civil infrastructure consulting

2 Johnston St Subdivision

Response to NPDC Request for Additional Information
7905565 (PPC18/00049)
19 December 2018

for Hareb Investments Limited

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Reviewed

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1.1 In Response to item 12 a (Part 1) Sewer Capacity

The NPDC Request for further information states:

"The plan change request provides high level calculations regarding sewer and water capacity. New Plymouth District Council has acknowledged that it does not have the resources to do this modelling. Can you provide a more detailed assessment to support this assumption? Water needs to be assessed at 60% Peak Daily Flow plus fire demand."

A high level pipe capacity analysis was carried out to determine if the existing sewer has the capacity for the proposed new development on Johnston Street. This analysis was carried out using the area method, based on:

- 70% of the total land area being used for residential development (the remaining 30% is road reserve).
- A minimum lot size of 450m^2
- 2.6 persons per property
- 250 L/p/day
- Dry weather diurnal peak flow of 2.5
- A dilution factor of 2.0

1.1.1 Receiving Sewer Reticulation and Calculation

It has been determined that the most likely point of connection into the existing sewer is manhole WA-RALEI0029SH on Raleigh Street.

The sewer capacity analysis is based on skeletonising the existing sewer pipes into three lengths as illustrated in Figure 1 below, namely:

1. Raleigh Street, 150mm diameter sewer pipe – From manhole WA-RALEI0029SH (pipe invert level 28.32) to manhole WA-MOULD0085SH (pipe invert level 7.78m).
2. Strange Street, 225mm diameter sewer pipe – From manhole WA-MOULD0085SH (pipe invert level 7.78m) to manhole WA-STRAN0092SH (pipe invert level 6.80m).
3. McNaughton Street, 225mm diameter sewer pipe – From manhole WA-STRAN0092SH (pipe invert level 6.80m) to manhole WA-BROAD0098SH (pipe invert level 3.91m)

Table 1 - Existing Sewer Pipe Capacity Calculation

Pipe dia	Area (ha)	Total Area Served (ha)	Sewer Location	Min Lot Size (m2)	70% Of land Area (ha)	Max No. of prop served	Peak Flow from properties (L/s)	US Pipe Invert (RL)	DS Pipe Invert (RL)	Pipe Length (m)	Pipe Slope %	Pipe Slope 1 in x	Max Flow (L/s)
150	7	7	Raleigh St	450	4.9	109	4.10	23.82	7.78	525	3.06	33	25.0
225	16.8	23.8	Strange St	450	16.66	370	13.93	7.78	6.8	225	0.44	230	32.5
225	15.5	39.3	McNaughton	450	27.51	611	23.00	6.8	3.91	330	0.88	114	48.0

As can be seen from Table 1 above, each of the sewer mains has additional capacity for 4.1 L/s of sewage from the proposed development area.

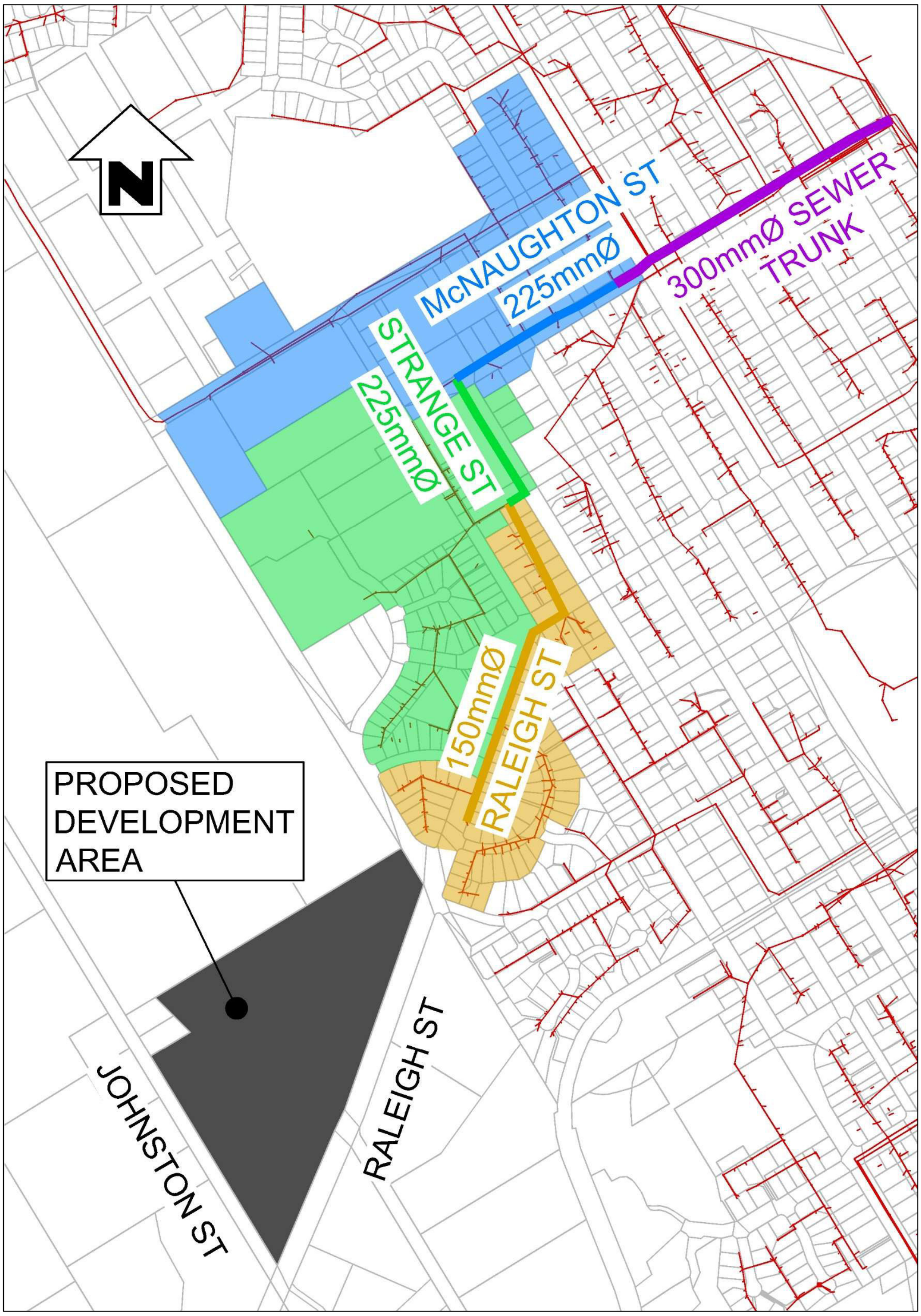

Figure 1 : Receiving Sewer Reticulation

The subsequent 250mm diameter sewer main downstream of the 225mm diameter sewer main on McNaughton Street is deemed to be a trunk sewer main and is assumed to have adequate sewer capacity for the additional sewage from the proposed development on Johnston Street. Short of modelling the whole sewer reticulation, this is deemed an appropriate approach for investigating the capacity of the existing sewer reticulation.

1.2 In Response to item 12 a (Part 2) Water Capacity

The NPDC Request for further information states:

"The plan change request provides high level calculations regarding sewer and water capacity. New Plymouth District Council has acknowledged that it does not have the resources to do this modelling. Can you provide a more detailed assessment to support this assumption? Water needs to be assessed at 60% Peak Daily Flow plus fire demand."

Civil Infrastructure Consulting engaged Watershed to carry out detailed water modelling utilising the recently calibrated Infoworks WS water supply model. Refer the *Raleigh Street Development Water Supply Assessment* report by Watershed, dated 03/03/2019

As requested by NPDC in the Request for Further Information, the domestic water demand was modelled at 60% (3.18 L/s) of the anticipated peak water demand of 5.3 L/s.

The detailed water modelling has identified that domestic water supply and fire fighting water supply of FW3 level can be provided to the proposed development area from the existing reticulation, without any requirement to upgrade the existing reticulation, if the development is fed off the existing 150mm diameter AC water main on Raleigh Street.

1.3 In Response to item 12 b – Onsite stormwater disposal

The NPDC Request for further information states:

"Given that the water table was measured at 3.5m BGL, can you comment on whether an alternative soakage tool such as a shallow raincell would be appropriate for the proposed plan change?"

As detailed in Section 3.4 of the *Johnston Street Waitara, Subdivision Feasibility Report, Civil Infrastructure Consulting Limited* report, the large majority of the upper soil layer of the subject site is comprised of a firm Taranaki Volcanic Ash layer of approximately 3.0m to 3.2m in thickness, which is well draining in terms of on-site stormwater disposal.

Given the depth of the water table below existing ground level, shallow soak pits or rain cells are a viable option for surface stormwater runoff from residential building roofs, hardstand areas and road pavement.

Alternatively, a stormwater detention pond could be designed to attenuate additional stormwater flows from residential building roofs, hardstand areas and road pavement if required.

1.4 In Response to item 12 c – Onsite stormwater disposal

The NPDC Request for further information states:

"Based on volume levels provided in the report for the stream and gully and its use for detention, additional detail is required as to how the calculations were completed and the final numbers derived."

Table 3.1 of the *Johnston Street Waitara, Subdivision Feasibility Report, Civil Infrastructure Consulting Limited* report tabulates both:

- The peak stormwater flow for a 1% storm event – using the Rational formula Qpeak=2.78CIA

- The total stormwater volume for a 1% storm event – by multiplying the total 1% Rainfall depth by the stormwater catchment area.

1.4.1 Stormwater Detention Volume

The stormwater detention volume in the gully was identified from the Autocad Civil 3D software. The software has the capability to measure the volume between two surfaces. The proposed ground model was created by designing a new bund within the gully and adding this on top of the existing ground model which was obtained from NPDC GIS contours and on-site survey around the extents of the gully.

Another flat surface was then created to represent the finished stormwater level. The Autocad Civil 3D software was then utilised to compute the volume between the two surfaces, which represents the total volume of water that can be detained.

Figure 2: Existing ground model with Proposed New Stormwater Detention Bund (view from south)

Figure 3: Proposed Stormwater Detention Bund with Stormwater to RL 28.25m (view from the south)

1.5 In Response to item 12 d – Stormwater Detention Bund

The NPDC Request for further information states:

"As the reserve will be vegetated the probability of vegetation debris will be high. Outline what size pipe would discharge from the detention bund to achieve the throttling of flow. Additionally, comment on how the risk of pipe blockage would be mitigated."

1.5.1 Stormwater Design Criteria

NZS4404:2010 with Amendment No. 1 is not detailed in the definition of hydraulic neutrality, where no peak design storm ARI is nominated.

Table 4.1 of NZS4404:2010 with Amendment No. 1 prescribes the following for residential Land and Residential floors for urban residential development:

- Primary stormwater systems to be designed to a 20% AEP storm event
- Secondary Stormwater to be designed for a 1% AEP storm event

Given that development from the Rural to the Urban residential standard will increase the peak stormwater flow. A stormwater detention outlet pipe designed for a 1% AEP event will, in fact, increase stormwater flows in the receiving catchment for all storm events equal to and less than 1%.

Given that the national NZS4404:2010 standard prescribes that Primary stormwater systems are designed with a 10% AEP, it is therefore, recommended by CIC that the stormwater outlet pipe is designed for the peak flow of the 10% AEP storm event. NZS4404:2010 with Amendment No. 1 also prescribes all road culverts to be designed to a 10% AEP storm event.

This will mean that for a 20% AEP storm event, the peak stormwater flow after development, will be slightly above that than before development, but, this will not occur for any storm events greater than the 10% AEP storm event. This is deemed to be a sensible approach to the issue of stormwater neutrality.

The existing Peak Stormwater flow for a 10% AEP storm event has been calculated at 448 L/s

1.5.2 Detention Pond Outlet Pipe

Below is an example of how to size the detention pond outlet pipe, however, this is not intended to be a final design solution and is purely to demonstrate the ability to size the stormwater outlet pipe to maintain hydraulically neutral flow.

Parameters:

- Peak pre-development stormwater flow for 10% AEP storm event = 448 L/s
- A stormwater bund RL of 28.25m (which would provide a detention volume of 1,416m3) provides a headwater level of 2.25m (RL of stream invert = 26.0m)
- A stormwater outlet pipe with an internal diameter of 375mm
- HW / D = 6.0

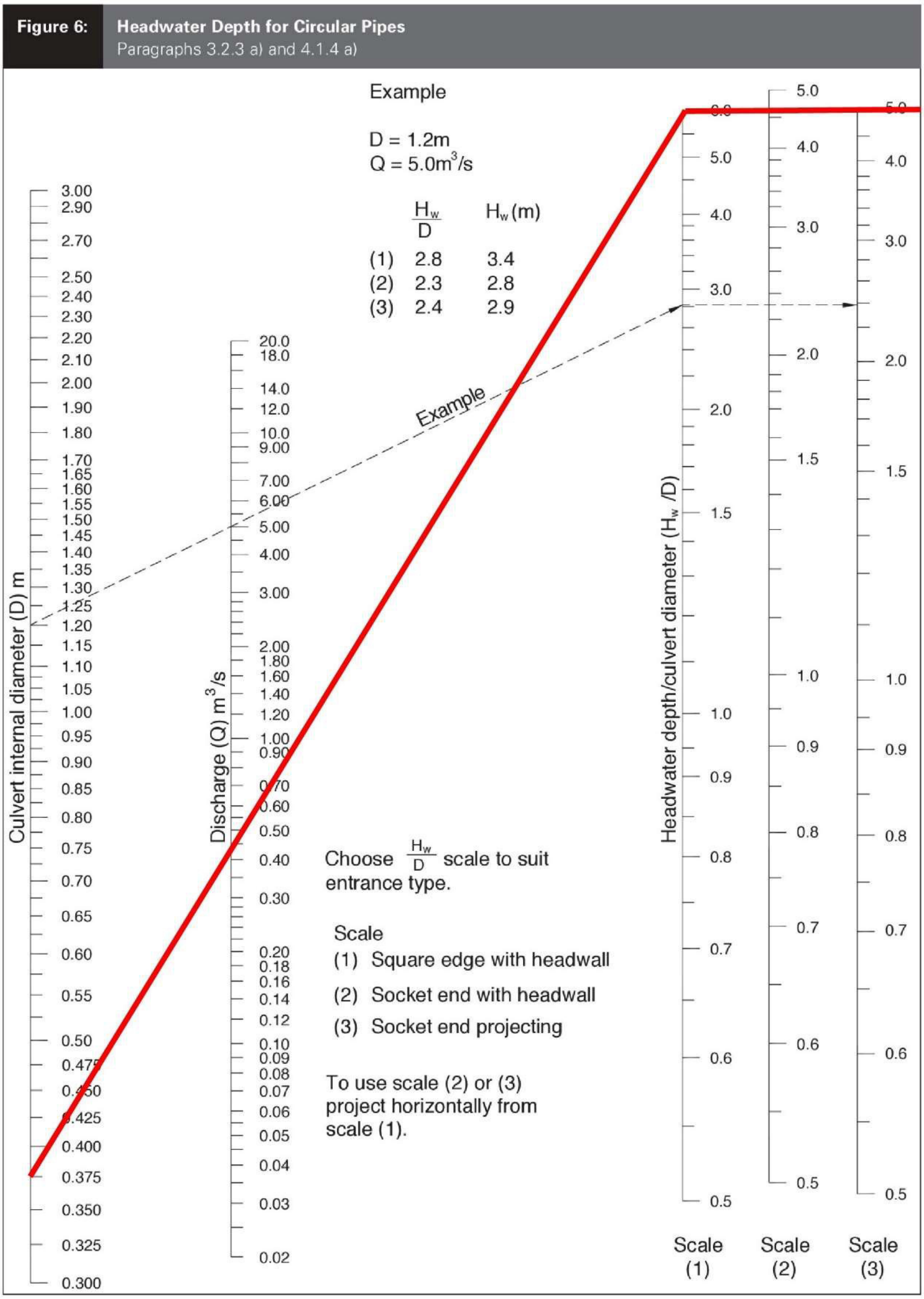

Figure 4: Detention Pond Outlet Pipe size – EXAMPLE ONLY

As can be seen from Figure 4 above, a bund constructed with a top RL of 28.25m (which provides a total of 1,416m3), and a 375mm diameter outlet pipe will result in a maximum of 448 L/s outlet flow.

1.5.3 Stormwater Outlet Pipe Blockage Prevention

A debris screen could be utilised to reduce the probability of blockage at the upstream end of the outlet pipe.

The detailed design of any debris screen is best left to the detailed design phase, however, there are numerous examples across NZ of debris screens down stream of vegetated streams/pond situations.

1.6 In Response to item 12 e – Stormwater Detention Bund

The NPDC Request for further information states:

"The plan change request details that hydraulic neutrality can be achieved. Provide a series of scenarios to demonstrate the most appropriate design event."

As detailed above in Section 1.5.1, a position needs to be agreed (between NPDC and developers) as to what design storm hydraulic neutrality will be defined. In this case, it is proposed to be, for the pre-development 10% AEP storm event.

REFERENCES

Johnston Street Waitara, Subdivision Feasibility Report, Civil Infrastructure Consulting Limited 16-09-2018.

NZS4404:2010 with Amendment No. 1

APPENDIX A WATERSHED – RALEIGH STREET DEVELOPMENT WATER SUPPLY ASSESSMENT

WATERSHED

TECHNICAL MEMO

Raleigh Street Development Water Supply Assessment

Waitara

Prepared by:	Tristan Jamieson	**Reviewed by:**	
Date:	03/03/2019	**Date:**	

INTRODUCTION

This summary report has been produced for Civil Infrastructure Consulting (CIC) and outlines the results of hydraulic analysis associated with a proposed development in Raleigh Street, Waitara. The assessment was requested by New Plymouth District Council to utilise the recently calibrated model of the water supply network, ensuring that any design accounts for the model-predicted impact on the network.

The assessment considers both normal consumption and fire-fighting requirements.

Figure 1 shows the location of the proposed development along Raleigh Street, Waitara

Figure 1 location of proposed development

MODEL LIMITATIONS

The following limitations should be considered when reviewing the finding of this study:

- The model is a calibration scenario, which has been factored for peak day consumption. Any controls are as they would have operated during the calibration period, and may not be representative of how the network would be operated during a peak demand period.
- It is understood that hydrant flow testing was not part of the calibration procedure, so there are significant reservations about the model's ability to assess firefighting capacity in the network. It is recommended that any fire flow assessments undertaken in the model are confirmed through field testing.
- Every effort has been made to utilise the provided model appropriately, and any alterations or updates undertaken to the model as part of this study assume that it was developed using an industry standard approach.
- The hydraulic model is a representation of the physical water supply system, and has limitations to its accuracy. The demands and peaking factors are based on assumptions and the actual final water demands may vary.

SUPPORTING INFORMATION

The following table (Table 1) sets out the information received for the assessment.

Table 1 Information provided for assessment.

Data	Description	Source	Comment
New Plymouth Hydraulic Model - 20190207.wspt	Model to be used for assessment of network	NPDC	Calibration Day Model.
NZ1-15157080-New Plymouth Water Supply Hydraulic Model Calibration Report.pdf	Reporting associated with model development and calibration	NPDC	
ECM_7932182_v2_Water Modelling Guidelines - Draft Feb 2019 Hydraulic Infoworks Development Guide.pdf	Guidelines for undertaking assessment-specific reference to Waitara.	NPDC	Some items in this guideline are inconsistent with the scope provided by CIC. Instructions were to follow the CIC Scope.
NPDC Amendments to NZS4404 - Develop and Subdivision Infrastructure Standards.pdf	Development Standards	NPDC	Not reviewed – assumed to be incorporated into scope by CIC
Project Scope	Provided by email	CIC	
Development Demand	To be modelled as a constant flow of 3.18 L/s	CIC	Note: Modelling guideline requires a diurnal profile, rather than a constant flow.

DEMAND ASSESSMENT

Peak Day Model

The model provided was a calibration model, and not representative of a peak day scenario. New Plymouth District Council are currently in the process of producing the Peak Day Scenario, so provided a region-wide residential peaking factor of 1.25. As consistent with the guidelines, all other modelled demand (non-residential, large metered customers and Non-Revenue Water) were assumed to stay static.

The Peak Day model was achieved by creating a demand scaling file in Infoworks. This was applied to the Simulation File for the peak day scenario. Table 2 shows a summary of the differences between the calibration day and peak day scenario.

Table 2 Peak day model demand summary

	Calibration Day Model	Peak Day Model
Total System Demand	38832 m^3/day	44430 m^3/day
Peak Flow Rate	663 l/s	78 l/s

Development Demands

Demands for the development have been provided by CIC. The development has been included in the model as a single customer point, connected to the 150mm AC pipe (Asset ID: 40126265) on the east side of the street. It is noted that this is the opposite side of the street to the development, and that a street crossing would be required. For the purpose of assessment, it has been assumed that the street crossing is at least 150mm in diameter.

It is noted that there is a 450mm pipe on the west side of the street. This pipe is live in the model, and operating at the Zone HGL. It is understood that this pipe may not be available to the development for connection, and it has not been considered in this assessment.

PEAK DAY IMPACT - at development site

The peak day model has been run with and without the proposed development consumption. As shown in Figure 2 the proposed development results in a localized pressure reduction of approximately one meter during peak demand. It is noted that this assessment assumes a constant flow; if a residential consumption profile is applied to the development, then this would result in a larger impact on minimum pressures.

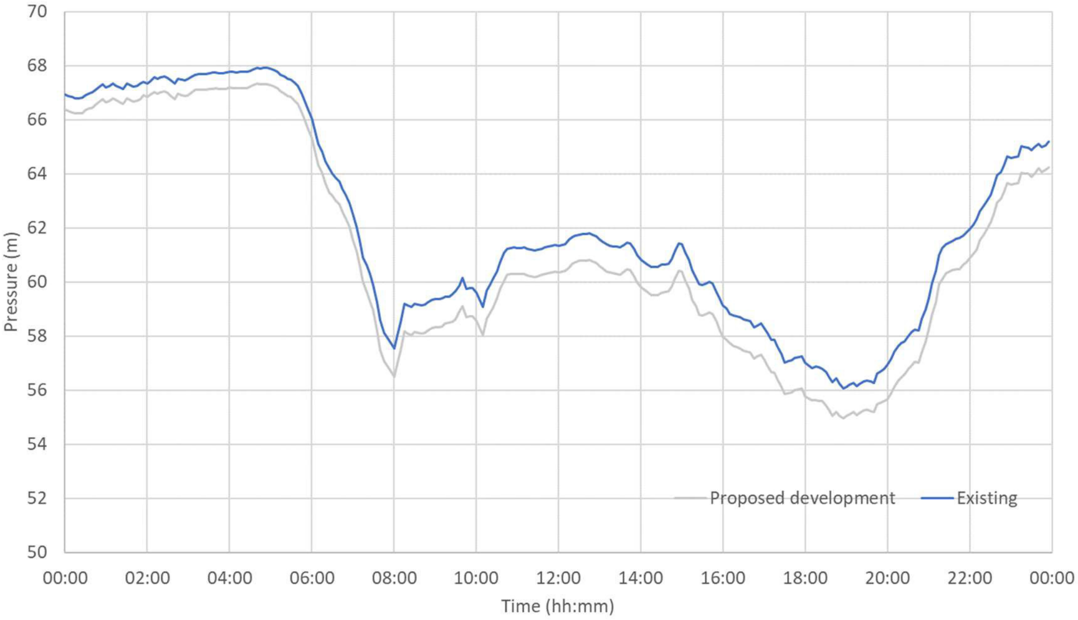

Figure 2 Impact of development on peak day pressures in the 150mm pipe along Raleigh Street

In general, the greatest effect of the development occurs in the localised area, resulting from headloss along the 150mm pipe. Current model-predicted head loss along this AC main is 2.2 m/km during peak demand, with an increase to 2.5 m/km as a result of the increased demand.

PEAK DAY IMPACT – in wider network.

The highest elevation point on the zone has been assumed to also be the critical point for supply pressure. This occurs in the vicinity of Lepperton, which is located close to the Mountain Road Reservoir site, and as a result has a pressure profile dominated by the water level in the reservoir. The headloss increase resulting from the development is not significant in the Lepperton area; however, as shown in Figure 3, the increased demand draws the reservoir down by a further 400mm over the period of the peak day. It is noted that the model predicts that the reservoir is insufficient on a peak day anyway, and has not recovered; however, it is also noted that the model has only been set up for peak day demand, and that the operation of the reservoir, and its filling mechanisms, have not been reviewed. For these reasons, no specific conclusions can be drawn from the model-predicted operation of the reservoir; however, it is recommended that further investigation is undertaken.

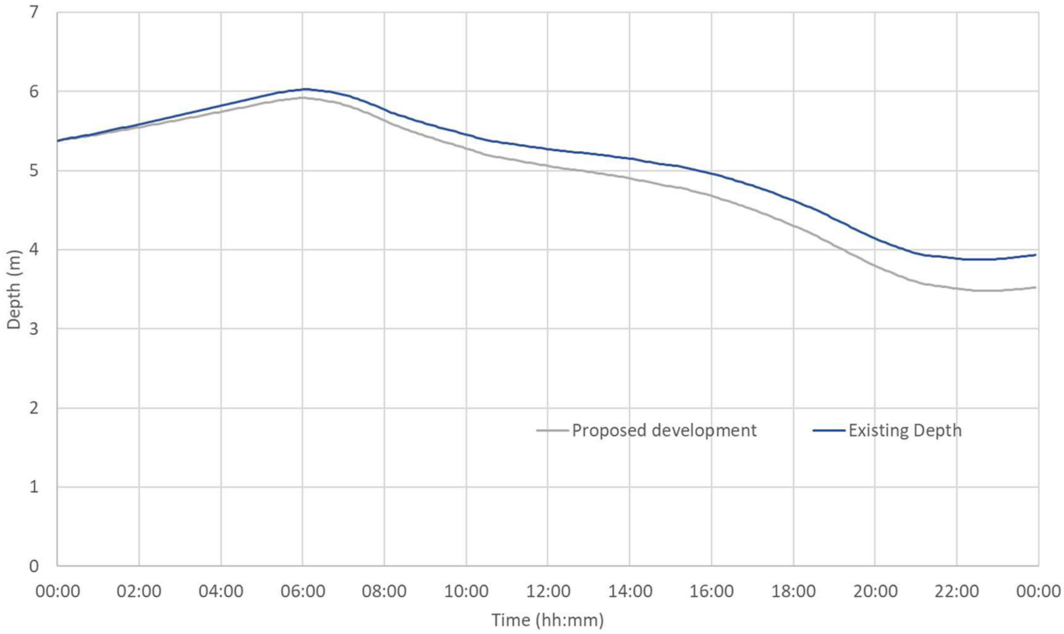

Figure 3 Mountain Road Reservoir Levels – current peak day

FIREFLOW ASSESSMENT

The model has been used to assess the fire flow capacity of the 150mm AC main, located on the East side of the Raleigh Street. Two separate fire flow simulations have been undertaken, assessing the impact of providing 25 l/s and 50 l/s. The assessment was undertaken at 10:30am on the peak day, which equates to 60% of the peak day demand. The proposed development demands were included as part of the assessment, to ensure that they did not significantly impact the existing firefighting protection provided by the network.

The 450mm pipe running along the west side of the street was not assessed; however, it is noted that there is a hydrant on this main (ID 40092477) which is located within less than 270m of partial areas of the proposed development.

As shown in Figure 4, the model predicts fire flows of both 25 and 50 l/s can be achieved while maintaining greater than 10m of head in the surrounding network. Specifically, the model predicted that 50 l/s could be extracted with a residual pressure of 16m, and 25 l/s with a residual pressure of 41m. It should be noted that this assessment is based on the flow available in the network itself. No account has been made for losses associated with individual hydrants, and it is expected that multiple hydrants would be required to achieve the 50 l/s.