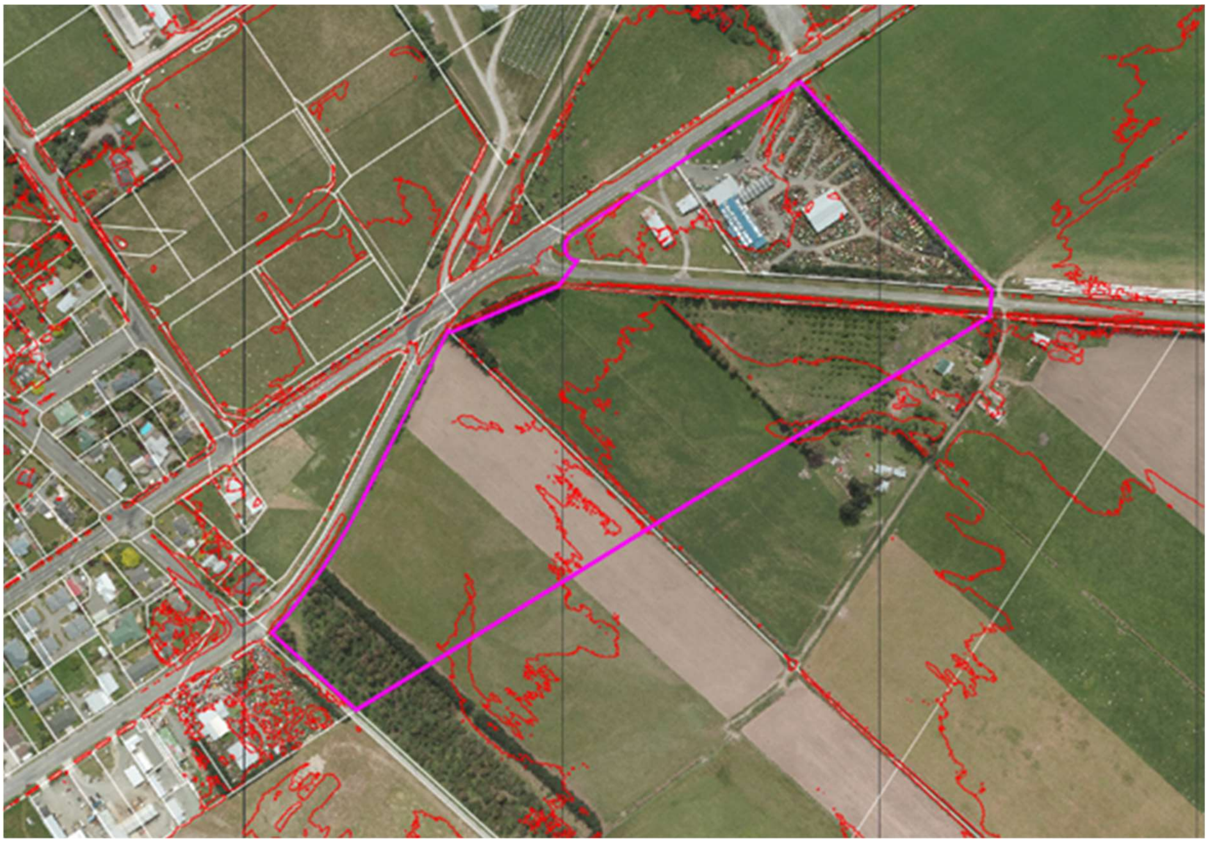


Figure 1 Boundary of Leeston Industrial Development

2 Stormwater

The following section outlines the assumptions that will be applied in assessing stormwater management infrastructure needs for the proposed development.

- Maximum 80% impervious coverage (allowing space for stormwater management and landscaping)
- On-site stormwater treatment systems will be provided as part of each sites development (approximately 15% of the site area to be set aside for stormwater management due to the limited fall across the site) - as agreed at the prior workshop with SDC
- The stormwater treatment systems will limit the discharge to the greenfield equivalent
- Maximum area of hard-stand discharging to the existing road corridor associated with each access does not exceed 100m^2 (provides for the access to the site)
- One access point per site onto the existing road corridor
- No new road infrastructure will be provided by SDC within the development area (to be provided by developers as per master plan scheme)

Figure 2 presents the overland falls across the site that will be used in assessing stormwater drainage.

Figure 2 Average Site Falls

3 Water Demand

Water and wastewater design flows for the industrial area were derived concurrently to provide agreement when assessing the two systems. Design flows for the industrial area have been determined based on:

- Canterbury-wide water usage data (Industrial Heavy)

- Christchurch City Council Infrastructure Design Standard (CCC IDS, 2016)
- Selwyn District Council Water Supply Code of Practice (SDC CoP, 2012)

Table 3-1 presents water usage statistics for key industrial zones in the Canterbury region.

Table 3-1 Water Demand Statistics for Industrial Areas Throughout Canterbury Region

Area	Zone Land Use	No Customers	Area (Ha)	Customer Density (Customers/Ha)	Total Average Demand (L/s)	Average Demand per Area (L/s/Ha)
Rolleston						
Izone Drive**	Business 2	91	44	2.1	2.7	0.06
Christchurch City						
All zones	Industrial General	1627	847	1.9	62	0.07
All zones	Industrial Heavy	1085	1122	1.0	67	0.06
All zones	Industrial Park	8	128	0.1	2.8	0.02
Timaru City						
Washdyke Industrial Park	Industry Heavy	51	150	0.3	16.9	0.52
Canterbury Average Demand Per Area (L/s/Ha)						0.15

The average water demand for the industrial areas assessed was 0.15 L/s/Ha, with a peak average value of 0.52 L/s/ha (Washdyke). Note that Washdyke Industrial Park in Timaru contains many high-water users, including a brewery and vegetable washing facilities.

The SDC CoP (Part 7: Water Supply) contains limited advice about designing for industrial and commercial and recommends a minimum peak flow rate of 1.0 L/s/ha which far exceeds any of the real-life data.

The CCC IDS (Part 7: Water Supply) contains a chart whereby business zone design flow rates can be estimated based on the number of business zone sites. This design methodology is not suitable for this project because there is no subdivision plan showing the number of allotments.

As a result, water demand and wastewater flows have been based on Table 2 in the CCC IDS (Part 6: Wastewater Drainage) showing average sewer flows for industrial and commercial areas.

Table 3-2 Commercial and Industrial Unit Average Sewer Flow Values (CCC Infrastructure Design Standard, 2016)

Zoning	Unit ASF (L/s/ha)
Commercial Local (CL)	0.09
Commercial Core (COR)	0.15
Industrial General (IG) - suburban	0.15
Industrial General (IG) - inner city	0.38
Industrial Heavy (IH)	0.38
Commercial Local (CL)	0.09

Based on the data presented in Table 3-1 and Table 3-2, it was concluded that average water use of 0.15 L/s/ha represents a realistic water demand and wastewater flow for an industrial area.

Please note the following:

- CCC IDS recommends a design flow of 0.38 L/s/ha for industrial heavy zones but this figure appears to be slightly conservative when compared to real-life water usage for Canterbury-wide industrial heavy zones.
- Significant wet industries can have very high water requirements that far exceed the per hectare design flows shown in Table 3-2. Individual customers at Washdyke Industrial Park extract average flows of up to 42 L/s year round. If the site needs to accommodate a significant water user then the infrastructure capacity will need to be reassessed.

A peaking factor of 1.8 will be applied to the average water flow. For the proposed industrial area of 10.58 ha, the peak design demand that will be applied in the model is 2.84 L/s.

4 Wastewater Generation

The following section outlines the assumptions that will be applied in determining wastewater servicing needs for the proposed development area.

- As per stormwater assumptions 80% of the industrial area is assumed to be impervious cover. In determining wastewater flows, it will be assumed that all impervious cover is made up of building area. This will provide a conservative value for determining wastewater flows on the basis gross floor area.
- Wastewater generation rates will be determining using an average design flow rate for general industrial of 0.15 L/s/Ha as outlined in the CCC's Infrastructure Design Standard This is the same design value that will be applied to determine water servicing needs and assumes that 100% of water supplied to the area is discharged to the wastewater collection system.
- A peaking factor of 5 will be applied to the average flow to allow for inflow and infiltration. The peak wastewater discharge from the site will be loaded in the model as a constant flow from the site.
- Based on the site area of 10.58 ha the wastewater flow applied in the model will be 7.94 L/s.
- The load point to the network is assumed to be the 150mm diameter sewer on Cunningham Street (upstream of Station Street PS). Previous modelling work completed in Leeston recommended that this sewer be upgraded to a 300mm diameter pipe.

5 Wastewater Treatment

The following section outlines the assumptions that will be applied in determining wastewater treatment requirements for the proposed development area.

- Regarding Wastewater Flows from the industrial zone we will rely on assumptions made for Wastewater conveyance as stated above.
- Regarding Wastewater Flows from the rest of Leeston that come into the WWTP we will maintain the assumptions stated on "Leeston WWTP Options Report. Revised Population Options" prepared by Opus dated 9 August 1027
- According to those data, the population on Leeston will go from 3,304 (on 2013) up to 6,631 (on 2048).
- Expected flows from population were defined as 594 m^3/d (2013) and 1,259 m3/d (2048).
- Provision for an equivalent 20% population associated to industrial activity was made on the report raising the flow figures to 713 m^3/d and 1511 m^3/d (initial and future, 2048)

- Water Quality of the existing population was estimated (on the report) on the base of an average production per habitant (60 g/BOD; 70 g TSS; 12 g TKN; 7.5 g NH_3 and 5 g TP). Similar figures will be maintained for the general population and general industrial areas on Leeston.

- For the new Leeston industrial area to be developed (while we get additional information about the expected type of industries on the area) we will assume similar figures of daily contaminant production based on the average flow for each solution with an average of 180 l per equivalent habitant per day resulting in the following average concentrations (mg/l):

Parameter	Concentration Average (mg/l)
BOD	350
TSS	390
NH3	42
TKN	67
TP	27

- For the Required Effluent Quality for wastewater treatment we will consider the values stated on the report (taken from the actual consent):

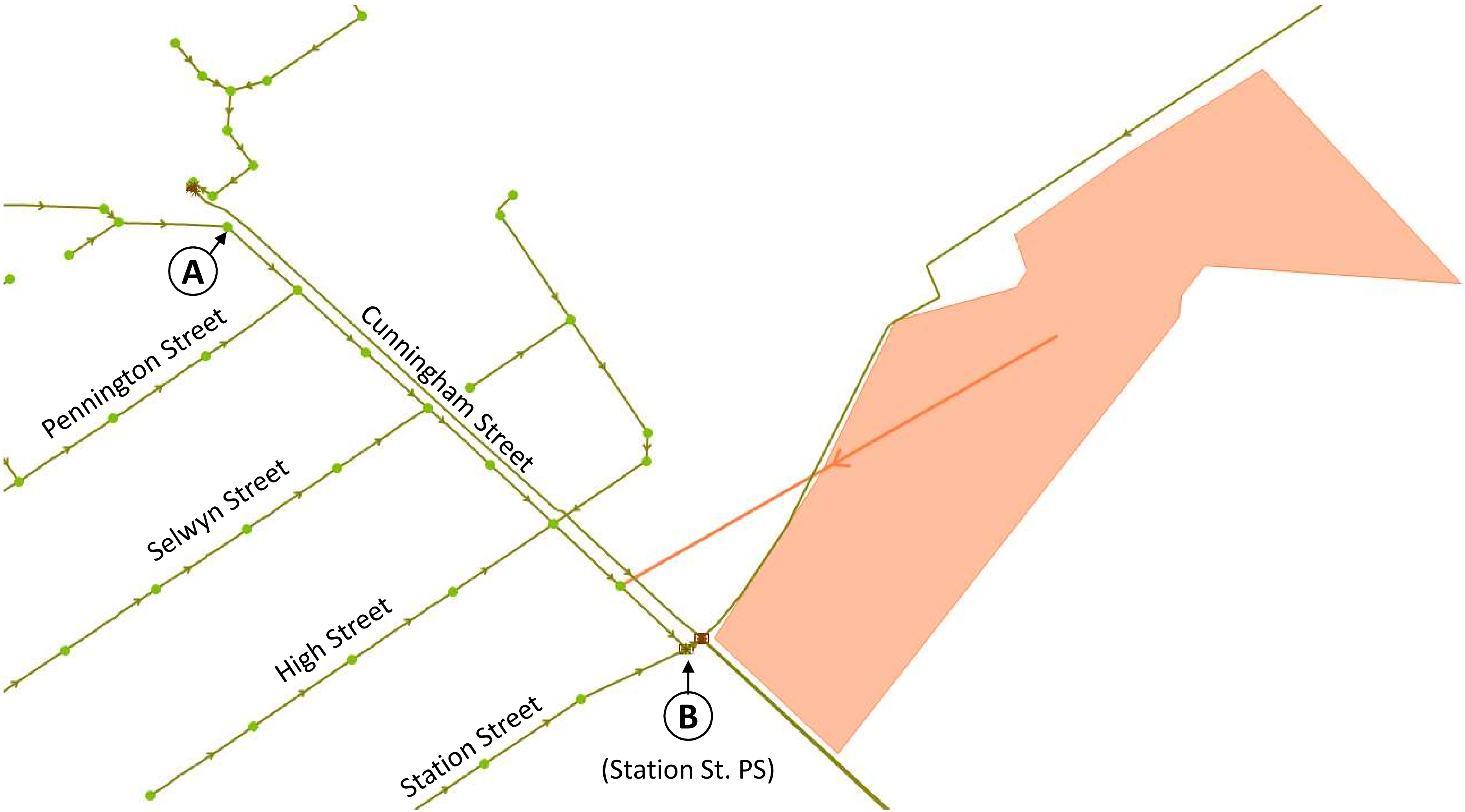
Parameter	Average	Unit
BOD	10	mg/l
TSS	15	mg/l
NH3	5	mg/l
TN	43.6	kg/d
TN	28.9	mg/l
TP	5.8	kg/d
TP	3.8	mg/l

6 Next Steps

Following approval of the assumptions detailed above we will proceed with the assessment of infrastructure needs for the proposed industrial area. Please do not hesitate to contact the project team should you have any questions or require further clarification.

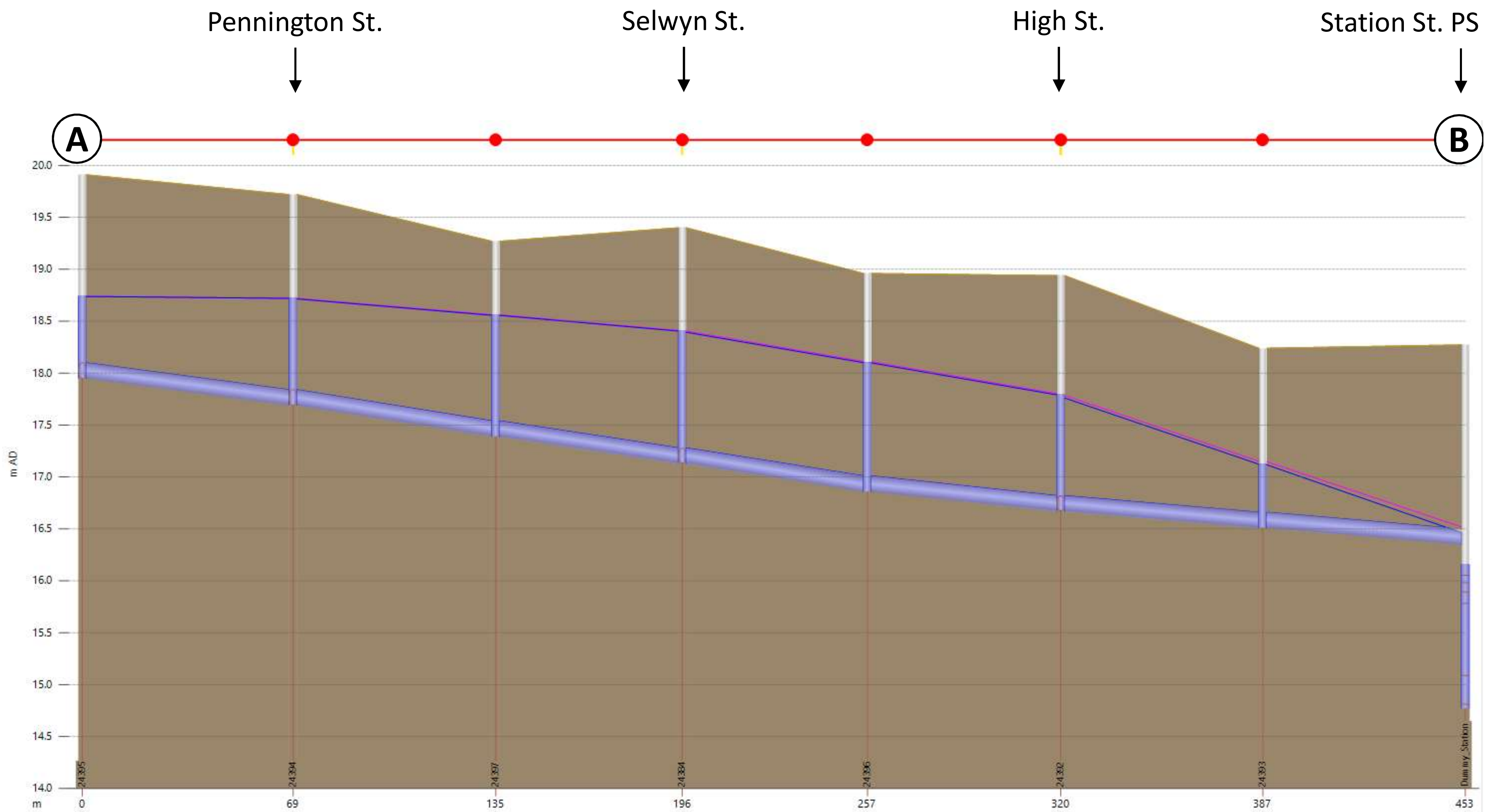
Appendix B
Wastewater Sewer Maximum Hydraulic Grade Line Profiles

Reference Plan View for Sewer Profiles



Scenario 1: Cunningham Street Maximum HGL Profile

5-Year Storm Event

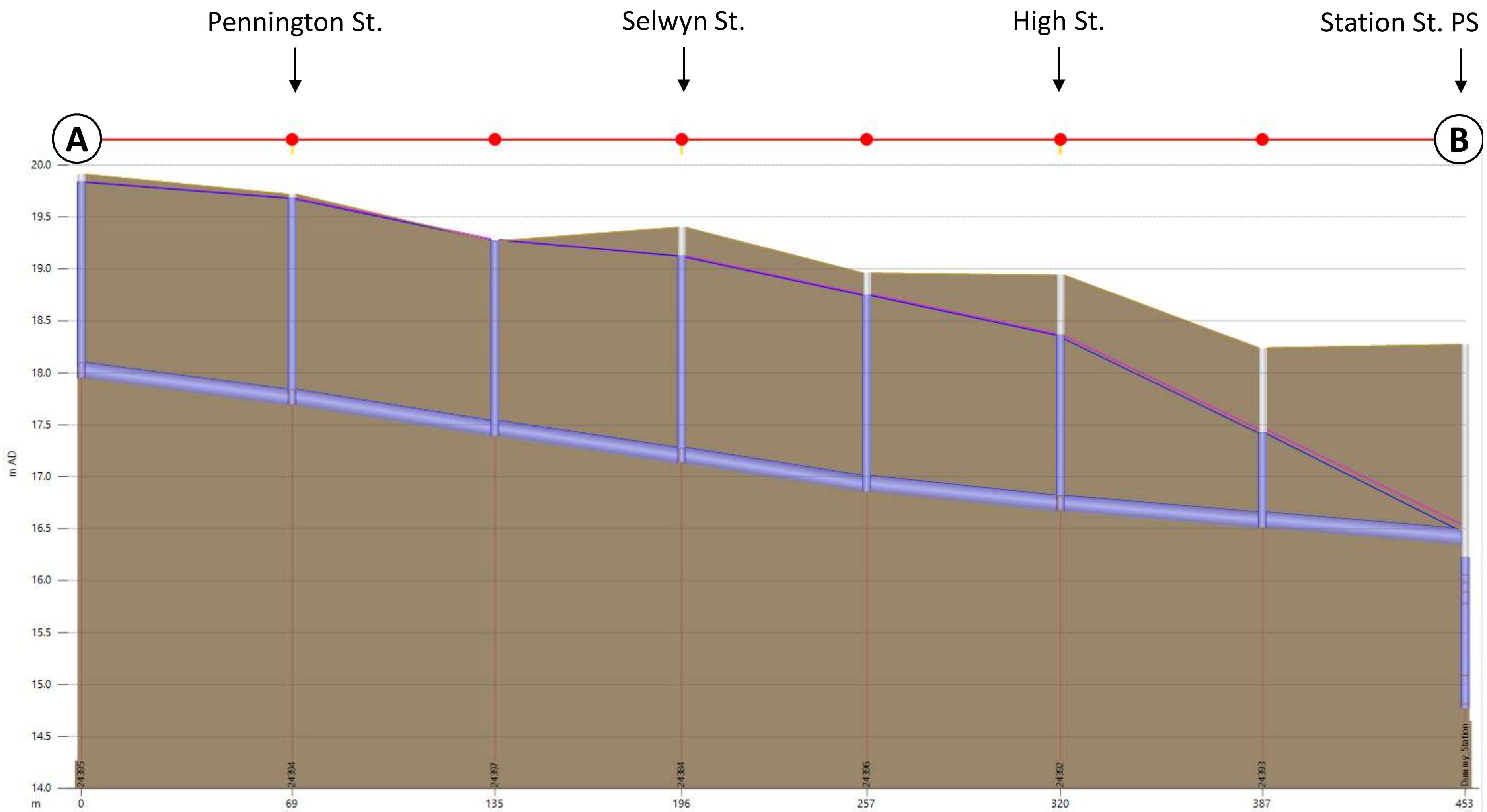


Link	24395.1	24394.1	24397.1	24384.1	24396.1	24392.1	24393.2
length (m)	69.1	66.4	61.0	60.7	63.3	66.2	66.3
width (mm)	150	150	150	150	150	150	150
Max surcharg	1	1	1	2	2	2	2
DS flow (m3/s)	0.00362	0.00822	0.00731	0.01040	0.01055	0.01487	0.01501

Node	24395	24394	24397	24384	24396	24392	24393	Dummy_Station
ground (m AD)	19.910	19.720	19.270	19.400	18.960	18.940	18.240	18.272
Ch floor lev (m AD)	17.950	17.690	17.390	17.130	16.860	16.670	16.510	14.772

Scenario 2: Cunningham Street Maximum HGL Profile

5-Year Storm Event

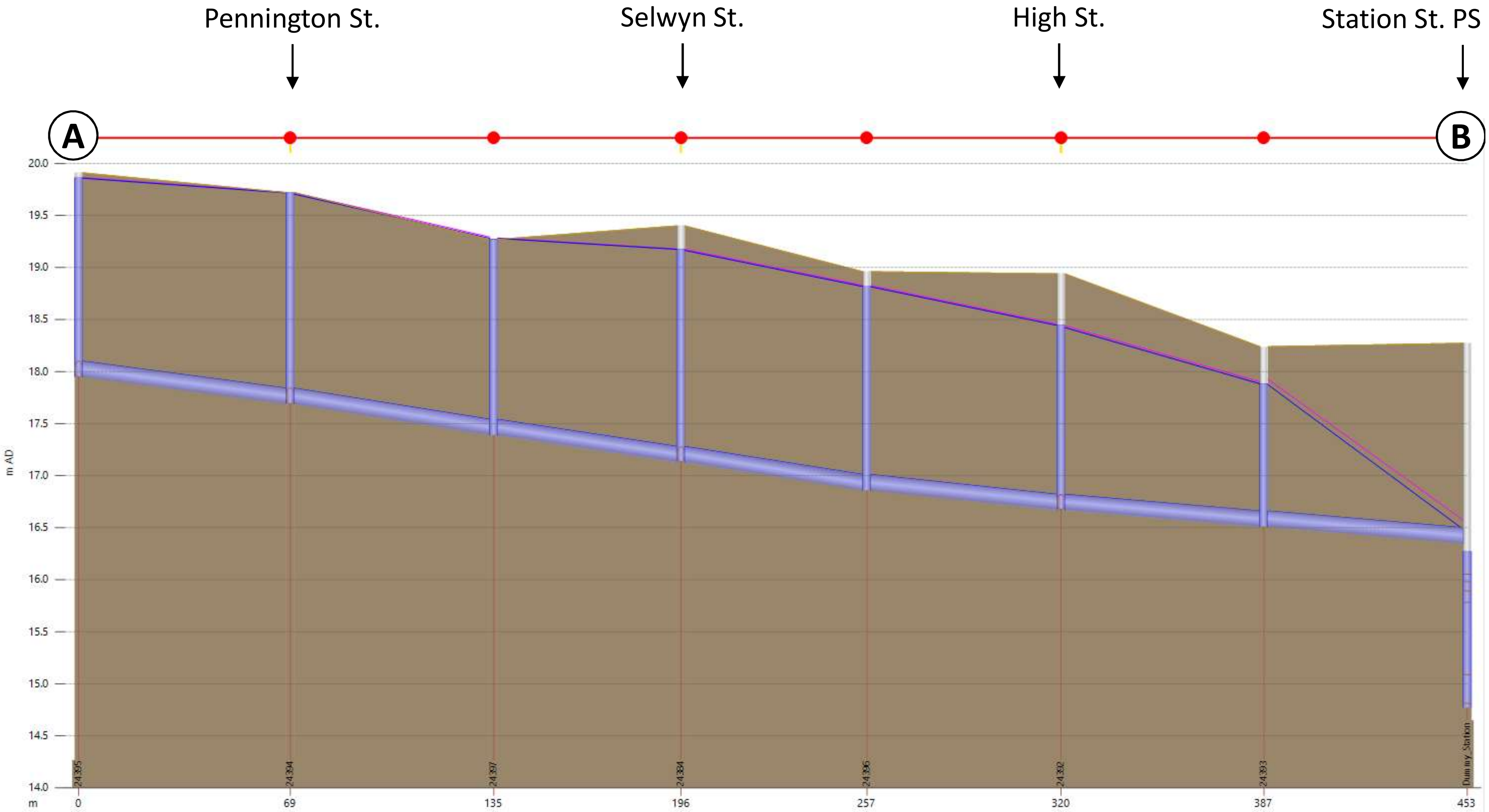


Link	24395.1	24394.1	24397.1	24384.1	24396.1	24392.1	24393.2
length (m)	69.1	66.4	61.0	60.7	63.3	66.2	66.3
width (mm)	150	150	150	150	150	150	150
Max surcharg	2	2	2	2	2	2	2
DS flow (m3/s)	0.01006	0.01148	0.01088	0.01284	0.01291	0.01777	0.01805

Node	24395	24394	24397	24384	24396	24392	24393	Dummy_Station
ground (m AD)	19.910	19.720	19.270	19.400	18.960	18.940	18.240	18.272
Ch floor lev (m AD)	17.950	17.690	17.390	17.130	16.860	16.670	16.510	14.772

Scenario 3: Cunningham Street Maximum HGL Profile

5-Year Storm Event

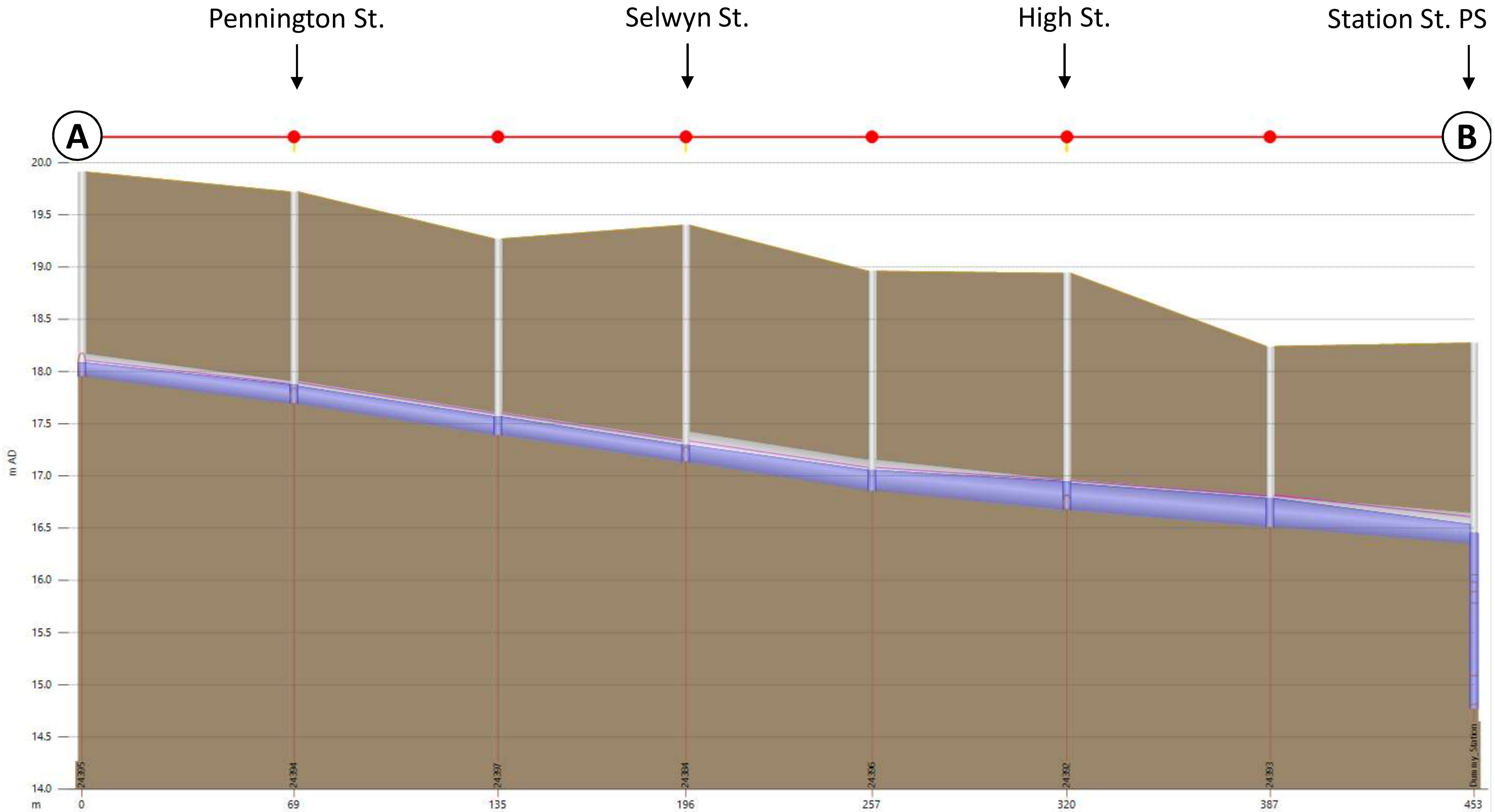


Link	24395.1	24394.1	24397.1	24384.1	24396.1	24392.1	24393.2
length (m)	69.1	66.4	61.0	60.7	63.3	66.2	66.3
width (mm)	150	150	150	150	150	150	150
Max surcharg	1	2	1	2	2	2	2
DS flow (m3/s)	0.00909	0.01197	0.00992	0.01139	0.01159	0.01367	0.02200

Node	24395	24394	24397	24384	24396	24392	24393	Dummy_Station
ground (m AD)	19.910	19.720	19.270	19.400	18.960	18.940	18.240	18.272
Ch floor lev (m AD)	17.950	17.690	17.390	17.130	16.860	16.670	16.510	14.772

Scenario 4: Cunningham Street Maximum HGL Profile

5-Year Storm Event



Link	24395.1	24394.1	24397.1	24384.1	24396.1	24392.1	24393.2
length (m)	69.1	66.4	61.0	60.7	63.3	66.2	66.3
width (mm)	225	225	225	300	300	300	300
Max surcharg	1	1	1	1	1	1	1
DS flow (m3/s)	0.01719	0.02683	0.02725	0.03563	0.03587	0.04728	0.05553

Node	24395	24394	24397	24384	24396	24392	24393	Dummy_Station
ground (m AD)	19.910	19.720	19.270	19.400	18.960	18.940	18.240	18.272
Ch floor lev (m AD)	17.950	17.690	17.390	17.130	16.860	16.670	16.510	14.772