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Corporate Authorization

This report entitled “2003 Water and Sewer Study” was completed for the City of Whitehorse by Stantec Consulting Ltd. in January 2004.

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Respectfully submitted,

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STANTEC CONSULTING LTD.
Signature ____________________
Date____________________
PERMIT NUMBER: P 0258
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

STANTEC CONSULTING LTD.

Todd S. Wyman, P.Eng.
Project Engineer

Executive Summary

Introduction

The City of Whitehorse (City) commissioned Stantec Consulting Ltd. (Stantec) for the provision of engineering services to conduct a water and sewer servicing study. The purpose of the study was to evaluate existing and future water and sewer infrastructure as it relates to capacity and growth and develop a Capital improvements implementation plan. Stanley Associates Engineering Ltd. (now Stantec) conducted a similar study in 1990. The 1990 report became the basis of Capital planning for City water and sewer infrastructure improvements for subsequent years.

The objectives of the current study were as follows:

- Determine the existing water demand and sewer flow generation rate within the City
- Upgrade the hydraulic model and develop a thermal model for the water distribution system
- Develop a hydraulic model for the sanitary sewer system
- Evaluate the capacities of the existing and future water and sewer infrastructure
- Evaluate thermal characteristics of the water system
- Identify deficiencies within the existing water and sanitary sewer networks and recommend improvements
- Prepare a stage implementation plan, which provides improvements necessary to meet future growth within the City
- Prepare conceptual level cost estimates for required improvements for the water and sewer infrastructure

Historical population data used for study analysis was based on data from Yukon Department of Health and Human Resources and Statistics Canada. Twenty (20) year population projections were based on the data from the City of Whitehorse Official Community Plan (OCP) population projections. Population projections were made based on low, medium and high growth scenarios of 0.5%, 1.5% and 2.0%.

Existing neighbourhood full development capacity was determined and future development areas were defined by combination of the OCP and area development plans. Future development areas include Porter Creek Extension, Lower Porter Creek Bench, Tank Farm Area Expansion, Riverdale Expansion and Beyond Copper Ridge.

The water system was studied by service areas based on water pressure zones created by major pumping stations and associated reservoirs. The sewer system was studied based on major collection areas of Marwell Contributing Area, Porter Creek Sewerage System and Crestview Sewerage System.

City base plans and mapping, land use analysis, infrastructure reports, consultation with City staff, field investigations, water use records and computer modeling were used to assess the water and sewer systems.

Water System Evaluation

In order to analyse existing and future water systems a set of evaluation criteria needed to be developed. Evaluation criteria was developed based on City of Whitehorse Servicing Standards, findings of the 1990 Water and Sewer Study and standard industry practice. Evaluation criteria were determined for water storage, pumping capacity and reliability, transmission and distribution systems, and system thermal characteristics.

The existing City water model was converted and updated to EPANET, which was developed, by the Water Supply and Water Resources of the United States Environmental Protection Agency's National Risk Management Research Laboratory. EPANET is open source and public domain software.

The chemical decay (used for chlorine residual analysis) formula in the existing EPANET software was adapted by Stantec to conduct thermal analysis of the water system.

The existing water system was analysed based on adopted criteria and the following provides a summary of findings and recommended measures for the next five years:

1) A very important alternative approach to infrastructure upgrade due to increased demand is demand side management. The City of Whitehorse should develop a water demand management strategy that encompasses: leak detection and repair, bleeder reduction, education, metering, rate structuring, economic incentive, regulation, politics, and plumbing fixtures. A water usage audit should also be incorporated into water demand management strategy. (Section 2.6.10 Water Demand Management)
2) Improving ground water supply and construction of a water treatment plant should be considered the City's top infrastructure upgrade priority.
3) The City should conduct further investigations into locating additional ground water sources due to the importance of the thermal gains, lack of turbidity and source security. (Section 3.3.3 Ground Water Wells)
4) The need for a water treatment plant has been recognised. Before commencing detailed design of a water treatment plant the City should undertake further study and pilot projects to ascertain the best treatment process. (Section 3.3.6 Treatment Plant Options)

5) The City needs to conduct a metering audit and implement a structured metering program. (Section 2.6.4)

6) McIntyre Pumphouse and Hamilton Pumphouse require standby power and booster upgrades to ensure water supply and fire flows (Section 5.3.1)

7) Fire flows are a concern near Transit, Hart and Centennial Circulation stations when circulation pumps are in operation. Fire flow bypass valves should be installed. (Section 4.2.2 Distribution Network)

8) Significant fire flow deficiencies were observed in Downtown's west side where fire flows of only 23 to 40 L/s were modeled. The deficiency is due to the limited interconnections of the systems and the smaller 150 mm diameter lines. (Section 4.2.2 Distribution Network)

9) Pressure sensors should be installed at Ponderosa Drive and Grove Street with interconnection to McIntyre Pumphouse control to ensure McIntyre Pumphouse booster pumps operate during fire flows. (Section 4.2.2 Distribution Network)

10) The existing 250 mm Industrial Road supply main should be upgraded to 400 mm from Two Mile Hill to Quartz Road and upgrading the 250 mm main to 300 mm from Quarts Road to Galena Road to improve fire flows in Marwell. The City may also wish to investigate reducing demand requirements by making automatic building fire suppression systems mandatory. Hydrant spacing should also be reviewed near buildings with high fire flow demand requirements. (Section 4.2.2 Distribution Network)

11) The Porter Creek undersized and should be twinned. (Section 4.2.3 Water Storage Reservoirs)

12) Thermal improvements are recommended in Riverdale, Crestview, Hillcrest, Airport and Takhini North. (Section 4.3)

Future water system analysis was conducted based on predicted water demands for existing and future neighbourhood development. Development phasing was predicted based on the City OCP and population projections. An infrastructure upgrade implementation plan is presented in Section 5.4.

Sanitary System Evaluation

An assessment of the recorded flow data has been carried out to establish the water consumption characteristics of the study area. The total measured annual sewage volume at the Marwell Lift Station for the year 2000 is 3,473,000 m^3 including the volume of inflow/infiltration whereas the total annual water consumption for the services area drained into the Marwell Lift Station is 3,883,636 m^3. The gross recovery rate is approximately 0.9. The annul sewage volume generated from the Porter Creek area is 752,100 m^3 and the annual water consumption volume for this area is 801,941 m^3 resulting in a recovery rate of 0.94.

There was no information available on the total annual sewage volume generated from the Crestview area. The existing composite sewage generation rate is about

655 Lpcd. The 1990 Water and Sewer Study reported an average sewage generation rate of approximately 1,100 Lpcd.

The Livingston Trail Environmental Control Facility (LTECF) facility was designed to fulfill the City's sewerage treatment needs until the year 2012 on the assumption that the City's sewage flow generation has to be reduced to 570 Lpcd. The Marwell Lift Station upgrade was also designed on the same assumption. The City of Whitehorse needs to continue with a flow reduction strategy in order to defer very costly capital improvement.

The peak flow ratio during wet weather flow events varies significantly from community to community. The wide variation is due to the rainfall induced inflow/infiltration and depends on the intensity and duration of rainfall events, soil type, pipe material etc. There is no wet weather flow data available for the City of Whitehorse.

The peak flow ratio for a particular location needs to be established by implementing a flow monitoring program to collect data during wet and dry weather periods.

Generally, experience elsewhere indicates that the peak flow ratio during wet weather events could vary from 3 to 4 under normal conditions; therefore, the hydraulic response of the Whitehorse sewer systems has been evaluated by utilizing a peak flow ratio of 3 and 4.

Computer modeling of the existing system was conducted using the RTSWMM model. The existing sewer system was analysed based on adopted criteria and the following provides a summary of findings and recommended measures for the next five years:

1) By reducing sanitary flows with demand side management strategies including public awareness campaigns, repair and rehabilitation programs and policy changes, the City could save a significant amount of operating costs and defer capital expenditures. The City should continue their water reduction program to reduce composite sewage inflows to 570 Lpcd. (Section 9.2.2)

2) The City of Whitehorse flow monitoring data is too limited to make and accurate assessment of model accuracy and upgrade requirements. Flow meters at all major facilities should be checked for accuracy and upgraded as required. The City also requires more portable flow monitoring to investigate suspect lines. Accurate rainfall data is also required to assess wet weather flows. (Section 9.2.1)

3) An operational assessment of Lift Station #1 and forcemain is recommended to make recommendation with regard to wet well sizing, operating levels and forcemain flushing. (Section 9.3.2)

4) Grinder installations are recommended at Lift Station # 1 and Porter Creek Gravity Forcemain. (Section 7.7)

5) Sections of sanitary trunk in Riverdale and Downtown were identified as having possible capacity concerns. Flow monitoring is required to confirm any upgrade requirement. (Section 9.3)

Future sewer system analysis was conducted based on predicted flow rates for existing and future neighbourhood development. An infrastructure upgrade implementation plan is presented in Section 10.5.

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1.0 Introduction

1.1 GENERAL

The City of Whitehorse (City) commissioned Stantec Consulting Ltd. (Stantec) for the provision of engineering services to carry out a water and sewer servicing study. The purpose of this study is to evaluate the existing water and sanitary sewer system and develop a Capital plan from current time to ultimate build out capacity. The study is intended to assist the City in planning for future growth, expansion of service areas, and system-upgrading requirements while maintaining the level of service in the existing areas. A similar study was conducted in 1990 and became the basis for capital planning in the City over the past 12 years. A significant amount of the recommendations of that report have been implemented over the years.

1.2 STUDY AREA

The City of Whitehorse is the largest community in Canada located north of the 60th parallel. It covers a very large area of approximately 2,300 hectares and has a variety of land uses. The estimated total developed area for various forms of urban use is 1,680 hectares with the balance of the area identified as open space. This study focuses on the existing urban development centres and future development areas as depicted in Figure 1.1.

1.3 BACKGROUND

The City has undertaken several major improvements to the water and sewer systems based upon recommendations of the 1990 study. The following encapsulates the improvement projects implemented over the years:

1) Crosstown Water Main Construction
2) Two Mile Hill Booster Station Replacement
3) Yukon Energy Dam Siphon Construction and Selkirk Pump Station Supply Line Partial Twinning
4) Hillcrest and Airport Transmission Main Twinning
5) Copper Ridge Pumphouse and Reservoir Construction
6) Granger Booster Station Construction
7) Hamilton Boulevard Pumphouse Header Bypass
8) Lift Station #3 Generator and Pump Replacement
9) Lift Station #1 Pump Replacement
10) Livingston Trail Environmental Control Facility Construction and Marwell Lift Upgrade

1.4 OBJECTIVES OF STUDY

The objectives of the study were to determine existing capacities of the water and sewer systems and provide a tool for planning future growth and ultimate build out capacity.

1.5 PROJECT SCOPE AND DELIVERABLES

The scope of work for the project was to undertake a technical evaluation of the existing water and sewer systems and, in conjunction with the City of Whitehorse, develop a staged infrastructure upgrade implementation plan. The main steps in performing the assessment include:

1. Determine the existing water demand and sewer flow generation rate within the City
2. Upgrade the hydraulic model and develop a thermal model for the water distribution system
3. Develop a hydraulic model for the sanitary sewer system
4. Evaluate the capacities of the existing water and sewer infrastructure
5. Evaluate the water system thermal characteristics
6. Identify deficiencies within the existing water and sanitary sewer networks and recommend improvements
7. Evaluate the projected water and sewer flow characteristics and identify necessary improvements to meet the future needs
8. Prepare a stage implementation plan, which identifies improvements necessary to meet future growth within the City
9. Prepare conceptual level cost estimates within the water and sewer implementation plans

Stantec reviewed all the available existing reports and background information pertaining to the works and undertook an updated assessment of the various facilities. The study considered the following with regard facility assessments:

- Population growth
- Future developments
- Adequate system capacity
- Water supply pressures and fire flows
- Water demand and sewer generation for the neighbourhoods in the City
- Surface and ground water supply, capacities, water quality, and treatment
- Pumping and circulation stations and capacity
- Transmission mains and capacity

- Reservoir sizing and location
- Sewage lift stations and capacity
- Sewer trunk main sizing and capacity

1.6 METHODOLOGY

The methodology used in this report follows the Work Program as detailed in Stantec's Proposal for Engineering Services submission of 17 October, 2001. The methodology is as follows:

Project Initiation and Management	Confirm terms of reference, complete data gathering, review and finalization of administrative procedures, review and confirmation of the project schedule and signing of engineering contract.
Data Collection and Literature Review	Collect and review all the available reports and documents, population data, existing and proposed land use plan, all the relevant data such as flows, pressures, as built drawings, information on all the water and sanitary sewer infrastructure, etc.
Surface and Ground Water Supply, Capacities, Water Quality and Treatment	Evaluate existing surface and ground water supply with respect to capacities, water quality and treatment.
Water Model Update and Calibration	Hydraulic model development, flow demand analysis, development of a field testing program, and model calibration
Thermal Model Update and Calibration	Thermal model development, flow demand analysis, development of a field testing program and calibration
Existing Water System Evaluation	Evaluate the performance of the existing system with respect to the desired level of service and common water distribution standards during different demand conditions such as average day demand, maximum day demands, peak flow demands, fire flow demands during summer and winter operating conditions
Sewer Model Development and Calibration –	Hydraulic model development and historic sewer flow generation rate analysis

Existing Sewer System Evaluation	Evaluate the performance of the existing system with respect to the desired level of service
Implementation Plan	Develop expansion servicing concepts, provide upgrading cost estimates and establish implementation plan for system upgrading and expansion
Documentation	Present findings in a meaningful format

1.7 SITE INVESTIGATIONS

As a part of the data input for the study City staff and Quest Engineering undertook several site investigations. These were utilized to field verify critical components on the water and sewer system. Field verification of all elevations and alignments will be required prior to commencing with pre-design.

1.8 ACKNOWLEDGEMENTS

The contribution of the following personnel in pursuing this study and in the preparation of this report is greatly acknowledged.

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2.0 Information and Criteria - Water Distribution System

2.1 GENERAL

Review of existing information provided the project team with background knowledge necessary for proper evaluation of the water system. Separate data collection efforts focused on water sources, storage reservoirs, transmission and distribution systems, performance data, and anecdotal City knowledge. The following data was collected:

- Base plans and mapping
- System inventory and physical data
- Existing water customer statistics, growth projections, and scenarios
- Current water demand data
- Performance data (flows, pressures, field tests, chlorine residuals) as monitored
- Groundwater elevation data
- Ground temperature and water temperature data
- Interviews regarding existing issues from City of Whitehorse Public Works staff Anecdotal information
- Applicable City policies and service provision criteria

Literature review of relevant studies and reports included but was not limited to the following:

- Annual water license reports (1999-2001)
- 1990 Water and Sewer Study – Stanley Associates Engineering Ltd. (Stantec)
- 1991 Crosstown Predesign Report for pumping facilities including thermal modeling (Stantec)
- 1998 Copper Ridge Stage 7 to 11 (Stantec)
- 1996 Copper Ridge Pumphouse and Reservoir (David Nairne & Associates)
- 1995 Crosstown Water Main Phasing Report (Stantec)
- 1997 Downtown District Predesign Report (David Nairne & Associates)
- 1997 Warm Water Well Development Program, revised May 1999 (Gartner Lee)

- 1999 Pumphouse and Lift Station Audit report (Stantec)
- 2002 Official Community Plan (OCP) planning work (UMA)
- Air Photos, for the City of Whitehorse
- 2001 SCADA Pre-Design Report for four major facilities (Dorward Engineering/Stantec)
- 2001 Marwell Planning and Pre-Design Report (David Nairne)
- 2000 Pre-Design Report for Porter Creek Water Improvements (Yukon Engineering Services)
- 2001 Water Treatment Plant – Feasibility Report (EPCOR)
- 2002 Marwell Area Engineering Predesign Report (David Nairne & Associates)
- 2003 Vanier School Ground Source Heat Pump Project (Gartner Lee Ltd)
- 2003 Marwell Water System Hydraulic and Thermal Assessment Summary (Quest Engineering Group)

The data collection and literature review provided information that can be grouped into the following categories:

1) Water Transmission and Distribution System and Facilities
2) Service Population
3) Water Demands
4) Temperature Monitoring Data
5) Thermal Modeling

The findings of the data collection and literature review are described below.

2.2 WATER DISTRIBUTION SYSTEM

The City of Whitehorse domestic water supply is provided by a piped water distribution system that draws water from Schwatka Lake and a series of water wells near Selkirk Street in the Riverdale Neighbourhood. The distribution system services an area of approximately 880 hectares extending from Riverdale in the south to Crestview located in the northern part of the City. Currently, rural areas within the City limits are not serviced by the water distribution system. In addition there are some "dry industrial" lands, Kulan Industrial Area and McDonald Industrial Area that are not serviced. Figure 2.1 indicates service areas for this study.

Various pumphouses, booster stations, reservoirs, pressure reducing stations and pressure sustaining stations control water distribution and pressures. Figure 2.2 indicates the locations of these facilities.

In order to prevent the distribution system from freezing, the City utilizes many circulation pumps, boilers and various types of bleeders throughout the system. Figure 2.3 indicates the locations of these frost protection systems.

In order to simplify the water system layout, a plan view water system schematic is provided in Figure 2.4 and a profile view water system schematic is provided in Figure 2.5.

2.2.1 Service Areas

The City of Whitehorse can be divided into two major elevation zones consisting of the upper escarpment zone and lower zone. These two zones are further divided into five service areas as depicted in Figure 2.1. The lower zone is comprised of Service Area 1 only. The upper escarpment zone is comprised of Service Areas 2 to 5. The Two Mile Booster Station represents the division between the two zones.

Selkirk Pumphouse and Riverdale Reservoir Service Area 1

Service Area 1 includes the neighbourhoods of Riverdale, Downtown and the Hospital Area. Area 1 receives water from the Selkirk Pumphouse and Riverdale Reservoir. All municipal water is supplied through Selkirk Pumphouse (HGL 679.0 m / Ground 636.19 m). The source water for Selkirk pumphouse is provided by a combination of Schwatka Lake and ground water wells. Water disinfection is provided by chlorine injection at Selkirk Pumphouse with an objective chlorine residual leaving the station of 0.8 mg/L. Selkirk Pumphouse was connected to the City SCADA network in 2001 for control and data acquisition.

The topography in the area is relatively flat but slightly rising in the Riverdale neighbourhood. The service area consists of only one pressure zone whose static hydraulic gradeline is controlled by the Riverdale Reservoir (HGL 675.13 m – 679.15 m). The existing population of Area 1 is approximately 7,554.

Two Mile Hill Booster and Valleyview Reservoir Service Area 2

Service Area 2 includes the Whitehorse International Airport, Marwell Industrial Area, Yukon College and the neighbourhoods of Takhini, Valleyview, Hillcrest, and McIntyre. Area 2 receives water from Area 1 via the Two Mile Hill Booster Station (HGL 764.15 m and ground elevation 644.8 m). Two Mile Booster Station supplies all pumphouses in the upper escarpment zone. Two Mile Booster Station also provides rechlorination to a chlorine residual of 0.8 mg/L. Two Mile Booster Station also provides the central station for the City SCADA system.

The majority of Service Area 2 pressure is controlled by the Valleyview Reservoir (HGL 754.0 m – 759.4 m). The areas that directly serviced by Valleyview Reservoir include the Airport, Hillcrest, Valleyview, McIntyre, Takhini South, Takhini West-South, Takhini East and Yukon College.

The Marwell Industrial Area is a sub pressure zone of Area 2. A pressure reducing valve located at the Two Mile Hill Booster Station (HGL 693.57m and ground elevation 644.57 m) reduces Marwell pressure from Area 2.

The old Takhini North area has not been upgraded and is a sub pressure zone of Area 2. Two pressure reducing stations control pressure to Takhini North. Elvin's Pressure Reducing Station is located on Range Road south of Two Mile Hill Road (HGL 748.83 m and ground elevation 691.1 m). Ortona Pressure Reducing Station is located at the intersection of Ortona Road and Arnhem Road near the north end of Takhini North (HGL 748.81 m and ground elevation 698.65 m). Though the set points slightly vary to control opening sequence these stations supply the same area.

Range Road North is a sub pressure zone of Area 2. This sub-zone includes Takhini Trailer Park, Northlands Trailer Park, Crow Street and Mountainview Place. The sub-zone is serviced by Range Road Pressure Reducing Station (HGL 681.9 m and ground elevation 684.3 m).

The existing population of Area 2 is approximately 3,419.

McIntyre Creek Pumphouse and Porter Creek Reservoir Service Area 3

Service Area 3 includes the neighbourhoods of Porter Creek, Kopper King and Crestview. Area 3 receives water from Area 2 via the McIntyre Creek Pumphouse (HGL 793.86 m and ground elevation 695 m). In addition to boosting water McIntyre Creek Pumphouse has boilers to temper water during times of near freezing water supply. The majority of the service area pressure is controlled by the Porter Creek Reservoir (HGL 769 m – 774.8 m).

The higher elevations of Ponderosa Drive are a sub pressure zone of Area 3. The Ponderosa Booster Station (HGL 805.88) and the Ponderosa Pressure Sustaining Station controls pressure in this sub-zone.

Most of Grove Street is a sub pressure zone of Area 3. Grove Street Booster (HGL 806.51) and Grove Street Pressure Sustaining Station controls pressure in this sub-zone.

The Crestview Neighbourhood upper area is a sub pressure zone of Area 3. Pressure in this sub-zone is controlled by the Crestview Pumphouse (HGL 807.18 m and ground elevation 723.8 m). Booster pumps in series with Circulation pumps provide operating pressure and winter water circulation in the upper area.

The lower area of the Crestview Neighbourhood is supplied through pressure reducing valve (HGL 775.12 m and ground elevation 723.8 m) located in Crestview Pumphouse. The pressure reducing valve reduces pressure from the high pressure side of the Crestview booster pumps. Crestview Pumphouse also has a small boiler to temper water during times of near freezing water supply. Additional frost protection is provided in the lower area by means of two thermostatically controlled watermain bleeders on Klukshu Avenue and Kathleen Road.

The existing population of Area 3 is approximately 5,254.

Hamilton Boulevard Pumphouse and Hillcrest Reservoir Service Area 4

Service Area 4 consists of the neighbourhoods of Granger, Arkell and Logan. Service Area 4 is supplied from Service Area 2 by the Hamilton Boulevard Pumphouse (HGL 801.5 m and ground elevation 723.9 m). Area 4 also supplies water to Copper Ridge Pumphouse. Hillcrest Reservoir (HGL 794 m – 799 m) controls the hydraulic gradeline of Service Area 4.

Hamilton Pumphouse contains boilers to temper water during times of near freezing water supply. Hamilton Pumphouse also has circulation pumps for Arkell/Logan Neighbourhoods in Area 4 and McIntyre Neighbourhood in Area 2. There is a third circulation pump for the Granger Neighbourhood that is no longer used since the construction of Granger Booster Station.

Granger Booster Station (HGL 812 m) and pressure sustaining stations on Thompson Drive and Wilson Drive control pressure in the upper portion of the Granger Neighbourhood.

The existing population of Area 4 is approximately 1,494.

Copper Ridge Pumphouse and Copper Ridge Reservoir Service Area 5

Service Area 5 includes the recently developed neighbourhood of Copper Ridge. Area 5 receives water from Area 4 via the Copper Ridge Pumphouse (HGL 830.44 m and ground elevation 786.03 m). Copper Ridge Reservoir (HGL 826 m – 830.7 m) controls the hydraulic gradeline for Service Area 5. Copper Ridge Pumphouse and Reservoir are connected to the City SCADA system for control and data acquisition.

Copper Ridge Pumphouse has circulation pumps for circulating water through two circulation zones within the Copper Ridge Neighbourhood. Copper Ridge Pumphouse also has boilers to temper water during times of near freezing water supply.

Yukon Emergency Services estimated the 1999 population of Service Area 5 at 554. Since Copper Ridge has undergone significant growth since 1999 any upgrade requirements will be based upon population triggers.

2.2.2 Existing Operations and Maintenance Programs

The City of Whitehorse currently has 6 full time employees for Water & Sewer System Maintenance activities. These employees engage in underground infrastructure repair and preventative maintenance. Preventative maintenance activities include unidirectional flushing, valve exercising, hydrant exercising/flushing, hydrant winterization, bleeder operation/checks, trouble manhole checks/maintenance, sanitary sewer cleaning and storm sewer cleaning.

The City of Whitehorse currently has 5 full time employees for Water and Wastewater Facility Operations and Maintenance. In addition to facility system repairs, these employees engage in facility inspection, control and data acquisition.

Based on recommendations of the 1990 Water and Sewer study the City improved leak detection capabilities by purchasing leak detection equipment and providing staff training. Although the City does not have a formal leak detection program, staff actively pursue leak detection while performing other preventative maintenance activities.

In 2002, the City of Whitehorse implemented a unidirectional watermain flushing program developed by Epcor. The unidirectional flushing program allowed the City to systematically and efficiently flush watermains to remove turbidity settlement. The valve exercising program and portions of the hydrant maintenance program were combined with the unidirectional flushing program. Unidirectional flushing also has the added benefit of allowing workers to inspect the entire distribution systems for leaks and anomalies. The unidirectional flushing program is beneficial to water quality and preventative maintenance should be continued.

The City has recently undertaken a multi-year Supervisory Control and Data Acquisition (SCADA) implementation program. The objectives of the SCADA system is to improve historical and instantaneous knowledge of system dynamics, provide improved automated control, provide remote operator control and investigation, provide improved alarm handling and dispatch, and provide improved historical data logging.

SCADA does not necessarily provide a net benefit in personnel timesaving. What SCADA does provide is vastly improved system knowledge, efficiency, reliability and accountability.

Existing SCADA implementation includes Selkirk Pumphouse, Riverdale Reservoir, Two Mile Booster, Valleyview Reservoir, Copper Ridge Pumphouse, Copper Ridge Reservoir and Marwell Lift Station. Current SCADA work underway includes McIntyre Creek Pumphouse and Hamilton Boulevard Pumphouse. Because of the benefits gained by SCADA the City should continue with its SCADA implementation plan and budgeting.

The City has in recent years actively strived to reduce water bleed and boiler operation times for frost protection. The City manually and automatically monitors water system temperatures and ground frost conditions to optimize when to turn on and off frost protection systems. This practice should be continued as apposed to automatically turning on and off frost protection systems on specific dates.

2.3 HISTORICAL SERVICE POPULATION

Yukon Department of Health and Human Resources prepares annual population estimates for the Yukon Territory and the City of Whitehorse. The Yukon Department of Health and Human Resources utilizes health care cards and postal code addresses to estimate population data. Statistics Canada conducts a census every five years to estimate population. This historic data is presented in Table 2.1.

Table 2.1 Historical Population of City of Whitehorse

Year	Yukon Health Care	Census Canada	Whitehorse Serviced Areas
1981	17,023	14,814	
1985	17,265	12,265	
1986	18,043	15,199	
1987	18,967		
1988	19,989		
1989	20,706		
1990	21,000		
1991	21,346	17,925	17,046
1992	22,242		17,942
1993	23,110		18,810
1994	22,854		18,554
1995	23,012	19,157	18,712
1996	23,611	21,065	19,311
1997	24,018		19,718
1998	23,406		19,106
1999	22,917		18,617
2000	22,649		18,349
2001	22,545	22,526	18,279

Note: Yukon Health Care value for 1990 was assumed by Stantec.

It is noted that there is a difference between the population estimated by the Yukon Department of Health and Human Resources and the population utilized by Statistics Canada. The discrepancy may be due to people residing in rural areas with a Whitehorse mailing address or people not cancelling Yukon Health Care while not residing in the Yukon.

Table 2.2 shows the existing population of each serviced neighbourhood. The population breakdown was based on information obtained from Emergency Services dated 1999. Further refinement and population distribution was based on physical lot counts utilizing the studies GIS model. The estimated population distribution was used to assess water demand for the hydraulic evaluation of the existing water and sewer system. Each parcel of land within the City was spatially linked to a unique pipe in the GIS model for both the water and sewer systems.

Table 2.2 Population Distributions by Service Areas and Neighbourhoods

Service Area	**Neighbourhoods**	**Population**	
		1999	2001 estimate
1	Riverdale	5,020	5,314
1	Hospital	40	40
2	Marwell	80	80
2	Trailer Park Areas (Range Road North)	1,056	1,450
1	Downtown	2,344	2,240
2	Valleyview	124	124
2	Takhini	536	539
3	Kopper King	316	316
2	Mc Intyre	528	534
2	Hillcrest	687	692
4	Granger	1,028	1,046
4	Arkell	270	270
4	Logan	183	178
5	Copper Ridge	554	750[1]
3	Porter Creek	4,091	4,139
3	Crestview	802	799
3	Porter Creek Extension	0	0
2	Lower Bench (Porter Creek)	0	0
5	Beyond Copper Ridge	0	0
2	White Pass Tank Farm	0	0
	Totals	**17,659**	**18,279**

2.4 POPULATION GROWTH SCENARIOS

Population projections are necessary to determine the sizing and staging of the water distribution system upgrades. In general growth projections involve certain assumptions and the accuracy of the growth projections is less reliable for longer-term projections. The uncertainty of population projections may not invalidate upgrading requirements but may affect the overall implementation timing.

Based on a review of the historical populations within the City of Whitehorse and consultation with City personnel, it has been determined that the annual population growth rate is 1.2%.

Figure 2.6. was reproduced from the latest Official Community Plan (OCP) which was completed in 2002. The figure indicates existing and future land use for the City of Whitehorse.

[1] Population for Copper Ridge was assumed due to high increase in recent years.

Existing population and growth information was obtained from the January 2002 OCP. Table 2.3 represents the projected population counts of the City of Whitehorse covering the period of 2002 to 2026.

Table 2.3 Projected Population Growth

	Population		
Year	**Low**	**Medium**	**High**
	0.50%	**1.20%**	**2.0%**
2001	18,279	18,279	18,279
2002	18,336	18,464	18,610
2003	18,428	18,686	18,982
2004	18,520	18,910	19,362
2005	18,613	19,137	19,749
2006	18,706	19,366	20,144
2007	18,799	19,599	20,547
2008	18,893	19,834	20,958
2009	18,988	20,072	21,377
2010	19,083	20,313	21,804
2011	19,178	20,556	22,241
2012	19,274	20,803	22,685
2013	19,370	21,053	23,139
2014	19,467	21,305	23,602
2015	19,564	21,561	24,074
2016	19,662	21,820	24,555
2017	19,761	22,082	25,046
2018	19,859	22,347	25,547
2019	19,959	22,615	26,058
2020	20,059	22,886	26,580
2021	20,159	23,161	27,111
2022	20,260	23,439	27,653
2023	20,361	23,720	28,206
2024	20,463	24,005	28,771
2025	20,565	24,293	29,346
2026	20,668	24,584	29,933

The population projections in Table 2.3 are based on the OCP utilizing a high, medium, and low growth rates. The growth rates were utilized to estimate timing of infrastructure upgrades. The City can monitor the growth in a particular area and adjust the Capital improvement timing.

The OCP estimated the following development phasing:

Phase 1	2002 to 2008
Phase 2	2009 to 2015
Phases 3 to 5	2016 and beyond

Table 2.4 shows the ultimate build out population capacities that are available for each neighbourhood and development phasing.

Table 2.4 Ultimate Population Capacities by Neighbourhood

Service Area	Region	Population			OCP Phasing	Notes
		2001 est.	Ultimate Capacity	Increase		
1	Riverdale	5,314	5,411	2%	Existing	
1	Hospital	40	40	0%	Existing	
1	Downtown	2,200	2,250	2%	Existing	
1	Riverdale Expansion	0	1,500	New	Not Identified	
2	Marwell	80	80	0%	Existing	
2	Takhini Trailer Park Areas (Range Rd North)	1,450	1,608	11%	1	
2	Valleyview	124	168	35%	Existing	Reflects an increase in people per lot
2	Takhini	539	1,163	116%	1	Infill and redevelopment
2	McIntyre	534	1,605	201%	Existing	Existing undeveloped lots and future area
2	Hillcrest	692	697	1%	Existing	
2	Lower Bench (Porter Creek)	0	7,200	New	2	
2	Tank Farm	0	1835	New	1	
3	Kopper King	316	322	2%	Existing	
3	Porter Creek	4,139	4,649	12%	Existing	Infill and redevelopment
3	Crestview	799	799	0%	Existing	No change
3	Porter Creek Extension	0	1,416	New	1	
4	Granger	1,046	1,046	0%	Existing	
4	Arkell	270	579	114%	Existing	Reflects an increase in people per lot
4	Logan	178	451	153%	Existing	Reflects an increase in people per lot
5	Copper Ridge	558	3,132	461%	Existing	Servicing in place for 1100 lots
5	Beyond Copper Ridge	0	6,000	New	3	
	Totals	**18,279**	**41,951**			

The ultimate capacity of all existing and planned neighbourhoods in the City of Whitehorse urban serviced areas is 41,951 persons.

Copper Ridge is the logical starting point for population increases since a majority of the infrastructure is currently in place. The OCP outlines other development locations and the projected timing of development.

In consultation with the City, growth horizons and Implementation Plans have been established as 0 to 5 years, 5 to 10 years, 10 to 20 years, and 20 years and beyond. Population growth horizons are outlined in Table 2.5.

Table 2.5 Growth Time Horizons

Time frame	Population Growth			Equivalent Increase to ADD Flows (L/s)		
	Low 0.5%	Medium 1.2%	High 2.0%	Low 0.5%	Medium 1.2%	High 2.0%
5 Years	463	1,135	1,937	2.9	7.2	12.3
10 Years	938	2,339	4,075	6.0	14.9	25.9
20 Years	1,923	4,975	9,043	12.2	31.7	57.6

It is difficult to predict population growth due to events, such as, economic fluctuations and resource and industry developments. Within a 20 year time period, the three identified growth rates will result in varying development scenarios.

Under the low growth rate it is expected that an additional 1,923 people will be added to the City. In theory Copper Ridge could accommodate this entire allotment.

The medium 20 year growth rate will result in an additional 4,975 people and the high growth rate will result in 9,043 people.

Growth will likely be distributed within Copper Ridge, Porter Creek Extension, Takhini, and the Tank Farm development. Table 2.6 outlines the major potential growth centres within the City.

Table 2.6 Major Growth Centres – Residential

					ADD flows	
Service Area	**Growth Centre**	**Existing**	**Ultimate Capacity**	**Increase**	**Existing**	**Ultimate**
2	Valleyview	124	168	35%	0.8	1.1
2	Takhini	539	1,163	116%	3.4	7.4
2	Mc Intyre	534	1,605	201%	3.4	10.2
4	Arkell	270	579	114%	1.7	3.7
4	Logan	178	451	153%	1.1	2.9
5	Copper Ridge	558	3,132	461%	3.6	19.9
1	Riverdale Expansion	0	1,500	New	0.0	8.0
3	Porter Creek Extension	0	1,416	New	0.0	9.0
2	Lower Poter Creek Bench	0	7,200	New	0.0	45.8
5	Beyond Copper Ridge	0	6,000	New	0.0	38.2
2	White Pass Tank Farm	0	1,835	New	0.0	11.7
	Major Centre Totals	**2,203**	**25,049**			

Ultimate system modeling was based on full build out capacity of existing and planned areas representing a total serviced population of 41,951 people. It is generally expected that neighbourhoods would fill in one or two at a time instead of an evenly distributed population growth in all proposed development areas. Timing of infrastructure upgrades will depend on staging of development.

Since ultimate build out capacity modeling was based on a potential capacity of 41,951 persons instead of a predicted population a detailed review of population projections and project phasing needs to be conducted prior to expansion of major facilities.

2.5 DEFINITION OF WATER DEMAND TERMINOLOGY

Water demands vary significantly within a given day and on a day-to-day basis throughout the year. The distribution system has to operate over a wide range of system demands and has to be able to deliver these demands within acceptable parameters.

The following six water demand categories in conjunction with fire flows are used to assess, storage, pumping, transmission and distribution infrastructure within a water system:

- Average Day Demand (ADD) – The ADD represents annual average daily demand. This value is determined by establishing total annual water consumption for the given year and dividing it by the number of days in that year. The average per capita demand is then established by dividing the ADD by the total population served.

- Maximum Day Demand (MDD) - MDD represents the maximum water consumption for a single day of the year. MDD is used to examine the ability of the water distribution system to maintain water service and fire flows.
- Peak Hour Demand (PHD) – PHD represents the maximum hourly demand of the maximum day consumption. The PHD rate is utilized to determine transmission main capacities and minimum system pressures.
- Maximum Five Day Demand (M5D) – M5D is the average of the five consecutive days of maximum consumption for a given year. If a system has no net loss of water during MDD, an M5D analysis is not likely necessary. M5D analysis allows supply shortfalls to be evaluated against storage.
- High Pressure / Reservoir Filling Demand (RFD) – RFD represents the lowest system demand period and is usually used to fill reservoirs. Low demand usually occurs at night and for this reason, High Pressure / Reservoir Filling Demand is commonly referred to as Night Filling Demand (NFD). The low demand period also represent the time of highest system pressures, as system head losses are lower at decreased flow rates.

In order to determine overall system ADD only yearly volumes are required whereas MDD, PHD, M5D and NDF requires accurate hourly pumping and reservoir flow data. Since accurate reservoir inflow and outflow data is not available, demand conditions were estimated based on past experience and known system conditions. Demands conditions are established in further detail in Section 2.6.

Water demand can be categorized into three sectors of water use as follows:

1) Composite Residential Demand is defined as the amount of water delivered to the system per day for use by individuals less Commercial-Industrial and Public Land Demands. Included in this demand is per-capita bleeder flow, and unaccounted water demand (system loss or leakage). Composite Residential Demand is expressed and the amount of water consumed in litres per capita per day (Lpcd)

2) Commercial-Industrial Demand is the amount of water required by commercial and industrial consumers on the system and is expressed in litres per hectare per day. Commercial-Industrial Demand includes retail centres, community halls, light industry, heavy industry, restaurants, hotels and service companies.

3) Public Lands Demand is defined as the amount of water required by municipal, territorial and federal institutions. Included in this demand category are schools, government offices and institutional buildings. The amount of water required by Public Lands Demand is expressed litres per hectare per day. Where possible, high individual public land demands such as hospitals or colleges are separated and given a specific demand rate.

This report contains references to existing, future and ultimate build out capacity time frames. The definition of each tense is given below.

- Existing refers to conditions or facilities that are presently being used by the City or within the City. Existing can refer to the system networks as they exist today or refer to non-tangible items such as current demand rates.
- Future is a general time frame between the existing conditions and the ultimate build out capacity. This time frame is left ambiguous for general discussion. Specific timing will be dependent on development phasing. Time frames, population increases or flow increases may be used to reference specific upgrade requirements.
- Ultimate build out capacity refers to total population capacity for all existing and future development areas as identified in Table 2.4. Ultimate can also refer to non-tangible items such as demand rates at the time of total system build out.

It is important to clarify that the ultimate systems referred to within this report may not be the finite limits of development. Ultimate represents the possible extents of all identified developments. Ultimate build out capacity provides the full time frame for this study.

2.6 WATER DEMANDS

2.6.1 General

The City of Whitehorse has several meters throughout the distribution system in order to acquire flow data. Flow data recorders used by this study are located at Selkirk Pumphouse, Two Mile Hill Booster Station, McIntyre Creek Pumphouse, Crestview Pumphouse, Hamilton Boulevard Pumphouse and Copper Ridge Pumphouse. Flow records along with past studies form the basis of determining the water demands for the City of Whitehorse.

The boundaries established by these pumphouses were used to separate the City into the metered service areas as discussed in Section 2.2.1. Flow meter volumes are inclusive of all residential, commercial/industrial, public lands consumption and system losses due to leakage and bleeders.

Many commercial/industrial customers within the City are equipped with individual flow meters. Stantec obtained the individual meter billing records for the years 2000 and 2001. These records were utilized to establish the commercial/industrial and public lands demands. The demand data was spatially located by address for higher volume meters. A total of 411 meter demands were spatially located of the 585 meters records in the database. The 411 meter demands modeled represents 92% of the total metered volume (1.27 million m^3 in 2001).

The City produces annual reports on the water use and sewage treatment as part of the City's water licensing requirements. The recorded and reported data consists of pump hour readings, pump flows and system flow totals.

The City of Whitehorse carried out a preliminary review of pumhouse recorded flows for 1999 and 2000. The analysis of data provided by the City indicated that some of the pumphouse meters were at times not working properly or not working at all. The data set provides indication of the water consumption within the City but some assumptions were required as described in later sections.

2.6.2 Historical Water Demand

The records available for the study include the information provided by the 1990 study plus every year since up to and including 2001. The data set consists of pump hour readings, pump flows and flow totals.

The average day demand, maximum day demand and the ratio of the maximum day demand to the average day demand for each year data is available is shown in Table 2.7.

Table 2.7 Water Consumption Records for the City of Whitehorse

Year	Population	Average Day Demand (ADD)			Maximum Day Demand (MDD)			MDD/ADD
		Lpcd	m^3/d	L/s	Lpcd	m^3/d	L/s	
1978	13,000	1,262	16,406	190	1,741	22,633	262	1.38
1979	13,065	1,169	15,273	177	1,764	23,047	267	1.51
1980	12,975	1,211	15,713	182	1,795	23,290	270	1.48
1981	13,500	980	13,230	153	1,528	20,628	239	1.56
1982	13,300	890	11,837	137	1,376	18,301	212	1.55
1983	12,507	889	11,119	129	1,535	19,198	222	1.72
1984	13,085	967	12,653	146	1,421	18,594	215	1.47
1985	NO DATA AVAILABLE							
1986	13,730	1,112	15,268	177	1,597	21,927	254	1.44
1987	14,250	1,087	15,495	179	1,731	24,669	286	1.59
1988	14,880	1,080	16,070	186	1,470	21,874	253	1.36
1989	15,375	1,086	16,697	193	1,789	27,506	318	1.65
1990	16,720	992	16,587	192				
1991	17,046	921	15,698	182				
1992	17,942	775	13,909	161	1,353	24,278	281	1.75
1993	18,810	780	14,679	170				
1994	18,554	822	15,247	176				
1995	18,712	849	15,888	184				
1996	19,311	808	15,604	181	1,226	23,674	274	1.51
1997	19,718	797	15,715	182				
1998	19,106	814	15,547	180	1,402	26,784	310	1.72
1999	18,617	822	15,301	177	1,453	27,043	313	1.77
2000	18,349	738	13,542	157	1,398	25,661	297	1.89
2001	18,245	691	12,608	146	1,435	26,179	303	2.08

Figure 2.7 shows a plot of the average day demand for the city expressed in L/s for historic data available.

Figure 2.7 – Yearly Average Day Demand in L/s

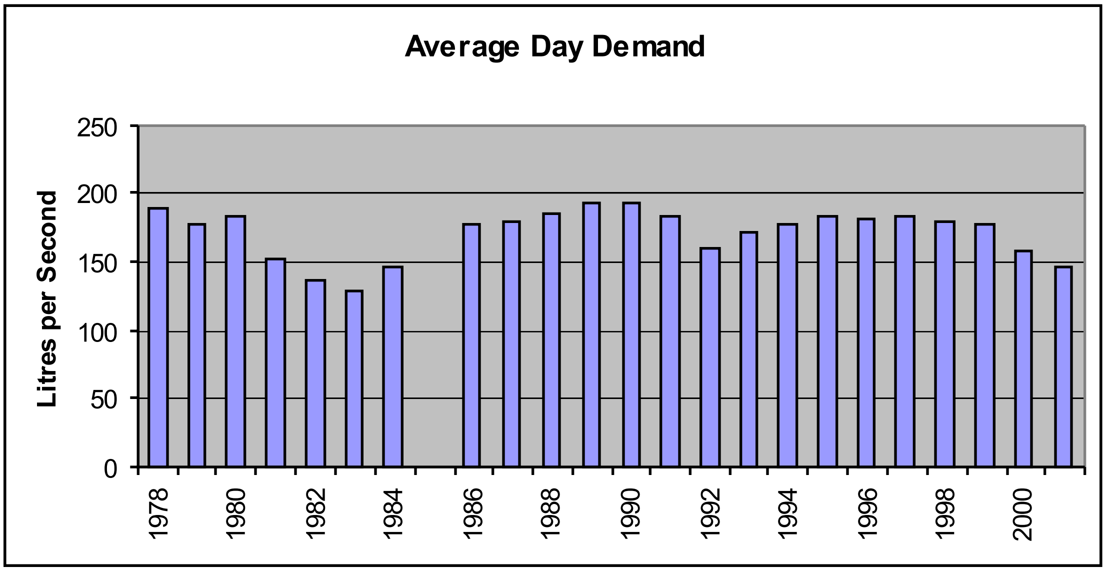
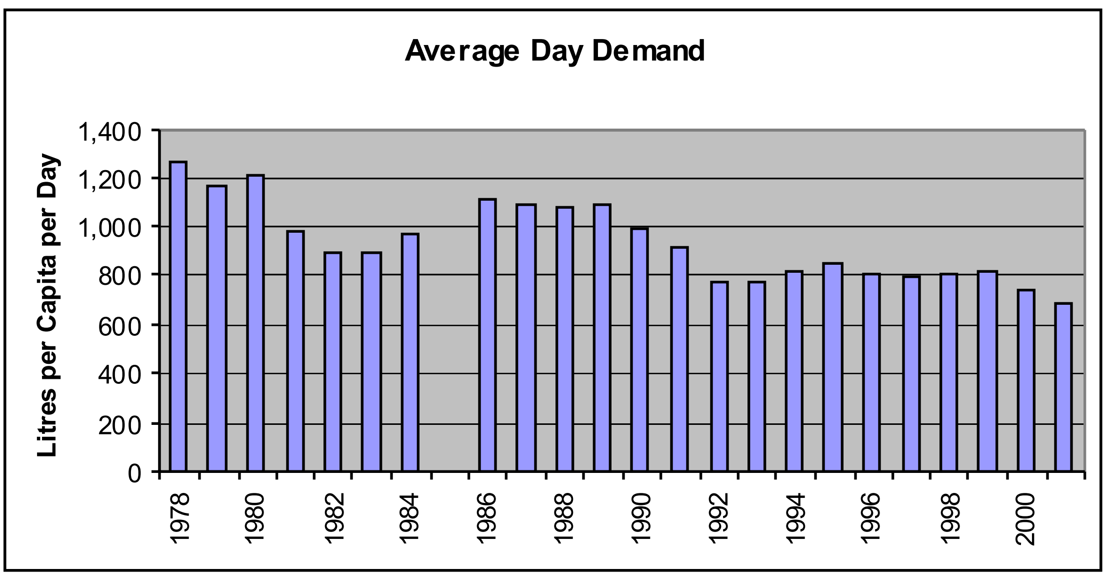


Figure 2.8 shows a plot of the average day demand for the city expressed in litres per capita per day for historic data available.

Figure 2.8 Yearly Average Day Demand in Litres per Capita per Day



It is evident from Tables 2.7 and 2.8 that water consumption is decreasing. The decrease in water demand can be attributed to water conservation, bleeder reduction programs, replacement of old infrastructure and increased leak detection and repair.

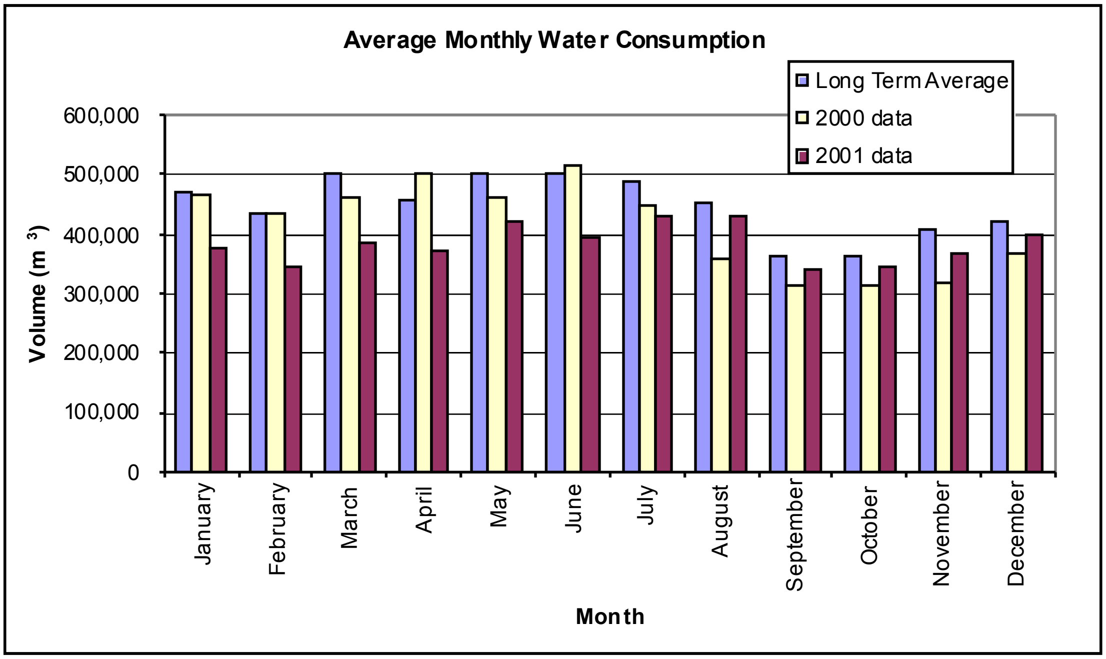
In older neighbourhoods, the City of Whitehorse water distribution system relies heavily on the use of bleeders to prevent the system from freezing. Bleeding occurs at individual residences and commercial establishments for service and fire hydrants protection. Dead end mains are also bled for watermain protection. Bleeders drain directly into the sanitary sewer system. Bleeders can represent a significant portion of the water demand during winter if not properly managed. Newer developments rely on watermain looping, water circulation pumps and service heat tracing for frost protection.

Although maximum day demand (MDD) flow rate is expected to increase as population increases it has not been increasing proportionally for Whitehorse. This is likely due to the reduction of water loss and water conservation efforts. It should also be noted that the MDD/ADD ratio has been increasing. This is not surprising if system losses and bleeder flow has been reduced.

MDD/ADD ratios vary widely in Table 2.7. The MDD/ADD ratios range between 1.36 and 2.08 with an average value of 1.5. The historic average is much lower than what is considered typical of southern water distribution systems (typical values are in the order of 2.0). The lower average ratio was likely due to the high "base load" on the City's system due to bleeders and system leakage. It can be noted that the MDD/ADD ratio is trending towards 2.0 in recent years with a ratio of 2.08 recorded in 2001.

Historic water demand data was reviewed to determine the distribution of monthly demands. Average monthly demands typically reach a maximum during May through July and decrease significantly to a minimum during the months of September to November. Water demands increase significantly during the winter months (December to April) presumably due to the use of residential and City bleeders to prevent services and watermains from freezing. Figure 2.9 shows the annual flow by month for the period 1978 to 2001 and individual volumes for the years 2000 and 2001. The results from 2001 look promising in terms of overall bleeder reduction. This may be attributable to the installation of residential thermostatically controlled bleeders (TCB) in 1999 and 2000.

Figure 2.9 - Average Monthly Water Consumption



In order to continue with flow reduction strategies, the City should consider implementation of residential flow metering. Since water metering is politically difficult to implement, the City could create an incentive for homeowners to switch to metering. A staged increase in the flat rate water and sewer assessment can be utilized. Installation of water meters can be made voluntary but it should be cost beneficial for the average homeowner to switch to water metering. Due to warranty requirements, homeowners should be made responsible for the installation of water meters but the City may consider a grant program or rebate on the homeowners sewer and water bill.

High water use customers, such as, those with high bleed rates for frost protection or wasteful customers may not voluntarily switch to water metering; therefore an all or nothing approach to water metering may still be warranted. More study into water metering may be beneficial.

2.6.3 Bleeders

The City of Whitehorse water distribution system relies heavily on the use of bleeders in order to prevent freezing of watermains, hydrants, water services, sewer mains and sewer services within older areas. Bleeding occurs at the individual residences, commercial establishments and un-looped or dead-end watermains, trouble sewer mains and at trouble sewer services. The older areas of Riverdale, Downtown and Hillcrest have offline fire hydrants. These offline hydrants rely on residential or commercial water services from the hydrants to provide adequate bleeder and consumption flow rates for frost protection.

Bleeders drain directly into the sanitary sewer system and represent a significant portion of the water demand and sewer generation for the City and form a high base load on the system.

There are four types of bleeders used within the city of Whitehorse as follows:

1. City free flow bleeders to protect watermains or problem sewer mains
2. City thermostatically controlled bleeders (TCB) to protect watermains.
3. Privately operated free flow bleeders to protect water services, problem sewer services and City fire hydrants
4. Private thermostatically controlled bleeders (TCB) to protect water services and City fire hydrants

City Free Flow Bleeders

City free flow bleeders consist of a 19 mm copper water service installed in a sanitary manhole. A combination of a reduced pressure backflow assembly and an additional check valve is used to secure against sanitary sewer cross-connection. Flow is restricted by means of a “Dow” valve.

The City provided a list of active winter free flow City bleeders that is shown in Table 2.8.

Table 2.8 Active City Operated Free Flow Bleeders [2]

MH #	Location	Neighbourhood	Flow (L/s)	
			Measured	**Modeled**
84	Main Street, West of 6th Avenue across from an apartment block	Downtown	0.50	0.5
133	Strickland Street, West of 8th Avenue	Downtown	0.05	0.5
144	Alexander Street, West of 8th Avenue	Downtown		0.5
159	Black Street, West of 8th Avenue	Downtown	0.76	0.76
173	Wheeler Street, West of 8th Avenue	Downtown		0.76
180	Cook Street, West of 8th Avenue	Downtown		0.76
190	Third Avenue, North End	Downtown		0.5
192	Ogilvie Street, East of 8th Avenue	Downtown	0.50	0.5
Total – Service Area 1				**4.78**
136	Adorna, Alaska Highway	Takhini		0.5
66	Burns Road	Hillcrest		1.0
Total – Service Area 2				**1.5**
45	Sycamore Street	Porter Creek		1.0
53	Aspen Drive	Porter Creek	1.0	1.0
54	Ebony Drive	Porter Creek		0.5
Total – Service Area 3				**2.5**
Total All City Free Flow Bleeders				**8.78**

Isolating bleeder demand can help identify and analyse the overall sources of water consumption. Based on the bucket tests and extrapolation of the data a total annual City free flow bleeder volume can be estimated. The calculation assumes that the bleeders are on for a 6 month period from mid November to mid May. Table 2.9 indicates total annual City free flow bleeder volume.

Stantec

[2] Table 2.8 provides the active free flow bleeder list. There are additional free flow bleeders that are no longer activated. Also there are a few residential bleeders that home owners are required to operate in order to protect City systems.

Table 2.9 Estimated Annual City Free Flow Bleeder Volume

Service Area	Total Annual Bleeder Volume (m^3)	Total Annual Demand Volume (m^3)	Bleeder Volume Percentage
1	75,371	2,659,000	2.83
2	23,652	829,000	2.85
3	39,420	984,000	4.01
Total	**138,443**	**4,472,000**	**3.10**

Based on the 2001 total annual City water volume of 4,601,848 m^3, City free flow bleeder flow represents 3.1 percent of total water consumption.

City Thermostatically Controlled Bleeders

City TCB's are similar to free flow bleeders but they also incorporate control, temperature sensors and a fail open solenoid valves to limit flow to only when water main temperatures fall below a set point. The following provides a brief description of the City operated TCB's.

a) Drury Street Bleeders Stations (Service Area 1 – Downtown)

Drury Street Bleeder Station is located on a dead-end water main on Drury Street in the southwest corner of Downtown in Service Area 1.

City TCB bleeder chambers consist of a control kiosk, a 19 mm copper water service installed in a watermain access manhole and a bleeder to a sanitary manhole. The stations accommodate a bleeder solenoid valve and temperature sensor to monitor and bleed the water line when the watermain temperature drops below 1.5 ^{o}C in the winter. Bleeding will stop when the watermain temperature rises above approximately 1.7 ^{o}C. The solenoid valves are the power closed type so that the valve will open on power failure.

b) Klukshu Avenue Bleeder Stations (Service Area 3 – Crestview)

Klukshu Avenue Bleeder Station is located in Service Area 3 and is used in the older area of Crestview Neighbourhood to remedy poor watermain circulation. Klukshu Avenue Bleeder Station is a similar to the Drury Street Bleeder Station. The station will bleed water when the temperature drops below 1.5 ^{o}C in the winter. Bleeding will stop when the temperature rises above approximately 1.7 ^{o}C.

c) Kathleen Road Bleeder Station (Service Area 3 – Crestview)

Kathleen Road Bleeder Station is located in Service Area 3 and is used in the older area of Crestview Neighbourhood on a dead-end watermain. Kathleen Road Bleeder Station is a similar to the Drury Street Bleeder Station. The station will bleed water when the temperature drops below 1.5 ^{o}C in the winter. Bleeding will stop when the temperature rises above approximately 1.7 ^{o}C.

A bucket test was performed resulting in a measured flow of 77.7 L/min (1.3 L/s). The 1999 Pumpho Consumption Audit Study recommended to conduct a hydraulic study to verify the need of this chamber as it has been reported that the water temperature in this area rarely drops below 1.5 ^{o}C during the winter. The station should be maintained as a safety precaution since the station will not bleed when the water temperature is greater than 1.5 ^{o}C.

The City of Whitehorse has in recent years allowed Kathleen Bleeder Station to flow free during summer months in order to refresh water in the area when chlorine residuals get too low. It is recommended that the City investigate improved circulation, re-chlorinization or use only intermittent bleeding for water quality.

Private Free Flow Bleeders

Privately operated free flow bleeders are constructed of a small diameter copper line to restrict flow. Proper installations should also include a small flow restriction orifice.

The majority of private free flow bleeders have been replaced by private TCB's in recent years; however, a few property owners did not participate in the program. Further, there are a number of free flow bleeders that are operated on trouble systems such as the properties along Copper and Quarts Roads with shallow services or properties where other frost protection has failed. A number of illegal bleeders can also be expected, as residences are not metered.

It was not possible to estimate the amount of bleed water created by free flow bleeders as the City does not have control over operation, various bleed rates can be expected and there is no completed record of locations.

Private Thermostatically Controlled Bleeders

The City of Whitehorse has installed thermostatically controlled bleeders (TCB) to replace free flow residential bleeders in the neighbourhoods of Riverdale (Service Area 1), Downtown (Service Area 2) and Hillcrest (Service Area 3). The object of the TCB program was to reduce the volume of bleed water by up to 90% over traditional residential free flow bleeders. From August 1999 to March of 2000, a total of 1,180 TCB units were installed.

Depending upon unit set up, private TCB's operate in various water temperature dependent modes that vary bleed time and length of time between bleed.

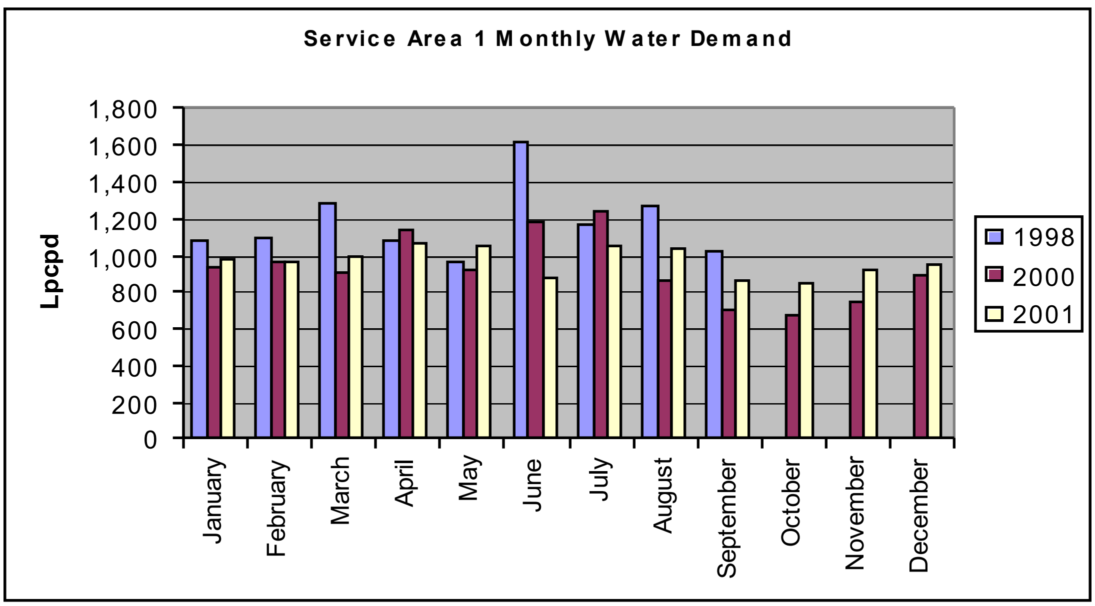
Since TCB's operate intermittently and residences are not metered, it is difficult to quantify the water consumption that can be attributed solely to residential TCB's. An analysis was conducted based on total monthly water usage for Service Area 1 to estimate water conservation resulting from replacement of free flow bleeders with TCB's. Table 2.10 indicates monthly water consumption and Lpcd (composite residential demand, commercial/industrial demand and public lands) for 1998, 2000 and 2001.

Table 2.10 Service Area 1 Monthly Water Demand[3]

Month	1998		2000		2001	
	(m^3)	Lpcd	(m^3)	Lpcd	(m^3)	Lpcd
January	253,390	1,082	219,045	935	228,229	975
February	230,084	1,088	204,593	967	205,115	970
March	301,723	1,288	212,061	906	230,280	983
April	244,166	1,077	259,338	1,144	239,849	1,058
May	223,804	956	213,162	910	245,132	1,047
June	365,478	1,613	267,597	1,181	197,819	873
July	271,214	1,158	289,122	1,235	245,072	1,047
August	297,369	1,270	201,840	862	243,908	1,042
September	231,129	1,020	159,220	703	193,714	855
October	NO DATA	NO DATA	155,838	665	199,719	853
November	NO DATA	NO DATA	169,157	746	208,060	918
December	NO DATA	NO DATA	210,433	899	221,836	947
Yearly Total	**3,622,436**	**1,314**	**2,561,407**	**929**	**2,658,733**	**964**

Figure 2.10 provides a trend for the data presented in Table 2.10.

Figure 2.10 Service Area 1 Monthly Water Demand



[3] Service Area 1 volumes were determined by subtracting Selkirk flow data from Two Mile Booster flow data. Data was not available for 1999 at Two Mile Booster and the existing Two Mile Booster did not exist prior to 1998. Lpcd were based on a service population of 7,554 people and includes composite residential demand, commercial/industrial demand and public lands demand.

Demand reduction during the winter and spring from January to May was an expected result of residential TCB installation. Since TCB's automatically quit bleeding during summer a small reduction should also be expected to account for people not turning off their free flow bleeders; however, summer demand fluctuations are more closely linked to climatic conditions. It should be noted that August and September 2000 were unusually wet months and could account for reduced water demand.

There have been a very small percentage of TCB failures resulting in customer service freezing. Some of the units seemed to have lost the temperature calibration and were therefore reading a warmer water temperature than was actual. A small percentage of other units were installed on services that were very susceptible to freezing. The City of Whitehorse Engineering and Public Works Departments have been addressing these issues as they arise and are still providing TCB warranty. Solutions should continue to be found in order to maintain customer confidence in the units. The operations staff has reported that some customers have been unplugging the units so that they free flow continuously. Free flowing the TCB units for frost protection is not a good idea because the bleeders were sized to quickly change the water in the service and as a result, TCB's can bleed much more water than the original free flow bleeders they replaced.

In situations where it is determined that services are highly susceptible to freezing free flow bleeders have been reinstalled; however, flow rated bleeders should be utilized and sized according to estimated time to freeze. The City of Whitehorse Meter Technician has been recently investigating the supply of small diameter flow rated orifices for bleeders.

Implementation of a residential water metering program would help ensure that customers are operating bleeders properly.

2.6.4 Existing Average Day Demand

Metered pump stations were utilized to determine Average Day Demand. Based on information from operations staff, some flow meter results may be suspect. The City currently does not have a structured meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement, calibration and data collection. Some work has been recently undertaken to install new meters and get non-functioning or improperly function meters operating but the metering program still needs improvement.

The flow meter at Selkirk pump station is very primitive. The metering is based on a differential pressure across and online orifice plate. According to operations staff, there has been no method of accurately calibrating this meter and an operations manual cannot be found. It should be noted that there is a potential for a high degree of error associated with the data obtained from this meter. Since this meter represents total City water consumption, it is recommended that this meter be replaced prior to design of a water treatment plant. One replacement option would be to install a meter external to the station that can be maintained after construction of a treatment plant.

The recorded flows at the Selkirk pump station were analysed to derive estimated yearly total flows for the City. These total flows were then divided by the number of days in a year to establish the systemwide average day demand.

Selecting 2000 as being a representative year for determination of flows per area, the totalizer readings for the seven metered pump stations were reduced to arrive at the average day demand for each metered water service area.

Non-residential demand rates were determined by dividing the available individual metered flows by the land areas of the users. The values were then compared with traditional published values for similar industries and uses and modified slightly to arrive at the values used for the City of Whitehorse. Table 2.11 presents the average day demand for non-residential demand (Commercial, Industrial and Public Lands).

Table 2.11 Non-Residential Areas Average Day Demand

Neighbourhood	Service Area	Estimated Average Day Demand (m^3/ha/day)
Downtown	1	32
Riverdale	1	24
Porter Creek	3	11
Hillcrest / Airport	2	8
Marwell	2	4
Others		3

Upon establishment of the total non residential average day flows, these totals plus the estimated annual bleeder volumes were subtracted from the average flow per metered area. The remaining flows represent the composite residential portion of the daily average demand. Dividing this by the service population results in the composite residential per capita demand. Table 2.12 shows the composite average day residential demand for each metered area. Composite residential demand is the total of residential consumption, TCB's and unaccounted water loss.

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Table 2.12 Composite Average Day Residential Demand[4]

Service Area	1990 Report (Lpcd)	2001 Calculated (Lpcd)	Used In Model (Lpcd)
Area 1 (Riverdale, Downtown, Hospital)	1,100	597	600
Area 2 (Airport, Hillcrest, Valleyview, Takhini, McIntyre)	750	358	400
Area 3 (Porter Creek, Crestview, Kopper King)	750	492	500
Area 4 (Arkell, Logan, Granger)	n/a	358	400
Area 5 (Copper Ridge)	n/a	358	400

It is apparent from Table 2.12 that water conservation efforts since the 1990 study has provided a significant decrease to composite residential demand.

Table 2.13 represents the completed analysis of existing flows by area.

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[4] Areas 2, 4 and 5 were assessed as one area due to the metering errors at the Hamilton Boulevard Pumphouse and Copper Ridge Pump Pumphouse.

Table 2.13 Existing Flows by Service Area and Neighbourhood

Area	Serviced Pop	Composite Residential Demand Lpcd	Composite Residential ADD L/s	Non Residential Area ha	Non Residential Rate m^3/ha/day	Non Residential ADD L/s	Total ADD L/s	Total MDD L/s
Area 1								
Riverdale	5314	600	36.9	30	24	8.3	45.2	90.5
Riverdale Expansion	0	0	0.0	0		0.0	0.0	0.0
Downtown	2200	600	15.3	49.7	32	18.4	33.7	67.4
Hospital	40	600	0.3	19	24	5.3	5.6	11.1
Area 1 Totals	7554		52.5	98.7		32.0	84.5	169.0
Area 2								
Marwell	80	400	0.4	91.2	4	4.2	4.6	9.2
Takhini	539	400	2.5	84	3	2.9	5.4	10.8
Hillcrest	692	400	3.2	8.9	8	0.8	4.0	8.1
Valleyview	124	400	0.6	0		0.0	0.6	1.1
McIntyre	534	400	2.5	2.6	3	0.1	2.6	5.1
Trailer Parks	1450	400	6.7	0		0.0	6.7	13.4
Lower Porter Creek	0	0	0.0	0		0.0	0.0	0.0
White Pass Tank Farm	0	0	0.0	0		0.0	0.0	0.0
Airport	0	0	0.0	26.5	8	2.5	2.5	4.9
Area 2 Totals	3419		15.8	213.2		10.5	26.3	52.7

Table 2.13 Existing Flow by Area (Continued)

Area	Serviced Pop	Composite Residential Demand	Composite Residential ADD	Non Residential Area	Non Residential Rate	Non Residential ADD	Total ADD	Total MDD
		Lpcd	L/s	ha	m^3/ha/day	L/s	L/s	L/s
Area 3								
Porter Creek	4139	500	24.0	11.6	11	1.5	25.4	50.9
Crestview	799	400	3.7	0		0.0	3.7	7.4
Kopper King	316	500	1.8	2	11	0.3	2.1	4.2
Porter Creek Extension	0		0.0	0		0.0	0.0	0.0
Area 3 Totals	5254		29.5	13.6		1.7	31.2	62.4
Area 4								
Granger	1046	400	4.8	7.2	3	0.3	5.1	10.2
Arkell	270	400	1.3	0		0.0	1.3	2.5
Logan	178	400	0.8	0		0.0	0.8	1.6
Area 4 Totals	1494		6.9	7.2		0.3	7.2	14.3
Area 5								
Copper Ridge	558	400	2.6	14.5	3	0.5	3.1	6.2
Beyond Copper Ridge	0		0.0	0		0.0	0.0	0.0
Area 5 Totals	558		2.6	14.5		0.5	3.1	6.2
Existing System Totals	18279		107.3	347.2		45.0	152.3	304.6

2.6.5 Existing Maximum Day Demand

The available water records were further analysed to establish the existing maximum day demand for each service area of the city. The maximum day demand and the ratio of the maximum day demand to the average day demand (MDD/ADD) for the recorded flows of each year were presented in Table 2.7. The historical data indicates a MDD/ADD average ratio of 1.5; therefore, the 1990 study used a ratio of 1.5.

It is evident since the 1990 study that the ratio of maximum day demand to average day demand is moving towards 2.0; therefore, 2.0 x ADD was chosen for existing MDD model runs.

Assessment of a “true” maximum day demand was not possible since the meters at the major stations were not read every day. As a result the volumes are averaged over the days with no readings. Further, reservoir inflow and outflow totals are also required to determine system balance. With the advent of recent and current SCADA projects flow meter data collection is being improved. SCADA provides and records almost instantaneous flow measurements for the SCADA upgraded water stations of Selkirk Pumphouse, Two Mile Booster Station and Copper Ridge Pumphouse. With the exception of Two Mile Booster Station, the 2000 data set used in this study did not yet benefit from SCADA upgrades.

Only Copper Ridge and Valleyview Reservoirs have inflow and outflow monitoring capabilities but good data was not available for this study. It is recognized that during a maximum day flow the system can draw water from reservoirs and therefore not be recorded as maximum day flows at the pump stations. Using the averaged volume at the pump stations can compensate for this occurrence.

2.6.6 Ultimate Average Day Demand

The average day demands for the ultimate build out capacity system were determined using the Official Community Plan for the City of Whitehorse and the population and neighbourhood capacity projections established in Section 2.4.

The composite residential demand was assumed to remain at the same base rates for the existing areas. All new areas were selected to be at a lower rate of 500 Lpcd based on City Servicing Standards.

Non-Residential demands were selected to remain at the same rates while the developed areas were increased to match the Official Community Plan.

Table 2.14 details the average day demands for the ultimate build out capacity loading conditions.

Table 2.14 Ultimate Average Day Demand

Service Area	Development Area	Loading
Area 1	Existing areas	600 Lpcd
	Riverdale Expansion	500 Lpcd
	Non residential	30 m^3/ha/day
Area 2	Existing and new areas	500 Lpcd
	Non residential	10 m^3/ha/day
Area 3	Existing and new areas	500 Lpcd
	Non residential	10 m^3/ha/day
Area 4	Existing and new	500 lpcd
	Non residential	10 m^3/ha/day
Area 5	Existing and new	500 lpcd
	Non residential	10 m^3/ha/day

2.6.7 Ultimate Maximum Day Demand

One trend established in the analysis of the historic data was that as the overall per capita flow decreased the maximum day factor increased. This follows the general rules established for the analysis of municipal water supply systems. Ultimate build out capacity maximum day demand (MDD) was established as 2 times ultimate average day demand (ADD) for model runs.

Table 2.15 details the ultimate build out capacity water demands for the water distribution system. The table takes into account the decrease in per capita flows for the new development areas as well as the increased maximum day factor. Non residential areas are subjected to the same peaks as residential areas.

Table 2.15 Ultimate Flows by Service Area and Neighbourhood

Area	Served Pop	Composite Residential Demand lpcd	Composite Residential ADD l/s	Non Residential Area ha	Non Residential Rate m3/ha/day	Non Residential ADD l/s	Total ADD l/s	Total MDD l/s
Area 1								
Riverdale	5411	600	37.6	30.8	30	10.7	48.3	96.5
Riverdale Expansion	1500	500	8.7	10	10	1.2	9.8	19.7
Downtown	1250	600	15.6	63.5	30	22.0	37.7	75.3
Hospital	40	600	0.3	19	30	6.6	6.9	13.8
Area 1 Totals	9201		62.2	123.3		40.5	102.7	205.3
Area 2								
Marwell	80	500	0.5	105.5	10	12.2	12.7	25.3
Takhini	1163	500	6.7	84	10	9.7	16.5	32.9
Hillcrest	697	500	4.0	9.4	10	1.1	5.1	10.2
Valleyview	168	500	1.0	0	10	0.0	1.0	1.9
McIntyre	1605	500	9.3	8.6	10	1.0	10.3	20.6
Trailer Parks	1608	500	9.3	0	10	0.0	9.3	18.6
Lower Porter Creek	7200	500	41.7	25	10	2.9	44.6	89.1
White Pass Tank Farm	1835	500	10.6	10	10	1.2	11.8	23.6
Airport	0	500	0.0	61.8	10	7.2	7.2	14.3
Area 2 Totals	14356		83.1	304.3		35.2	118.3	236.6
Area 3								
Porter Creek	4649	500	26.9	63	10	7.3	34.2	68.4
Crestview	799	500	4.6	0	10	0.0	4.6	9.2
Kopper King	322	500	1.9	19	10	2.2	4.1	8.1
Porter Creek Extension	1416	500	8.2	0	10	0.0	8.2	16.4
Area 3 Totals	7186		41.6	82.0		9.5	51.1	102.2
Area 4								
Granger	1046	500	6.1	7.5	10	0.9	6.9	13.8
Arkell	579	500	3.4	0	10	0.0	3.4	6.7
Logan	451	500	2.6	0	10	0.0	2.6	5.2
Area 4 Totals	2076		12.0	7.5		0.9	12.9	25.8
Area 5								
Copper Ridge	3132	500	18.1	14.5	10	1.7	19.8	39.6
Beyond Copper Ridge	6000	500	34.7		10	0.0	34.7	69.4
Area 5 Totals	9132		52.8	14.5		1.7	54.5	109.1
Ultimate System Totals	41951		251.7	531.6		87.8	339.4	678.9

2.6.8 Peak Hour Demand

The available water records are further analyzed to establish the peak hour demand for the City of Whitehorse. In order to obtain a true representative peak hour demand several years of data is required at a resolution of at least one-hour increments. The flow must then be isolated to each particular service area and a mass balance performed on the data. Flow meters at all pump stations and reservoirs would be required. Each booster and reservoir must be monitored simultaneously and the net flow calculated. The City has some information available within the newly installed SCADA system. Unfortunately the data was not adequate to perform a proper assessment for all service areas.

Since limited data exists for the determination of peak hour demand the values for peak hour demand will be consistent with the 1990 study and other subsequent studies utilizing 1.5 x MDD or 3 x ADD.

2.6.9 High Pressure / Reservoir Filling Demand (Night Filling Demand)

The flows required to fill the reservoirs is based on the ability of the system to fill the reservoirs during the low demand hours at night (night filling demand NFD) or high pressure / reservoir filling demand. The factor is expressed as a function of the maximum day demand and is typically equal to 0.25-0.35 times the consumptive portion of the maximum day demand. Again for consistency with the 1990 study and other subsequent studies a factor of 0.3 times the MDD (0.6 x ADD existing) for the existing and 0.25 (0.5 x ADD ultimate) for the ultimate build out capacity system plus bleeder flows was utilized.

As a general rule, reservoirs should be filled at a rate between 1.8 and 2.0 times ADD to provide MDD recovery and facilitate an adequate volume turnover for the reservoirs.

2.6.10 Water Demand Management

Canada has a high per capita water demand in comparison to European nations and the City of Whitehorse has a high per capita water demand as compared to other Canadian municipalities.

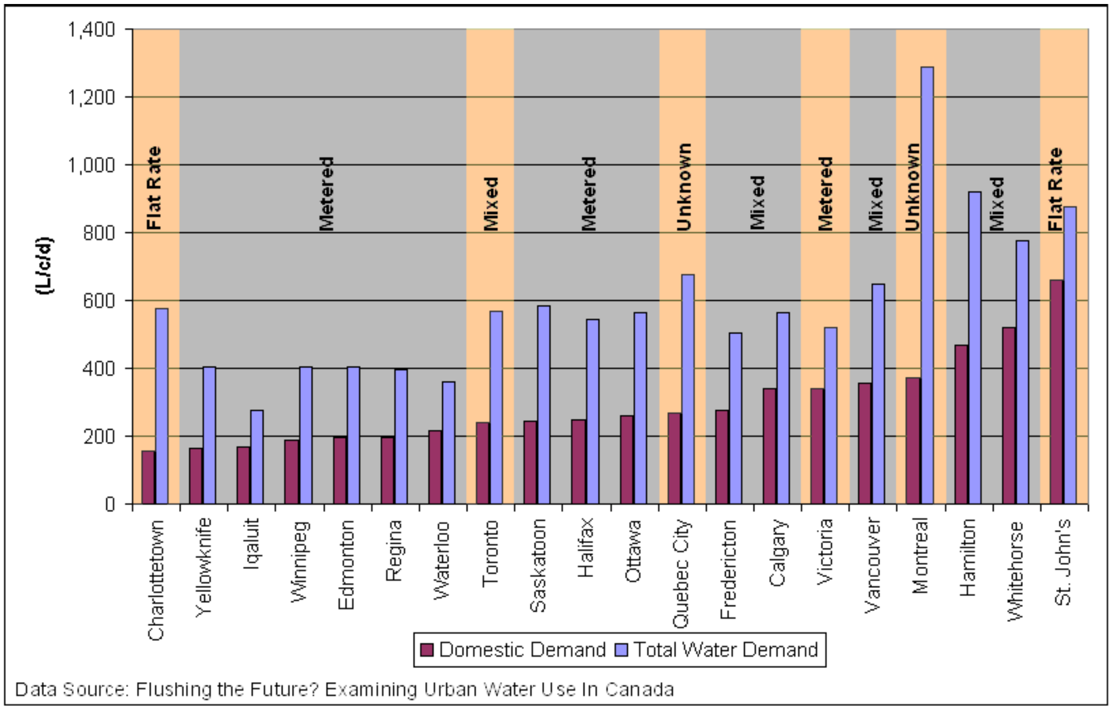
Table 2.17 provides 1999 water use data for a sample of 20 Canadian municipalities. The data is based on the 2003 Flushing the Future? Examining Urban Water Use in Canada, Polis Project on Ecological Governance, the University of Victoria report.

Table 2.17 Canadian Municipality Water Consumption (1999)

Municipality	Population	Domestic Water Use (L/c/d)	Degree Metered	Pricing Structure	Total Municipal Water Use (L/c/d)
Charlottetown	28,600	156	0%	Flat Rate	578
Yellowknife	17,250	164	100%	Volume	406
Iqaluit	4,500	167	100%	Volume	278
Winnipeg	620,000	190	100%	Volume	403
Edmonton	636,000	195	100%	Volume	406
Regina	190,000	197	100%	Volume	395
Waterloo	78,000	215	100%	Volume	359
Toronto	2,393,790	239	73%	Volume / Flat	568
Saskatoon	207,000	245	100%	Volume	583
Halifax	280,000	247	100%	Volume	544
Ottawa	336,269	259	100%	Volume	563
Quebec City	167,300	270	0%	Not Available	675
Fredericton	45,000	278	98%	Volume / Flat	505
Calgary	819,334	339	57%	Volume / Flat	566
Victoria	86,000	340	100%	Volume	519
Vancouver	554,000	357	1%	Volume / Flat	650
Montreal	1,030,678	373	0%	Not Available	1,287
Hamilton	322,252	470	65%	Volume / Flat	921
Whitehorse	200,000	519	50%	Volume / Flat	775
St. John's	106,000	659	0%	Flat Rate	878

Figure 2.11 charts the data provided in Table 2.17.

Figure 2.11 Canadian Municipality Water Consumption (1999)



It is evident from Table 2.17 and Figure 2.11 that Whitehorse is a high water demand city as compared to other Canadian municipalities. High demands for any municipality can be attributed to a number of factors, such as, climate, lack of metering, cost to consumer, conservation efforts and water leakage.

A very important alternate approach to infrastructure upgrade due to increased demand is demand side management. Reducing water demand and resulting sewage generation rates can significantly defer or reduce Capital outlay requirements while providing sound environmental management.

Demand side management for water demand reduction can be divided into two general categories of Means and Policy Instruments. Means for reducing water demand may include taking physical measures such as the use of water efficient fixtures and repairing leaks, and changing consumer water use habits and values. Policy instruments may include education and economic incentives.[5]

The City of Whitehorse should develop a water demand management strategy that encompasses: leak detection and repair, bleeder reduction, education, metering, rate structuring, economic incentive, regulation, politics, and plumbing fixtures.

A monitoring program and water sewer consumption audit is warranted in order to establish firm water demand rates and aid in consumption reduction programs. A

[5] *What the Experts Think: Understanding Urban Water Demand Management in Canada, Polis Project on Ecological Governance, the University of Victoria.*

meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement, calibration and data collection in each service area should be established prior to a water audit. In addition to water meters at pumphouses, water meters should be considered at reservoirs. Water meters at reservoirs can provide more accurate peak demand data. A less expensive alternative to installing water meters at reservoirs is to calculate inflow and outflow by changes in reservoir level; however, this approach may not be as accurate.

The City may also wish to consider an audit of all commercial services to identify type of installation, wastage of water and types of fixtures. An audit can also identify storm to sanitary cross connections concerns and water cross connection concerns. Data can be recorded on the City's GIS system.

2.7 WATER MODEL

In order to simulate citywide water system characteristics a water model is required.

The original water model that was developed as part of the 1990 study used the Water Works ad-in program for Lotus 123 (MS DOS) spread sheet software. In 1995 the City switched from a MS DOS to Windows operating systems. The water model was imported and updated in Water Works for MS Excel spread sheet software. Between major model updates the software was updated and maintained by City staff.

Hydraulic water model update and calibration was part of the scope of work for this study. Stantec and the City of Whitehorse agreed to use the Window based EPANET software package for water modeling. EPANET was developed by the Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory. EPANET is public domain software that may be freely copied and distributed.[6]

EPANET is also open source software, which means the software can be developed or modified by software users.

Some of the features of EPANET are as follows:

- Extended period simulation that was not capable in the Water Works model
- Unlimited pipes, nodes (pipe junctions), pumps, valves, storage tanks and reservoirs can be modeling
- Colour-coded network maps, data tables, time series graphs, and contour plots with AutoCAD (dxf) or Windows meta file (wmf) exporting capability

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[6] *EPANET web page*

- Water quality can be tracked by water source blending, water age, chemical residual, tracer material through the network or reactive material (disinfection by-products)
- Computation of head loss by Hazen-Williams, Darcy-Weisbach or Chezy-Manning formulas and head loss accountability for bends and fittings
- Constant or variable speed pump modeling with energy and cost computations
- System control based on tank level, timer, flow control valve or complex rule-based controls
- Variable shape storage tanks
- Multiple demand categories at nodes with time variation (ADD, MDD, PHD, NFD)
- Pressure-dependent flow issuing from emitters (sprinkler heads or bleeders)

The City of Whitehorse personnel converted the model from Waterworks input files to the EPANET input files. Further refinements and additions were undertaken by Stantec staff and then implemented in the model.

The model was updated based on information provided by the City and calibrated based on fire flow testing. Section 3.4 provides information on fire flow testing. Section 3.6 provides information on model development and Section 4.1.1 provides information on model calibration.

2.8 FIRE FLOW REQUIREMENTS

The fire flow requirements were calculated as per the Fire Underwriters Survey guidelines provided in the document entitled Water Supply for Public Fire Protection, 1999. This document is intended as a guide, not a standard, to summarize the recommendations of the Fire Underwriters Survey with respect to fire protection in municipal water works systems design.

Fire flows are considered at critical points in the distribution system in conjunction with the maximum day demand. The requirements will remain the same as documented in the 1990 study as follows:

Single Family Residential	76 to 100 L/s for duration of 2.0 hours
Multi-Family Residential	90 to 150 L/s for duration of 2.5 hours
Commercial/Industrial	150 to 250 L/s for duration 3.5 hours.
School/Public Lands	243 L/s for duration of 3.2 hours.

2.9 OPERATIONAL PRESSURES

2.9.1 Operating Pressures

A pressure of 240 kPa (40 psi) has been considered the minimum acceptable pressure at any point on the system for all domestic demand conditions such average day demand, maximum day demand, peak hour demand, night fillings etc.). This criteria is consistent with criteria used in previous studies for the City of Whitehorse. Maximum acceptable distribution pressures were established as 700 kPa (100 psi). The most desirable pressure is around 400 kPa (70 psi).

2.9.2 Fire Flow Pressure

The system was analysed under the assumption that required fire flow must be available with a minimum system residual pressure of 140 kPa (20 psi), which is in compliance with recommendations of the Insurer's Advisory Organization.

2.10 WATER STORAGE REQUIREMENTS

2.10.1 Criteria

Storage is used to provide the following within a water works system:

1. Fire Flows for critical fire demand
2. Peak Demands when demand exceeds the input capacity of the supply facilities (usually tested at MDD)
3. Supply interruption / emergency storage should provide water during shutdown of the supply trunks for maintenance, power outage or failure
4. Additional storage

2.10.2 Fire Storage

As mentioned in Section 2.8, the fire flow requirements also require a fire duration, which in turn determines the storage requirement. The most critical land use building type in a given service area determines the fire storage requirements for the reservoir.

2.10.3 Peak Demand Storage

Peak demand storage can be determined by subtracting the pumping supply capacity from a peak demand flow over a period of time (commonly 4 to 6 hours). If accurate 4 to 6 hour peak demand flow data is not available, typically 25% of MDD is used to calculate peak storage.

Future monitoring of the reservoir levels in conjunction with pumping rates can assist in verifying this design assumption.

2.10.4 Supply Interruption / Emergency Storage

Storage for supply interruption can range from 15% to 100% of an average day demand depending on the reliability of the supply system and the time required to make repairs at a pumphouse, repairs to a supply main break or switch to an alternate source. Whitehorse has the ability to cascade water from higher to lower reservoirs and can provide water supply from either Schwatka Lake or ground water wells; however, Whitehorse is a remote location for repair materials and supply interruption storage of 100% average day demand should be considered.

Emergency Storage of 15% may be considered for losses due to a distribution system break. Generally a distribution system break can be isolated rather quickly and does not significantly impact reservoir supply.

2.10.5 Additional Storage

During long periods of high demands (ie the maximum five to seven day demand) the system may have problems replenishing the reservoir volumes at night. Additional storage can assist in buffering this type of occurrence, however, water turn over within the reservoir to maintenance of chlorine residual has to be considered for water quality issues. If pumping capacity is capable of meeting MDD, 5 day or 7 day storage is generally not required.

Operational storage may also be considered to account for changes in reservoir level due pump call and pump stop elevation settings.

2.10.6 Total Storage Requirement

The following evaluation criteria were used to determine storage requirements:

- Fire Flow Storage varies depending on land use. Typically utilizing 250 L/s x 3 hrs. = 3150 m^3
- Peaking Storage or Balancing Storage = 0.25 x Maximum Day Demand
- Emergency or Supply Interruption = 1.0 x Average Day Demand and/or Emergency Storage = 0.15 x Average Day Demand

2.11 PUMPING CAPACITY AND SYSTEM RELIABILITY

Under normal operating conditions, all pump stations should be capable of generating flow rates equivalent to the maximum day demand (MDD) for the service area. Demands exceeding the maximum day demand such as fire flows and peak hour demands can be made up from storage. All pumping stations must have an additional backup pump above the number of duty pumps required to meet MDD capacity. The backup pump is required for duty pump failure or maintenance activities. The backup pump must be at least equivalent in capacity to the largest normal duty pump.

Electrical power backup is also recommended to maintain system reliability. Backup power must have the capability of running control systems and enough pumps to meet MDD for a period of two days.

2.12 TRANSMISSION AND DISTRIBUTION MAINS CAPACITY

All the transmission and distribution mains should be able to convey the required amount of flow with a reasonable head loss and mean velocity as per the City's design standard. Flows in excess of 1.5 m/s during a peak hour simulation and 3.0 m/s during a fire flow simulation are general indicators of a potential system bottleneck.

2.13 THERMAL EVALUATION CRITERIA

2.13.1 General

The Whitehorse water system should be in continuous motion and temperatures should not drop below 1.5 ºC to protect against freezing. The Whitehorse water system employs four primary strategies to prevent freezing as follows:

1. Blending of warmer ground water from wells with Schwatka Lake water to increase Selkirk Pumphouse output water temperature

2. Localized boilers to increase water temperature

3. Bleeders to maintain watermain flow and dissipate colder water

4. Circulation systems to maintain watermain flow and for uniform mixing with warmer supply water

The existing four ground water wells supply a significant portion of the water system's demand throughout the winter months. Mixing of the groundwater and surface water raises overall water system temperature to help prevent freezing.

In addition to water blending, service Areas 1 and 2 primarily rely upon circulation systems and bleeders to prevent freezing. The remainder of the service areas require water tempering boilers during times of very low water temperature. As population and water demands increase additional thermal sources will be required in the form of increased ground water supply or additional boilers.

The following tasks have been carried out in this study to assess existing and future thermal conditions:

- Explore the possibility of utilizing the Water Quality Module of the EPANET software instead of the existing thermal model (developed by Stantec in a previous study) to simulate temperature distribution of the water system
- Assess available temperature data of various sources such as ground temperature data, boilers temperature, etc.
- Conduct a field program to collect temperature data at various locations of the existing water distribution system
- Calibrate the temperature model
- Carry out the thermal evaluation of the existing system
- Identify deficiencies (if any) and propose improvements
- Expand the thermal model to include the future growth areas
- Carry out the thermal evaluation of the future water system
- Identify deficiencies for future growth scenarios

- Identify and evaluate system improvement alternatives
- Develop cost estimate and implementation plan for the selected alternatives

The winter conditions of 1971/1972 were established as the design winter conditions to provide a worst-case scenario for the model.

2.13.2 Selection of a Thermal Model

A thermal model was developed during the Crosstown Watermain Predesign completed in July 1991 and was updated by Stantec in 1995. The thermal model was stand alone software. The input data for the thermal model was obtained from the Waterworks water distribution system model.

As was described in Section 2.7, EPANET software was chosen for water distribution system modeling for the current study. The decision also was made to adapt EPANET for thermal modeling. Utilizing EPANET for both hydraulic and thermal modeling allows for simultaneous examination of hydraulic and thermal performance and reduces model maintenance to only one application.

Stantec determined that the EPANET Chlorine Decay Model could be modified to perform thermal analysis and a heat transfer formula was developed. The heat transfer formula accounted for the temperature gradient across a water pipe (difference between the ground temperature and water temperature), soil type, depth of cover and pipe material.

In order to test the thermal model performance, the heat loss through a hypothetical pipe network for a given flow, climatic and physical conditions was computed using a Microsoft Excel spreadsheet. The spreadsheet results were compared to the EPANET model and both methods produced identical results.

The hydraulic model provides flow, velocity and pipe physical data to the thermal model for given demand conditions and system operational characteristics. Temperatures are simulated at each pipe junction point based on source temperatures, heat transfer characteristics and time.

3.0 Water Distribution System Components

3.1 GENERAL

The City of Whitehorse's water distribution system is a complex multi zone system consisting of a surface water source, ground water source, pumping facilities, boosters stations, water storage reservoirs, chlorination systems, circulation systems, watermain looping, boiler systems, pressure reducing stations, pressure sustaining stations and fire flow valves.

3.2 REVIEW OF 1990 WATER AND SEWER STUDY

The 1990 Water and Sewer Study became the basis for capital planning for the City over the past 12 years and a significant amount of the recommendations of that report have been implemented. Since the current study is based on the 1990 study an assessment of the 1990 study findings needs to be made.

The following are excerpts from the 1990 study. Brief commentary based on findings of this study is provided and more detailed discussion can be found in appropriate sections of this study.

1990 Water System Conclusions

1990 Existing Water Supply System

1990 Water Supply

.1 *The existing warm water wells are sufficient to meet the current needs of the water distribution system. Increased thermal requirements and demands make the expansion of warm water well network, or the installation of water heating system necessary. A detailed thermal analysis, using the design flows and proposed upgradings of this report is required.*

During times of high surface water turbidity and high demand existing well water supply is marginal. This study also recommends expansion of the water well network to meet water quality and future thermal requirements. (Section 3.3.3)

.2 *The water supply line from Schwatka Lake is adequate for the existing flows.*

This finding still remains true, however, increasing well capacity can also address any long range supply concern, provide a more secure water source, provide less treatment requirements and increase water temperatures for thermal protection. (Section 3.3.3)

.3 *The City of Whitehorse has high per capita water consumption rates. These high rates are in part due to high leakage and bleeder rates. A bleeder reduction program would benefit the City in reduced capital and operating expenditures. A leakage reduction program for the skeletal and local water distribution network is warranted in order to reduce wastage of water.*

The City has developed better leak detection ability through purchase of new equipment and training. The City has discontinued using a large number of bleeders since the 1990 Study. The need for many bleeders was eliminated after Crosstown Watermain construction and installation of Thermostatically Controlled Bleeders in Crestview and Downtown. A large number of bleeders were simply no longer used after re-evaluating thermal conditions. (Section 2.6.3)

.4 *A water usage monitoring program is warranted in order to establish firm water demand rates and aid in consumption reduction programs.*

Again a very important task that has seen some success but more work is still required. A formal and complete water and sewer consumption audit should be performed including a structured meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement, calibration and data collection in each service area. (Section 2.6.10)

1990 Pumping Facilities

.1 *The Selkirk Pumphouse is at its pumping capacity, increased capabilities are required to meet future demands. Upgrading can take the form of replacing the existing pumphouse for a 20-year design horizon, or replacing the existing pumps and supplying adequate power and standby power for a 5 to 10 year fix up, followed by the replacement of the existing structures.*

Selkirk Pumphouse has not been upgraded. The 1995 Crosstown Water Main Phasing Report (Stantec) redefined upgrading the requirement to 2012. This study also recommends upgrading booster capacity along with constructing a water treatment plant. (Section 4.2.1.1)

.2 *The Two Mile Hill Booster Station is at its flow capacity. Upgrading can take the form of replacing the existing pumphouse for a 20-year design horizon, or replacing the existing pumps and supplying adequate power for a 5 to 10 year fix up, followed by the replacement of the existing structure.*

Two Mill Hill Booster Station was reconstructed in 1997 to meet ultimate population horizons identified in the 1990 Study. (Section 4.2.1.2)

.3 *McIntyre Creek Booster Station is sufficient for the current demand. Upgrading of the station in the form of pump replacement will be required as water distribution network upgradings in the area occur.*

Boosting capacity has not been upgraded at McIntyre Creek. This study has found that McIntyre Creek Booster will need future upgrading for capacity but should also be upgraded in the near future to improve fire flow requirements on Grove Street and Ponderosa. Further, Porter Creek Reservoir has only one cell and does not meet capacity requirements; therefore, reliable booster capability is even more important. (Section 4.2.1.3)

.4 *Hamilton Boulevard Pumphouse is sufficient for the current and ultimate demands. Installation of two 50 Hp pumps and replacement of the existing 40 Hp pumps will be required as upstream developments occur.*

Hamilton Boulevard Pumphouse has not been upgraded. This study also recommends future booster upgrades. Although this station can meet current maximum day demand requirements with only one booster, pumping dynamics when the upstream Copper Ridge booster and reservoir are deep cycling can be a concern. (Section 4.2.1.6)

.5 *Crestview, Ponderosa and Grove Booster Stations are all adequate for the current and ultimate demands. Back up power or standby supply is not recommended for Ponderosa or Grove Booster Station.*

This study has come to the same conclusion. (Sections 4.2.1.4 and 4.2.1.5)

.6 *A water metering program should be instituted at the water distribution pumphouses and reservoirs. Total and instantaneous data should be collected daily at all facilities.*

More work is still required. (Section 2.6.10)

1990 Distribution Network

.1 *The available fire flow on Hyland Crescent in the Riverdale area is substantially less than recommended guidelines, installation of the lines as shown on Plates 3 and 4 are required to increase the fire flows.*

This has not been completed and Riverdale fire flows are still a concern. (Section 4.2.2)

.2 *The available fire flow in the Downtown area is not in accordance with recommended fire flow guidelines, drawing such a fire flow can create negative suctions pressures at the Two Mile Hill Booster Station.*

Installation of the crosstown transmission pipe as shown on Plate 3 is required to increase suction head.

Improvements have been made during Downtown reconstruction projects and the Crosstown Watermain has been constructed to address Two Mile Hill Booster Station supply capacity. Fire flows are still a concern Downtown and can be addressed during future phases of the Downtown reconstruction projects. This study also makes recommendation for improvement under Downtown Westside Thermal and Fire Flow Improvements. (Section 4.2.2)

.3 *The available fire flow in tested areas of Marwell does not meet recommended standards for light industrial and warehouse development, bypassing of the pressure reducing valves in the Two Mile Hill Booster Station will provide sufficient flows.*

Two Mile Hill Booster Station was constructed with low demand and high demand pressure reducing valves to supply Marwell. Fire flows for high demand buildings are still a concern. (Section 4.2.2)

.4 *Fire flows to the Whitehorse Hospital are sufficient with the valve north of the hospital open. A method of ensuring the operation of the valve is required to permit fire flows.*

This has been addressed and fire flows at the hospital are now adequate.

.5 *High head losses between Selkirk Pump Station and Two Mile Hill Booster Station result in low suction pressures at the Two Mile Hill Booster Station. A crosstown transmission line is required to deliver water to the booster station.*

The Crosstown watermain has been installed.

.6 *High head losses above the Two Mile Hill Booster Station in the Takhini area result in a high total dynamic head required at the Two Mile Booster Station. Replacement of the lines above the booster station as shown on Plate 3 and 4 are required.*

Watermain improvements have been made.

.7 *The available fire flow in the upper portion of the existing Hillcrest are will not meet recommended standards for even single family dwellings, and part of this area is zoned for multiple family dwellings. Installation of the lines as shown on Plate 3 and 4 are required to increase the fire flows.*

This work has not been completed and the current study makes the same recommendation. The construction of the Airport watermain would have improved fire flows to Hillcrest but the model still indicates a marginal deficiency. (Section 4.3.2)

.8 *The available fire flow at the Airport is not adequate to meet recommended standards for fire flows for the type of buildings prevalent in the Airport area. Installation of the lines as shown on Plate 3 is required to increase the fire flows.*

Airport fire flows have been addressed with the construction of the 500 mm airport transmission main.

.9 *Fire flows in Granger, McIntyre, Kopper King, Yukon College and Crestview meet recommended guidelines.*

This study has identified parts of Granger as a concern. Yukon College and McIntyre Neighbourhood have adequate fire flow. Fire flows are adequate on City mains fronting the Kopper King neighbourhood. The private trailer parks in Kopper King do not have fire hydrants. (Section 4.2.2)

.10 *Fire flows in Porter Creek are adequate.*

This study found that there are fire flow concerns on Ponderosa Drive, Grove Street and in Porter Creek West if no boosters are operating at McIntyre Creek Pumphouse. (Section 4.2.2)

1990 Water Storage Reservoirs

.1 *Riverdale and Hillcrest Reservoir have excess storage capacity by 6400 and 4900 cubic meters respectively.*

Hillcrest and Copper Ridge Reservoirs have adequate storage capacity. This study found that Riverdale Reservoir is currently and ultimately under capacity by 1,816 cubic meters and 5,126 cubic meters respectfully. (Section 4.2.3)

.2 *Valleyview and Porter Creek Reservoirs have storage deficiencies by 900 and 1750 cubic meters respectively. The timing to expand the Porter Creek Reservoir is limited to the ability to supply sufficient night filling flow through the existing water distribution network. The reservoirs can only be expanded after completion of the Two Mile Hill Booster Station, Selkirk Pump Station and Crosstown Pipeline.*

These reservoirs have not been expanded. This study also recommends expansion of Porter Creek and Valleyview Reservoirs. The City has upgraded the watermain along Holly Street and portions of Wann Road and the Alaska Highway crossing. Replacement of the reservoir supply line through Kulan Industrial Area is still planned. (Section 4.2.3)

The need to expand the Porter Creek Reservoir can be delayed by the ability of McIntyre Creek Booster Station to provide maximum day demand, thus presenting a secure system in terms of adequate back up supply.

McIntyre Creek Booster can still meet MDD capacity but does not have back up power. The station does have a diesel fire pump but operations staff has raised a concern about reliability. (Section 4.2.1.3)

.3 *Valleyview and Hillcrest Reservoirs combined have an excess storage capacity of 500 cubic meters, thereby delaying the requirement to twin Valleyview by 5-10 years.*

The current combined surplus of Hillcrest Reservoir and Copper Ridge Reservoir is 398 cubic meters which is less than the deficiency at Valleyview Reservoir. It should also be noted that manual cascade from the upper reservoirs would be required in emergency situations. This study also recommends upgrading Valleyview Reservoir within 5 to 10 years. (Section 4.2.3)

1990 - 5 Year Capital Plan

Table 1-1 Water Distribution System 5-year Capital Plan Plus Alternate Plan

(Includes Study and Analysis Cost)

Upgrading Item	*Year of Upgrade*	*Upgrade Cost ($)*	*Alternate Upgrade Cost ($)*	Comments
Selkirk/Two Mile Capacity Study	*1*	*70,00*	*70,000*	Completed [7]
Two Mile Hill Booster	*1*	*4,400,00*	*1,510,000*	Completed
Crosstown Pipeline	*1*	*4,200,00*	*4,200,000*	Completed
Selkirk Pump Station	*2*	*3,300,00*	*710,000*	Deferred [8]
Crosstown Tie-in	*3*	*420,00*	*420,000*	Completed
Takhini Upgrade – 1	*4*	*1,000,00*	*1,000,000*	Completed
Riverdale Upgrade – 1	*4*	*850,00*	*850,000*	Not Completed [9]
Hillcrest Upgrade – 1	*4*	*780,00*	*780,000*	Not Completed [5]
Hospital Upgrade	*5*	*78,00*	*78,000*	Completed
Riverdale Upgrade – 3	*5*	*52,00*	*52,000*	Completed
Thermal Analysis Study	*5*	*130,00*	*130,000*	Completed
Total 5 Year Plan Cost		*15,280,00*	*9,800,000*	
Total Upgrading Year 1		*8,670,00*	*5,780,000*	
Total Upgrading Year 2		*3,300,00*	*710,000*	
Total Upgrading Year 3		*420,00*	*420,000*	
Total Upgrading Year 4		*2,630,00*	*2,630,000*	
Total Upgrading Year 5		*260,00*	*260,000*	

Note: All costs are in 1990 dollars

[7] 1995 Crosstow n Watermain Phasing Report
[8] Deferred after 1995 Crosstown Watermain Phasing Report evaluation
[9] Similar project still recommended for thermal and fire flow improvements

1990 Ultimate Water Supply System

1990 Water Supply

.1 *The water supply line from Schwatka Lake to the Selkirk Pumphouse is required to be twinned. This would allow better system hydraulics, lowered horsepower pumps and greater supply security.*

The Schwatka Lake intake has been reconstruct as a siphon. Part of the intake line near Schwatka Lake has been twinned. The remainder of the intake line to Selkirk Pumphouse may have to be twinned if additional ground water supplies are not developed. (Section 4.2.1.1)

1990 Pumping Facilities

.1 *An eighth pumphouse located above Hillcrest Reservoir will be required to service the south half of Expansion Areas C and D and McLean Lake. The pump station should have an installed horsepower of 150 Hp complete with 2 service pumps plus a standby pump. Backup power generation should be by diesel generator.*

Copper Ridge Pumphouse and Reservoir was constructed in 1996.

Distribution Network

.1 *The water distribution network in Marwell can be expanded as shown on Plate 4. Fire flows are sufficient with the installation of a pressure reducing valve bypass in the Two Mile Hill Booster Station. The setting of the pressure reducing valves can be increased to 580 kPa if higher pressures are required.*

Two Mile Booster was reconstructed in 1997 with new pressure reducing valves to supply Marwell. Platinum Road water main looping will be completed in 2003.

.2 *Completion of the replacement of the lines in the Takhini area in order to adequately ensure proper flows to Northlands and Lower Porter Creek is required. The actual location of the lines will be dependent on the development of the area. The lines shown on plate 4 follow the existing routes, the final routing can vary so long as sufficient transmission capability is maintained.*

A significant amount of infrastructure has been upgraded since 1990. The City is currently working on an area development scheme for the old western portion of Takhini.

For the purpose of this report, timing of the replacement of the water supply line to Northlands Trailer Park is selected to be after the replacement of Selkirk Pump Station, Two Mile Hill Booster Station, *Crosstown transmission line and the Takhini-1 upgrading. The reason for this selection is that from a hydraulic point of view, these major facilities are more important than the Northlands Trailer Park supply line. The actual timing of the installation can be moved forward if desired.*

The supply line to Northlands Trailer park has been upgraded and sized to accommodate Lower Porter Creek Bench Expansion.

.3 *Kopper King and Yukon College can be serviced from Area 2 (suction side of McIntyre Creek Booster Station) thereby saving pumping costs and extra pumphouse upgrading.*

Yukon College used to be supplied from the discharge side of McIntyre Creek Pumphouse. Yukon College is now serviced from Area 2 (Valleyview Reservoir) by construction of a connection on the inlet side of McIntyre Creek Pumphouse near the station. Flow was reversed through Range Road PRV and the College supply line became an additional supply line for McIntyre Creek Pumphouse. Kopper King supply line was left on the discharge side of McIntyre Creek pumphouse to meet fire flow requirements.

.4 *Twinning of the water supply line from Area 2 to Kopper King may be required depending on the level of development. Should the area be developed such that greater than 190 L/s of fire flow are required, a second 300 mm diameter supply line would be required. Fire flow with the twin line is estimated to be 360 L/s.*

This work has not been completed, as the area has not seen any significant development.

.5 *Lower Porter Creek can be served from Area 2 by the installation of a 350 mm supply line from Northlands Trailer Park to Lower Porter Creek, with an additional supply line from Porter Creek complete with a PRV station for emergency supply.*

Lower Porter Creek Bench has not yet been developed. This study makes the same recommendation for future development. (Section 4.4.1)

.6 *Riverdale Expansion can be serviced from Area 1 by the installation of a 300 mm supply line from below the Riverdale Reservoir.*

Riverdale Expansion has not yet been developed. This study makes the same recommendation for future development. (Section 4.4.5)

Stantec

.7 *McLean Lake Neighbourhood can be serviced from the proposed Hillcrest "B" Reservoir by a 350 mm diameter supply line complete with a series of pressure reducing stations.*

McLean Lake Neighbourhood is now referred to as Beyond Copper Ridge. Hillcrest "B" Reservoir has been constructed and is now called Copper Ridge Reservoir. Beyond Copper Ridge has not yet been developed. This study makes the same recommendation for future development. (Section 4.4.3)

.8 *Expansion Area E can be serviced as shown on Plate 4.*

This area is referred to as the Tank Farm Expansion in this study. Possible service looping from Hamilton Boulevard still applies. (Section 4.4.4)

.9 *Expansion Areas C and D can be serviced as an extension to the existing water distribution network as shown on plate 4.*

Expansion Areas C and D are now developed as Copper Ridge Neighbourhood.

.10 *The replacement of watermain above McIntyre Creek Booster Station is required in order to ensure MDD and NFD of the Porter Creek Reservoir.*

Supply line modifications at McIntyre Creek Pumphouse by the City improved MDD flows since the 1990 Study. The McIntyre Creek Pumphouse currently has a 200 mm and a 250 mm discharge line. Stantec determined that even under ultimate build out capacity conditions with upgraded pumps at McIntyre Creek Pumphouse running at peak hour demand the watermain flow characteristics were adequate. Refer to Section 5.3.2 for further details. (Section 5.3.2)

1990 Water Storage Reservoirs

.1 *A fifth water storage reservoir, referred to as the Hillcrest "B" Reservoir is required to service the area above the existing Hillcrest Reservoir including McLean Lake. The volume of the proposed reservoir is 6500 cubic meters.*

Completed in 1996. This is the Copper Ridge Reservoir

.2 *Twinning of the Valleyview and Porter Creek Reservoirs with 5000 m^3 reservoirs is required to meet storage requirements. By combining the volumes of the reservoirs for servicing the Lower Porter Creek area, greater storage area is available.*

This work has not been done. This study also recommends twinning of Valleyview and Porter Creek Reservoirs. (Section 5.3.5)

.3 *The method of controlling the water storage reservoir elevations should be reviewed in order to adequately maintain storage volumes.*

The implementation of SCADA has greatly improved the ability to adjust reservoir set points at Riverdale, Valleyview and Copper Ridge Reservoirs. Porter Creek and Hillcrest Reservoir still need to be incorporated into the SCADA system. Reservoir dynamics is very complicated as system demand, reservoir turnover rates and chlorine residuals need to be taken into account. Any automated control logic would be complicated and prone to error. Operator intervention is recommended to make adjustments to reservoir set points as system dynamics dictates. SCADA can provide very good data trending to aid in adjustment of reservoir level set points. (Section 4.2.3)

1990 Recommendations – Water Supply System

The following recommendations pertain to upgrading the operation and management of the water supply system for the City of Whitehorse.

.1 *The City should implement the five-year upgrading plan as presented in Table 5.1. The plan involves upgradings required immediately and those required within 5 years in order to present a solid foundation for expansion of the water distribution system. The decision to upgrade or replace the Selkirk Pumphouse and the Two Mile Hill Booster Station as outlined in the upgrading program is left open for the City to select. The decision to construct the new pumping facilities now or later will be based on the availability of construction capital to the City. Either option will produce sufficient results within the 5-year time frame, however, replacement of the pump stations will ultimately by required.*

Selkirk Pumphouse has not been upgraded.

.2 *A water distribution data collection program should be initiated by the City in order to obtain accurate data for future studies and municipal planning. The basis of the data collection program should centre around the pumping facilities and reservoirs and contain the following information:*

1. *instantaneous flow records*
2. *total flows*
3. *pump hour meters*
4. *pump down time*
5. *pump repairs and maintenance records*
6. *reservoir elevations*
7. *reservoir flows*
8. *fire flow times and locations*

Some efforts have been made but more w ork is still required.

.3 *The City should initiate a water consumption reduction program. Areas to concentrate under this program would be bleeder reductions,* *leakage location and repair, residential and commercial consumption reduction.*

Thermostatically controlled bleeders have been installed in private homes in Riverdale, Downtown, and Hillcrest. A large number of free flow bleeders have been eliminated. The City has improved leak detection and repairs all leaks, as they are located. There has been past water conservation education campaigns. There are a few more bleeders that can be eliminated during area reconstruction. The City can implement residential water metering and continue with educational efforts. The City should also consider developing a demand management strategy.

.4 *The City should allow the development of the Expansion Areas C, D, and E, Lower Porter Creek, Riverdale Expansion and McLean Lake. The servicing of these areas would be timed such that they would be coincidental with required system upgrading and the provision of sanitary sewerage system discussed in Sections 7 through 11 of this report.*

This study has addressed servicing requirements for expansion areas identified in the current Official Community Plan. Area development will be dependent upon demand and financial resources.

.5 *Implementation of the required upgradings beyond the 5-year plan will be dictated by the direction of growth of the City. Section 4 details the planned 20-year upgrading program based on infilling existing areas, then allowing for expansion. The timing of upgrading events must be reviewed with respect to the direction of growth.*

This recommendation still applies.

.6 *A flushing and monitoring program for the water distribution system should be implemented to remove foreign particles from the network and identify problems, such as line breaks in the system.*

A unidirectional watermain flushing program was developed by Epcor and implemented in 2002.

.7 *A pumphouse and reservoir inspection program should be instituted. Records of all flows, repairs, maintenance and improvements should be kept. Pumphouse data recording should be daily, while inspection of the facilities is recommended to be bi-weekly.*

The 1999 Pumphouse and Lift Station Audit was performed to address facility condition and make recommendation for improvement. The implementation of SCADA has greatly increased data recording. The City does not have a good maintenance management system to record repairs and inspection results.

3.3 WATER SHED AND WATER SUPPLY SOURCES

Selkirk Pumphouse supplies all water for the City of Whitehorse. Selkirk Pumphouse is located in the Riverdale Neighbourhood on Selkirk Street and adjacent to the Yukon River. Selkirk Pumphouse draws water from Schwatka Lake and nearby groundwater wells. Chlorine is injected for disinfection and booster pumps at Selkirk Pumphouse supply Riverdale Reservoir and the water distribution system. The Selkirk water facility consists of the following:

- Intake and siphon at Yukon Energy Corporation's dam on Schwatka Lake. The siphon was recently constructed above the Schwatka Lake high water line in order to reduce the risk of dam failure due to potential water migration along the pipe zone. The intakes were not reconstructed.
- Currently, raw water is taken from Schwatka Lake by gravity to the Selkirk Pump station via a 550 mm diameter steel intake of which a small portion has been recently twinned as part of the intake siphon construction.
- A group of four active groundwater wells draw water from the Selkirk aquifer and discharge to the suction side of Selkirk Pump Station. The wells are used for water tempering in the winter and early spring and turbidity reduction in late spring and early summer.
- Discharge from Wells #1, #4 and #5 are combined into a 300 mm main header which then enters the building on the pump supply side. Flow from Well #6 is measured at the well house and ties to the 550 mm diameter Schwatka Lake intake line upstream of the pumphouse.
- Total flow from the station is measured by differential pressure across an orifice plate. The metering system primitive and has not been calibrated in years.
- The Selkirk Pump Station has three 100 hp Gould horizontal split case pumps to supply water to the entire city.

3.3.1 Existing Water Treatment

Historically Whitehorse was always considered to have good source water quality and therefore did not require a high degree of water treatment. The only form of treatment provided is initial chlorination for disaffection at Selkirk Pumphouse and rechlorination as required at Two Mile Booster Station and Copper Ridge Pumphouse. A rechlorination facility was constructed at Crestview Pumphouse but was never maintained in an operational condition.

Since the City does not have a treatment plant, turbidity levels have historically been maintained by blending well water with Schwatka Lake source water. The purpose was to meet the Canadian Drinking Water Quality Guidelines for an Aesthetic Objective of 5 NTU (the City formally used an assumed equivalent of 5 FTU and has since upgraded instrumentation to utilize the more common NTU testing method). The Canadian Drinking Water Guidelines for water treatment is 1 NTU. Turbidity reduction is a very important facture in pathogen removal or inactivation.

Recent incidents and resulting developments in water treatment guidelines and better general understanding have forced a change for all Canadian water purveyors. Water sources are no longer assumed to be pristine and always safe.

The City has improved water testing, improved system configuration and is pursuing improved treatment options by implementing the following.

1) Turbidity monitoring had traditionally been done manually and usually only once per day during times of high turbidity. The City has automated turbidity monitoring to gain more turbidity data prior to the design of a water treatment plant and provide interlocks to shut down boosters during times of excess turbidity.

2) The City is undertaking reservoir cleaning and watermain unidirectional flushing programs to reduce turbidity in the distribution system.

3) The City has upgraded the chlorination system at Selkirk to a compound loop automated controller and injection system. The new chlorinator automatically adjusts chlorine feed based on flow and end residual. The old manual system only injected at a rate dependant on how many booster pumps were running. The city has also ensured that the chlorine system is fully alarmed and has interlocks to shut down boosters upon chlorine system failure.

4) The City has made efforts to improve CT. CT is a disinfection rating based on water pH, temperature, chlorine residual and contact time. The city has increased chlorine residuals throughout the City and has improved contact time to the first customer by installing a normally closed valve near Selkirk Pumphouse on the Selkirk Street transmission main. The installation diverted flows a greater distance to the Selkirk Street education facilities who used to be the first customer. Hydraulic changes were modeled by the City and were considered acceptable. Stantec compared a few fire flows in Riverdale and found that the line made a negligible difference.

5) The City has in recent years expanded field water sampling. The City now conducts more system water sampling for coliform bacteria and has increased chlorine residual monitoring. Increased chlorine residual monitoring has resulted in year round operation of circulation systems and conducting periodic system flushing to remedy low chlorine residuals. The City has also increased testing for Giardia and water physical parameters, such as, chemical properties, hardness, pH, alkalinity and colour.

3.3.2 Water Quality

A 2001 Water Treatment Plant – Feasibility Report (Epcor) was conducted to address the feasibility of constructing a water treatment plant. The following recommendations were made as a result of the work completed under the Epcor study:

- Additional water quality data should be collected to allow for planning of future water treatment. As a minimum the City should undertake physical and chemical analyses with particular attention given to seasonal fluctuation in colour and turbidity. City operations staff have recently undertaken more frequent monitoring.

- The new treatment plant should use a multiple barrier design (i.e. pathogen inactivation or removal) with UV disinfection added post filtration.
- A preliminary design study should be initiated as the next phase in the development of a water treatment plant for the City of Whitehorse.

The City will have to consider treatment of the Schwatka Lake source to provide protection to meet increasingly more stringent water quality regulations, and provide protection against Giardia and Cryptosporidium. The long-term security of supply should be paramount to all other initiatives undertaken for the utility. This includes an adequate, reliable and a safe water source.

Control of turbidity is principal in water treatment and is the most important factor for control of pathogens. Higher turbidity levels effect disinfection since higher dosages of chlorine or ultraviolet would be required. Proper removal of turbidity also has the effect of aided removal of larger pathogens, such as, Giardia and Cryptosporidium.

Drinking water quality has become and important issue in recent years. The City should continue to develop and implement a water quality management program. Water quality management is currently being addressed though a water shed management plan, in house sewer and water task force, a back flow prevention program, staff training, water license revision and development of operating procedures. Development of an all-encompassing plan may be beneficial to provide plan management direction and corporate priority.

At the time of writing of this report a watershed management plan was being drafted. A wellhead protection plan is required if not already part of the watershed management plan scope of work.

The City is in the initial phases of developing backflow prevention program. A backflow prevention program should eventually encompass City infrastructure and operations along with commercial and private services.

The City of Whitehorse water license has been undergoing a revision for the past few years and the new license will be issued in the near future.

The City currently only has a few official critical operating procedures documented. This work needs to be completed and there may not be enough internal resources to complete the task. The City may want to consider including this work in its capital plan to either hire more staff or hire a consultant to complete the work.

3.3.2.1 Raw Water Characteristics

The water sampling characteristics of raw water supplied to the City for the years between 1986 and 2000 are summarized in Table 3.1.

Table 3.1 Raw Water Quality Profile

Parameters	**Sampling Station WH1**	**Sampling Station WH2**	**CDWG (Canadian Drinking Water Guidelines)**
	Raw Water from Schwatka Lake	**Raw Water from Selkirk Aquifer**	
Turbidity	0.13 to 6.2 NTU	0.1 to 2 NTU	1.0 [10]
Colour	<5.0 to 25.0 TCU	<0.5 to 5.0 TCU	15
Total Alkalinity	33 to 179 mg/L	37 to 167 mg/L	-
Iron	0.019 to 1.15 mg/L	<0.003 to 0.281 mg/L	0.30
Manganese	<0.001 to 0.024 mg/L	0.0006 to 0.049 mg/L	0.05
Hardness	41 to 160 mg/L	43 to 164 mg/L	None
pH	6.9 to 8.6	7.62 to 8.3	6.5 to 8.5
Temperature	1.2 to 13 ^{o}C	2.1 to 9 ^{o}C	-

Results listed in Table 3.1 are from water sampling that has been submitted for independent testing. In discussions with City staff, it has been indicated that turbidity has been as high as 19 NTU from the Schwatka Lake source during spring melt. The City has only recently installed continuous turbidity monitoring on Schwatka Lake/Well 6 inlet to Selkirk Pumphouse and Selkirk Pumphouse outlet to enable gathering of accurate historical information. The station outlet turbidity meter is interlocked with the station control to send an alarm if turbidity exceeds 4 NTU and shut down the booster if turbidity exceeds 5 NTU. Traditionally, since turbidity monitoring was only conducted once per day during times of high turbidity, the City likely experienced unknown turbidity spikes in the distribution system.

Raw water at the Selkirk inlet is tested for viable Giardia cysts. The City has had one positive test for viable cysts in November 2002. Treated water has never been tested for Giardia cysts.

3.3.2.2 Regulations and Objectives for a Proposed Water Plant

The CDWG sets either a Maximum Acceptable Concentration (MAC) or an Aesthetic Objective (AO) or both for over 100 substances and parameters listed. MAC is considered as a "Health" standard, while AO is an “aesthetic objective” (consumer acceptance) standard.

[10] Canadian Drinking Water Guidelines specify 1.0 NTU as a treatment objective and 5.0 NTU as an Aesthetic Objective.

The guidelines also set microbial standards based upon counts of coliform bacteria. There are no standards for the presence of viruses, or cysts such as Giardia. New water treatment plant designs should consider the USEPA Interim Enhanced Surface Water Treatment Rules (IESWTR). The IESWTR specifies log removal requirements for protozoa depending on the size of the population and the concentration of protozoa in the raw water.

All new water plants should be designed as a minimum to produce water that meets Canadian Drinking Water Guidelines at all times. In addition, provincial or territorial agencies may have more restrictive requirements, which should also be included in the design requirements.

Just as important, however, is to ensure that the water treatment design addresses developing regulations. Canadian Drinking Water Guidelines (and other agencies such as the US Environmental Protection Agency or the World Health Organization) take 2-5 years to develop a guideline once a parameter becomes a concern. During this process, various draft or "straw man" versions of the guideline may be produced, so it is possible to see what type of regulatory levels are being proposed and what is likely to be in place in the next 2-5 years. Beyond that timeframe (5 or 10 years) it is more difficult to predict changes, but the chances are that guidelines will keep changing and will likely be more restrictive in most cases.

Two very important guidelines under review at the present, by Health Canada, are the Turbidity and Trihalomethane guidelines. The current Canadian turbidity guideline is 1.0 NTU, but concerns with improving removal of small particles to allow better pathogen removal or inactivation are pushing the industry towards lowering of this number. Health Canada will introduce a new guideline in the near future that may increase turbidity removal to 0.5 or 0.3 NTU. Similarly the trihalomethane current guideline of 100 ppb is under review and a new number will likely be produced in the near future that could be as low as 75 or even 50 ppb.

Other guideline levels under review include new limits for Haloacetic Acids (another disinfection by-product), Methyl Tertiary-Butyl Ether (MTBE), which is a gasoline additive, algal toxins, bacteriological quality and a possible lowering of the limit for Arsenic.

Another issue of concern is whether to set any limits for protozoan (Giardia & Cryptosporidium). Because the monitoring methods are not very accurate and are expensive and time-consuming, other agencies such as United States Environmental Protection Agency (USEPA) have gone the route of setting water treatment requirements for control of these organisms (Surface Water Treatment Rule CT requirements). Health Canada to date has not recommended any treatment requirements but may address some of these issues in the turbidity guideline documentation. Some provinces such as Alberta have adopted the USEPA CT requirements and written them into plant operating approvals (e.g. minimum 3-log Giardia, 4-log virus requirements). Alberta may also introduce some requirements for Cryptosporidium in the near future (based on developing USEPA requirements).

The colour, and iron levels in the Schwatka Lake water are sometimes higher than the AO and MAC as set by the CDWG.

A new water treatment plant should be designed to provide 4-log (99.99%) removal or inactivation of Giardia, Cryptosporidium, and viruses. The treatment process should be designed using a multiple barrier approach.

The City has taken measures to ensure that the quality of water better than the Aesthetic Objectives (AO) as set by the Canadian Drinking Water Guidelines (CDWG). However, turbidity levels frequently exceed the Maximum Acceptable Concentrations (MAC) of 1.0 NTU for treatment objectives.

The presence of turbidity can have significant effects on both the microbiological quality of drinking water and on the detection of bacteria and viruses. High turbidity can compromise the effectiveness of disinfection. Coliform, Giardia, Cryptosporidium and viruses can be embodied in particulate, which could prevent adequate disinfectant exposure. Inactivation cannot be assured during high turbidity events, even when the CT requirements are met. The production of low turbidity filtered water has been shown to be effective against the passage of Giardia cysts. Various authorities, such as the American Water Works Association, have adopted a target turbidity of 0.1 NTU.

3.3.3 Groundwater Wells

The long-term continuous supply capacity of the Selkirk Aquifer is 150 L/s with a flow rate of up to 190 L/s on an interim basis (Stanley Associates, 1979). The current ADD demand for the city is 154 L/s

The operations staff has reported that the Selkirk ground water aquifer is utilized near capacity in spring and early summer and cannot support continual operation of all wells without reducing aquifer levels below safe pumping levels. The City relies heavily upon the ground water source to provide water tempering during cold surface water conditions in winter and spring and for turbidity reduction in late spring and early summer.

In 1997 Gardner Lee Ltd. conducted a study entitled “Warm Water Well Development Program”. This report documented the rehabilitation and subsequent performance testing of Well #5 and other City wells. The report goes on to describe the potential for increased well water production. Ground water modeling concluded that the Selkirk Well Field could support a new production well, thus increasing the well water supply to the City. There is a finite production capacity beyond which colder water from the Yukon River will be drawn directly into the well field. The maximum capacity of the well field was considered in the order of 189 l/s (2500 igpm).

The test well was developed in 1997 that was drilled deeper than the existing wells. Although adjacent to the existing wells, the final depth was determined to be a separate aquatard (same parent aquifer but different layer characteristics). The water quality was found to be significantly harder than the existing wells.

Gartner Lee Limited also conducted a geothermal heat pump investigation at Vanier School in Riverdale on behalf of the Energy solutions Center in 2003. Vanier School is approximately 700 meters southeast of the Selkirk Pumphouse. The investigation confirmed the location of two (2) relatively warm aquifers below Vanier School. The warm aquifers were identified as lower Selkirk Aquifer (54.9 m to 60.4 m) with a temperature of 5.8 °C and Miles Canyon Basalt Aquifer (60.4 m to 143.1 m) with a temperature of 6.9 °C to 7.1 °C. The report indicated that a 254 mm diameter well in the Selkirk Aquifer would produce a long-term yield of 25 L/s. The Miles Canyon Basalt Aquifer would produce a long-term yield of 10.6 L/s with one 254 mm diameter well.

Water quality test results from the heat pump investigation were more promising than the City 1997 test well. The report indicated that both aquifers meet Canadian Drinking Water Quality Guidelines and Yukon Contaminated Sites Regulations Aquatic Life standards. The report recommended construction of a heat pump into the Selkirk Aquifer.

There is fish hatchery operated by Yukon Energy located adjacent to the Selkirk Pumphouse. The hatchery is a requirement of Yukon Energy's hydro dam water license and is used to breed Salmon fry for release to the Yukon River. In addition to Salmon fry, the hatchery is also used to hatch other species of fish for release in various parts of the Yukon. The hatchery is a significant user of the Selkirk aquifer and has a ground water well immediately south of the City wells. The hatchery has recently expressed an interest in increasing groundwater usage for its operations. The City has begun initial discussions to come up with a workable solution for both parties' needs.

Due to the importance of the thermal characteristics and lack of turbidity from ground water, the City should conduct further investigations into locating a usable ground water source. Ground water well development is most likely more economical and environmentally friendly in the long term than tempering water with diesel-fired boilers. Even if new aquifers are found to be difficult to treat for mineral content and hardness the City should investigate the potential use of heat pumps to temper surface water.

The 1997 Gartner Lee study was conducted based on limited information. Groundwater studies tend to be theoretical and there has been some evidence of different aquifer characteristics in the Riverdale area.

The groundwater supply is also important because it provides a secondary source of water for the City in the event of surface water contamination. The Schwatka Lake watershed is impacted by current usage and upstream development pressures.

This study recommends new watermains from Selkirk Pumphouse to south and east Riverdale to improve fire flows and thermal conditions. If a viable aquifer is located along the new watermain alignment, it may be possible to connect a new chlorinated well source to the distribution system using the new alignment; however, CT required for disinfection may be prohibitive.

3.3.4 Blending of Surface and Well Water

The blending of cold water from Schwatka Lake and warmer water from the wells is done during the winter months to help prevent the water distribution system from freezing. Well water temperatures have historically varied from 2.1 to 9 ^{o}C depending on the time of year and usage. High well demand can result in excessive aquifer draw down and river water can short circuit to the aquifer. Schwatka source water has historically varied from 1.2 ^{o}C (winter) to 13 ^{o}C (summer).

The objective water blend temperature is 4 ^{o}C. During normal operations (Well 6 and a booster for lead and a combination of Wells 1, 4 & 5 and a booster for lag) the current blending ratio well water to Schwatka Lake water is approximately 1 to 1. Currently, the existing wells have sufficient capacity to maintain the required blending volume to prevent freezing.

As the City grows, not only must more water be supplied to meet the demand, but the thermal requirements of the supplied water also increase. Increasing source temperature reduces the need to introduce water tempering or bleeding within the water system. A complete thermal evaluation of the water distribution system is presented in Section 4.3 of this study.

Blending for the purposes of turbidity reduction has proven to be a problem for the City. The current Maximum Day Demand for the City is 308 L/s and the sustained capacity of the aquifer is believed to be 150 L/s. During times of high demand and high surface water turbidity, the City has difficulties supplying water below 5 NTU.

In 2002, spring run off surface water turbidity levels was high into June. Because of warm weather, this coincided with the time that people wanted to water lawns and gardens. The blending ratio used to maintain turbidity levels was approximately 90% well water and 10% surface water. The City issued voluntary water restrictions and came very close to issuing mandatory restrictions or a boil water advisory.

3.3.5 Long Term Water Supply Quality Security

Historically Schwatka Lake has been identified as a long-term water supply source for the City of Whitehorse. This source is not without risks as the lake is used for recreation and commercial activities, such as, a floatplane base and commercial boat tour activity. With the consideration of water quality issues such as turbidity and pathogens the City is revisiting the concept of increasing supply from the Selkirk aquifer. In addition, the City is currently undertaking a Water Shed Management Study to address watershed quality and security.

3.3.6 Treatment Plant Options

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Water treatment is a complex issue and the best solution is beyond the scope of this study. Based on preliminary water quality results the main objectives of a treatment plant for the City of Whitehorse will be turbidity avoidance or removal and pathogenic organism inactivation or disinfection.

The objectives of the 2001 Water Treatment Plant – Feasibility Report (Epcor) were to identify the need for treatment upgrades and identify a preliminary treatment option for budgetary purposes. The 2001 report focused primarily on the conventional treatment option of Coagulation/Flocculation, rapid sand filtration and disinfection by means of chlorinization and ultra violet treatment. Based on the findings of the Epcor report the City recognises the need for a water treatment plant.

Some other options for turbidity removal include slow sand filtration and membrane filtration.

At some point the City was investigating the possibility of constructing a contact chamber at Selkirk to improve chlorine CT (residual chlorine concentration multiplied by effective contact time in minutes) for disinfection. CT is required to inactivate viruses; however, reasonable dosage and contact times have been found to be not very effective at inactivating larger organisms, such as, Giardia, Cryptosporidium.

Based on United States Environmental Protection Agency recommendations, a total of 4-Log inactivation was recommended by Epcor. The Epcor study identified the use of a clear water contact chamber to provide additional 1-Log inactivation above that provided by ultraviolet, coagulation/flocculation and rapid sand filtration.

Recent studies into the use of ultraviolet disinfection have proven promising for inactivation of all pathogenic organisms. Past studies found that ultraviolet was not an effective killer of Giardia and Cryptosporidium; however, recent studies indicate that ultraviolet affects Giardia and Cryptosporidium DNA. In other words, the ultraviolet dosage may not kill the organisms but it affects their DNA and ability to reproduce. The effectiveness of ultraviolet treatment is highly depended on water turbidity and therefore should only be used in conjunction with turbidity removal. Ultraviolet does not provide a residual for downstream disinfection; therefore, chlorination is still required for protection of distribution system.

Ozone disinfection has been widely adopted in Europe and parts of North America before the effectiveness of ultraviolet was better understood. Ozone disinfection has proven to be a complex and expensive treatment option.

Chlorine dioxide provides much longer residual protection than chlorine gas. However, a chlorine dioxide system would be more expensive than a chlorine system for residual disinfection because it requires both chlorine and sodium chlorite. Chlorine dioxide may also not favourable because of the potential for formation of chlorate and chlorite residuals.

Before commencing detailed design of a water treatment plant the City should undertake further study and pilot projects to ascertain the best treatment process.

Further investigation into the water supply sources should be conducted. Water quality and temperature profiling of Schwatka Lake may indicate that intake relocation will be beneficial. Additional groundwater source and quality investigation is important. Increased ground water supply will decrease the requirement for turbidity removal and improve system thermal characteristics during cold water periods.

As a pilot project, the City may also want to investigate the potential use of a large-scale infiltration gallery to reduce Yukon River turbidity. One possible option would be to install large diameter perforated pipes that parallel the existing intake supply line. Since an infiltration gallery would be classified as groundwater under direct influence of surface water, filtration would still be required but lower expected turbidity levels may influence filtration design and maintenance.

3.4 FIRE FLOW TESTING

In order to develop an accurate water model, system physical characteristics need to be entered, such as, pipe lengths, pipe diameters, pipe elevations, pump operating curves, control valves set points and reservoir locations. The water model then computes watermain head loss due to friction and system operating characteristics (velocities, flows and pressures) by Hazen-Williams equation. Hazen-Williams equation relies on a unit-less C value to account for pipe wall friction. Since individual pipe C values are generally assumed when building a model, adjustment of C values are used to calibrate water model results against known flow characteristics.

Measuring the actual water distribution system under high demand (high head loss) provides more accurate data for model calibration; therefore, hydrant flow testing was chosen to calibrate the water model.

Hydrant testing locations were determined and procedures were developed in consultation with the City staff. Testing procedures included recording of boundary conditions (system reservoir levels, PRV conditions and booster states), hydrant flow rates and residual pressures (HGL). Flow testing was completed in July 2002.

The hydrant flow testing sites are depicted in Figure 3.1 and findings are summarized in Table 3.2.

Table 3.2 Hydrant Flow Testing Results

Site No.	Location	Area	Flow Hyd No.	Flow Hyd Elev (m)	Flow (L/s)	Press. Hyd No.	Press Hyd Elev (m)	Press Hyd HGL (m)	Press (kPa)
10	16 Tigereye	Copper Ridge	51	794	76	52	794	829.1	269
11	Finch and Nut Hatch	Hillcrest	11	761.27	72	12	762.61	790.7	241
12	#1 Hayes	Granger	21	774.94	55	20	776.75	808.4	124
9	Ponderosa	Porter Creek	113	732.51	76	114	739.74	797.3	220
8	Sycamore and Willow	Porter Creek	136	721.71	85	10	725.22	770.2	427
16	Centennial & 17th Ave	Porter Creek	11	724.31	80	10	725.22	768.8	365
7	Kathleen and Squanga	Crestview Old Area	4	715.44	58	3	720.33	764.6	145
4	Hillcrest Drive @ Park Lane	Hillcrest	13	719.73	62	9	720.18	755.3	172
20	Burns Road	Hillcrest	22	702.95	77	23	702.90	755.6	427
14	Transport. Museum	Airport	35	702.77	85	29	701.90	757.4	503
5	Kopper King	Kopper King	2	724.06	76	3	724.58	782.9	462
3	Yukon College	Takhini	16	713	70	15	713	755.1	303
19	Range Road @ Takhini Arena	Takhini	19	693.53	198	22	694.05	757.3	489
21	Tlingit Road	Marwell	8	633.82	79	24	634.77	691.0	496
6	Copper Road	Marwell	17	635.20	74	18	635.29	690.8	393
18	Chilkoot Centre	Downtown	?	633	103	?	633	672.3	331
17	6th & Main	Downtown	228	636.76	37	288	637.10	675.7	103
15	Lewes Blvd.	Riverdale	135	642.54	74	218	642.93	679.5	338
1	Hyland Cres.	Riverdale	177	644.77	55	175	644.98	679.4	248
13	McClimon Dr	McIntyre	11	723.03	154	4	723.84	753.3	138
2	#2 Hospital Road	Riverdale	207	642.05	72	208	642.43	677.5	324

In order to calibrate the EPANET water model, hydrant flow tests were modeled and C values were adjusted until model results matched field hydrant flow test results. Table 3.3 provides a comparison of field flow data and model results.

Table 3.3 Field Fire Flow and Model Comparison[11]

Flow Hydrant Number	Residual Hydrant Number	Flow Model Node	Residual Model Node	Flow Hyd/Node Difference Static HGL (m)	Flow Hyd/Node Difference Flow (L/s)	Press Hyd/Node Difference Static HGL (m)	Press Hyd/Node Difference Residual (m)
11	12	19180	19170	-9.1	3.2	-7.8	0.0
21	20	14230	14250	-6.8	8.0	-4.9	-9.4
113	114	16760	16580	-15.3	31.7	-8.1	2.4
136	10	15230	15840	-7.6	-1.2	-4.2	-2.0
11	10	15900	15840	-6.5	6.2	-5.6	-2.1
4	3	18360	18290	-14.7	21.4	-9.8	-5.3
13	9	11030	11050	-6.4	13.7	-6.3	-8.8
22	23	11440	11450	-6.3	1.0	-6.4	-9.1
35	29	11600	11590	-3.7	6.0	-4.6	0.0
2	3	16880	16880	-1.0	19.8	-0.5	8.4
16	15	16900	16900	-4.3	4.0	-4.3	-7.2
19	22	6070	6440	-3.8	5.7	-3.2	-7.9
8	24	5120	5110	-3.5	9.5	-2.6	4.4
17	18	5080	5070	-2.9	13.7	-2.8	-6.9
new	new	4220	4270	-6.2	-2.6	-6.2	-9.3
228	288	3400	3410	-4.0	6.1	-3.6	-21.7
135	218	1470	1430	-0.7	2.4	-0.2	0.4
177	175	1560	1570	-0.4	16.6	-0.2	1.4
11	4	13050	13110	-6.9	11.4	-6.1	-7.5
207	208	2060	2100	-2.0	-0.9	-1.6	-2.7

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[11] Model node location does not exactly match hydrant location as model nodes represent pipe junctions

3.5 TRANSMISSION FACILITIES

3.5.1 Water Storage Reservoirs

The City of Whitehorse water distribution system utilizes five storage reservoirs. The details of each of these reservoirs are summarized in Table 3.4.

Table 3.4 Water Storage Reservoirs[12]

Name	Service Area	Total Capacity (m^3)	Effective Capacity (m^3)	Bottom Elevation (m)	Overflow Elevation (m)
Riverdale	Area 1	13,406	12,076	675.0	680.0
Valleyview	Area 2	5,508	5,032	754.0	760.0
Hillcrest	Area 4	6,058	6,058 [13]	796.0	800.0
Copper Ridge	Area 5	7,285	6,449	825.5	830.5
Porter Creek	Area 3	5,716	5,190	770.0	777.0

A dead storage capacity reduction was made to calculate Effective Capacity due to inlet geometry or potential vortexing issues.

If a reservoir does not have an adequate sump, anti-vortexing plate or baffles to prevent vortexing, additional volume must be deducted. Vortexing occurs when water swirls as it drains. The flow rate available from a reservoir will decrease as vortexing increases.

The reservoir vortexting elevation was estimated by calculating the amount of head required before a negative pressure occurs at each reservoir cell outlet under fire flow conditions.

[12] Total Capacities were provided by the City and is the volume to overflow. Effective Capacity calculated by Stantec.
[13] Vortexing issues are not anticipated at Hillcrest Reservoir.

The following equation was used to estimate the votexing elevation:

$$h_f = 2.8\frac{V^2}{2g}$$

where:

h_f is the head required under fire flow conditions

2.8 is a constant to represent outlet conditions

V is the velocity at each outlet during fire flow

g is the acceleration of gravity at 9.81 m/s

The head required was calculated from the point of highest velocity in the outlet piping.

The vortexing calculations assume that no boosters were running at the supply pumphouse since fire storage calculations have to be based on the same assumption. Under normal conditions, it is anticipated that pumphouse operators will respond to low reservoir levels and ensure supply pumps are operating and dealing with any other issues that may arise.

Vortex remediation is not recommended at this stage, as more detailed calculation and risk assessment is required. Vortexing was presented in this study only to provide awareness of an issue and assess a worst-case scenario.

Possible vortexing remediation measures may include installation of anti-vortexing plates or baffles. A plat is a horizontal installation above a vertical drain. A baffle is a vertical installation. Anti-vortexing plates or baffles maybe prefabricated from corrosion resistant materials prior to any reservoir maintenance activities and installed during maintenance shut down.

The constant of 2.8 was an assumed value applied to all reservoirs votexing calculations. Ideally, the constant should be modified to represent conditions at each reservoir outlet.

An assessment of reservoir operating levels was not part of the study scope. Maximum reservoir capacities were assumed to be to overflow elevations. Operating reservoirs at lower levels than dictated by storage requirements requires an assessment of risk as both storage and pressures can be impaired. An assessment of reservoir operating levels is recommended and the results should be posted at relevant pump stations, reservoirs and SCADA systems.

An assessment of reservoir mixing was not part of the study scope. Many water system operators in recent years have begun to introduce measures to improve reservoir mixing in order to improve reservoir thermal and chlorine residuals. It can easily be assumed that poor reservoir mixing will result from having a shared inlet and outlet.

Under normal operating conditions poor reservoir mixing may not affect the distribution system, as chorine residuals and thermal conditions may always be adequate in the vicinity of the reservoir inlet and outlet. High demands, system breaks, supply disruptions or fire flows may result in higher reservoir demands, which could introduce poor chlorine residuals to the distribution system.

An assessment of reservoir mixing any remedial measures is recommended. New internal reservoir piping and check valve systems may be considered in the future to improve reservoir mixing. These systems may also improve vortexing as more outlets can be introduced.

3.5.2 Pump Facilities

The City of Whitehorse operates 9 water distribution pump stations and 7 circulation facilities.

Table 3.5 details each major pump station configuration and Table 3.6 details the circulation facilities.

Table 3.5 Existing Pump Facilities Configuration

Pump Station	Duty Pumps		Standby Pumps		Circulation Pumps		Boilers	ADD Rated Discharge One Pump [14] (L/s@Head)
	No	Power (hp)	No	Power (hp)	No	Power (hp)		
Selkirk	2	100	1	100	-	-	-	183@26m
Two Mile Hill	3	300	1	300	-	-	-	195@87m
Hamilton Boulevard	2	40	-	-	2	7.5	3	40@44m
Copper Ridge	2	25	-	-	3	15	2	39@32m
Mc Intyre	2	50	1	Elec/ Diesel	-	-	2	71@28m
Crestview	2	10	1	Diesel Pump	2	5	1	1.9@32m
Granger	1	10	1	10	-	-	-	28@16m
Ponderosa	1	5	-	-	-	-	-	8@24m
Grove St.	1	5	-	-	-	-	-	8@24m

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[14] Note that pump discharge rate and head depends on demand conditions.

Table 3.6 Existing Circulation Pump Facilities

Name of Circulation Station	No. of Pumps	Power (hp)	Boilers	ADD Flow (L/s)	Head (m)
Crestview (Upper Area)	2	5	1	21	6.8
Hamilton	1 (McIntyre) 1 (Arkell/Logan) 1 (Granger)	7.5	3	39 35 -	8.4 9.4 -
Copper Ridge	1 (Zone 1) 2 (Zone 2)	15	2	39 49	20 13.2
Redwood	1	1.5	-	7.6	8
Hart Crescent	1	1.5	-	11	4.6
Centennial	1	5	-	12.6	23
Kopper King	1	1	-	0.7	24
Transit	1	1	1	9.0	17
Yukon Electric	1	1	-	4	10
Mountainview Place	2	3	-	8.5	14

The circulation pump for Granger located at Hamilton Boulevard Pumphouse is no longer required since the construction of Granger Booster. There is an additional circulation facility on Twelfth Avenue at Fir Street that is no longer operated.

The Yukon Electric Circulation Facility was no longer be required for circulation of the City watermain after Platinum Street watermain looping was constructed in 2003.

3.5.3 Pressure Regulating Facilities

The City of Whitehorse has 9 pressure regulating facilities. These control the pressure either upstream (PSV) or downstream (PRV) of the station. Flow control may also be a component of the station depending on the physical need.

The salient features of these pressure reducing/sustaining stations are presented in Table 3.7.

Table 3.7 Characteristics of Pressure Reducing/Sustaining Stations

Name	Type	Ground Elev. (m)	Upstream HGL (m)	Upstream Pressure (kPa)	Downstream HGL (m)	Downstream Pressure (kPa)
Ponderosa	PSV	733.0	805.43	745	773.55	435
Grove	PSV	749.2	769.89	435	737.50	248
Elvins	PRV	692.47	761.69	690	748.83	565
Ortona	PRV	701.2	764.48	642	745.12	455
Range Road	PRV	684.3	769.89	863	737.5	545
Wilson	PRV	761.34	812.93	566	801.64	459
Thompson	PRV	757.28	812.24	528	801.38	421
Marwell	PRV	644.8	764.2		693.6	
Crestview	PRV	723.8	807.2		775.1	

3.5.4 Bleeders

Bleeding occurs at the individual residences, commercial establishments and un-looped or dead end watermains to prevent the water system from freezing. The bleeders drain directly into the sanitary sewer system and some bleeders are used to keep the sanitary sewer from freezing.

Most off-line fire hydrants (primarily in Riverdale and Downtown) rely on a residential or commercial services connected to the hydrant to prevent freezing. These residences or commercial business have bleeder to prevent freezing of their service and the City's hydrant.

The City maintains about thirteen (13) 19 mm uncontrolled bleeders with a Dow valve (flow rated orifice) for flow regulation. These uncontrolled City bleeders represent about 6.2 percent winter demand or about 3.2 annual demand.

In addition, the City also has three 19 mm thermostatically controlled bleeders (TCB) and has installed small diameter TCB's in approximately 1,500 households and businesses in the Hillcrest, Riverdale and Downtown areas.

Bleeders are discussed in detail in Section 2.6.3

3.6 WATER DISTRIBUTION SYSTEM MODELING

3.6.1 General

In order to simulate citywide water system characteristics a water model is required. EPANET was used by this study for water modeling. A description of EPANET and past City of Whitehorse water models is provided in Section 2.7.

3.6.2 Model Update

The existing City model was reviewed and the physical components updated to reflect current system conditions, including the addition of any outstanding past capital works project not yet included in the existing model. The model was updated in order to provide the correct spatial coordinates for the network nodes so that model components can be overlaid with the existing city aerial photograph or legal base. Further, City staff familiar with existing infrastructure can now immediately recognize the respective model components.

A functional GIS was developed in order to manage the modeling data for both the water system and the sewer system. This integrated tool will assist in maintaining data directly and indirect applicable to the models. Information such as base parcel, land use, population, sewer drainage areas, thermal probes, etc. was included and other facilities or themes can be added to the GIS by the City in the future if desired. A firm understanding of each existing facility (pumphouse, circulation station, reservoir, PRV, PSV) is obtained in order to ensure that the model will correctly simulate the facility.

Water model selection is described in Section 2.7 and model calibration is described in Section 4.1.1.

3.6.3 Demand Analysis and Update

Based on water demand analysis in Section 2.6 and the application of the GIS software, the estimates of nodal demands were produced and inputted into the EPANET computer model. These nodal demands were then as finely adjusted as possible depending on the resolution of available land use and population data. The model was then calibrated based on hydrant flow testing by adjusting pipe roughness (C values).

3.6.4 Review Temperature Monitoring Data

Thermal properties of the water in the distribution system are affected by the following environmental parameters:

- Raw water temperature in Schwatka Lake
- Groundwater temperature of the Selkirk aquifer

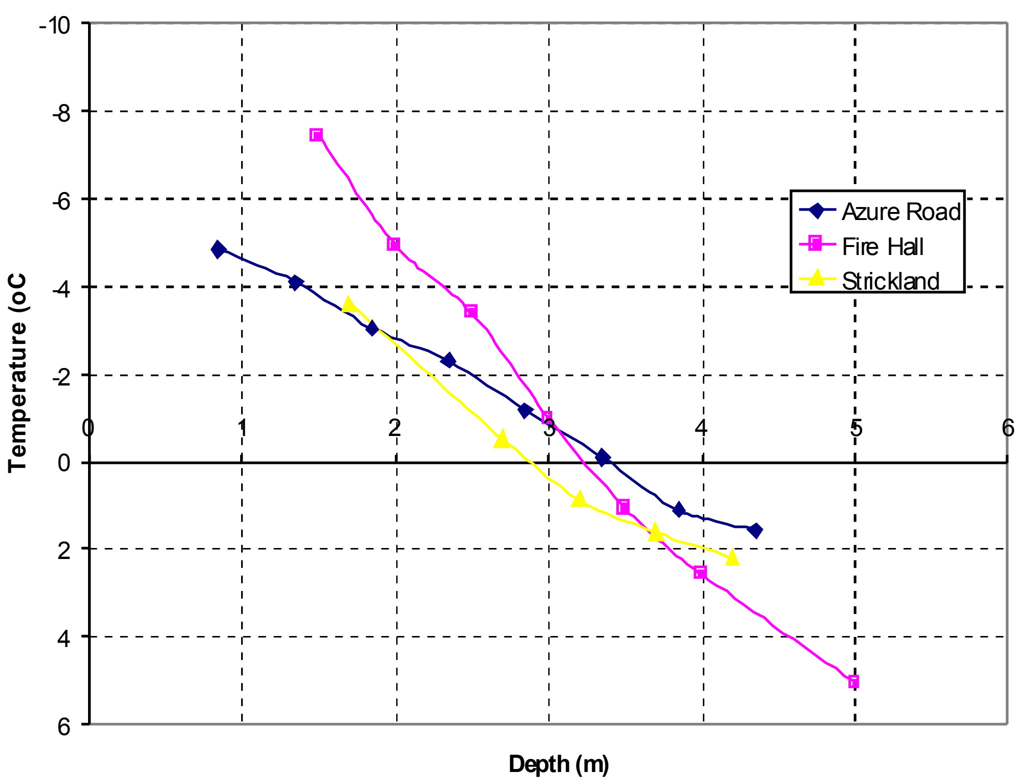
- Geothermal compositions of the soil
- Climate conditions
- Pipe materials and burial depths
- Characteristics of soil surrounding pipe

A significant amount of information is being collected for thermal parameters on an ongoing basis. The City is monitoring subsurface temperatures at thermisters located at strategic locations throughout the City.

The City recognises the benefit of ground thermal monitoring for both thermal model calibration and operations requirements, such as, when to advice the public to turn off frost protection devices and when City circulation stations can be turned off. The City has recently begun installing new thermisters at strategic locations during regular digging operations. Also, permanent thermisters should be installed during new developments.

The thermister data collected during the course of this study have been analyzed and presented graphically in Figure 3.2.

Figure 3.2 Thermister Readings 2002



Based on this review of the data and the previous studies carried out by Stantec, it has been concluded in this study that the worst case meteorological scenario (winter of 1971-1972) as established by Stantec in 1995 is still valid and has been utilized for evaluating the performance of the system under both existing and future development conditions. This study also utilized the soil properties and burial depths as used by the previous study. A sensitivity analysis has also been carried out to investigate the effects of variation of various parameters in simulating the thermal response of the water system.

The City is also monitoring the temperature data at various reservoirs and at the water circulation facilities. These as well as all other water temperature data collected throughout the system have been collected and reviewed to identify implications on the water system.

4.0 Assessment of the Water Supply System

4.1 GENERAL

The adequacy of the various elements of the City's water distribution has been evaluated. This evaluation includes the water supply source, water quality, pumping capacity, reservoir capacity, transmission and distribution mains capacity. The calibrated model has been used to examine the behaviour of the existing system with respect to the desired levels of service using the set design criteria and common water distribution system standards. The model has been split into two operational scenarios; namely winter and summer operational mode. Each model has been run under various hydraulic loading conditions to assess the hydraulic performance of the water system under the existing development conditions. The modeling analysis consists of the following hydraulic loading scenarios:

- Average Day Demand
- Maximum Day Demand
- Peak Hour Demand
- High Pressure / Reservoir Filling Demand (Night Filling Demand)
- Maximum Day Demand plus fire flows at various critical locations

4.1.1 Model Calibration

The updated model was calibrated to available system performance data. This data includes recorded system conditions. This calibration process is necessary to ensure that the model accurately reflects the behaviour of the system. With the model constructed and supplied with initial parameter estimates, initial simulations are executed and compared to the available system performance data. The comparison indicates that the existing updated model simulates the existing hydraulic condition reasonably well.

4.1.1.1 Preliminary Calibration

Available pressure gauges were inspected and the relative pressures and geodetic elevation at each of these locations recorded. This facilitates the calculation of the hydraulic gradeline (HGL) at each of the locations. The HGL is used as a common base point. At each source (pump, reservoir) it becomes the reference for the entire service or pressure zone. If no flow occurred this would be constant over the entire zone. As flow is introduced, the HGL drops due to headloss in the direction of flow.

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Primarily the pressure recordings were performed at major facilities such as reservoirs, pumphouses, and control valves. This information was presented in section 3.5. It primarily describes the system under average daily demand. The following is a discussion on the observations of this data.

4.1.1.2 Final Calibration

Upon acquisition of the data from the fire flow testing field program, final calibration of the model has been performed. Calibration of the model has been achieved by altering individual pipe roughness until recorded pressures were simulated in the model to within 5% of expected values.

Trial 1 of the calibration procedure used the initial Hazen-Williams roughness values developed from the original model. Trial 2 of the calibration procedure is a result of changing the Hazen-Williams C values of the supply pipeline. Trial 3 of the calibration procedure, observed the model converge to expected pressures within 5% by altering the Hazen-Williams C value of the supply pipelines within the test site. All other pipe roughness has not been changed in the calibration procedure.

Fire flow field-testing results were presented in Section 3.4.

4.1.1.3 Thermal Calibration

In order to calibrate the developed thermal model, a field temperature measurement program has been carried out during the week of April 10th, 2002 to measure the actual distribution system temperature at strategic locations throughout the City. The thermal model was then calibrated to represent the actual field conditions.

The key parameters that have been used in the computation of temperature distribution within the water distribution system are as follows:

- Pipe material type
- Burial depth
- Soil type and soil thermal conductivity
- Ground Temperature
- Source Temperature

Pipe materials and burial depths are relatively well defined and are reasonably accurate. The typical ground temperature based on analysis of the monitored data has been used as input temperature. The measured input water temperature at various locations has also been used as point source of heat into the model. The thermal conductivity of the soils, however, has a broad range of values for each soil group. The typical values of the soil group as utilized in the previous studied have been used in this study. The summary of the calibration results and the comparison between the field data and the simulated data in included in the appendix. It can be inferred that the model simulates the field condition reasonably well.

The calibrated model has then been applied to assess the sensitivity of various parameters used to compute the heat loss through the system. It is found that the model is not very sensitive to the variation of soil conductivity but it is very much sensitive to the temperature gradient (difference of water temperature and ground temperature). The following provides some discussions of the important thermal features of the water system encountered during the calibration process.

Some areas show a large variance between the measured and modeled temperatures. These variance my be due to localized conditions causing an increase or decrease in expected temperatures. Since these could not be explained via any physically based means a conservative approach was adopted. Matching exactly to the measured temperature may introduce a risk in the overall system performance.

The calibrated thermal model has then been applied to assess the thermal performance of the existing water distribution system under the design winter condition of 1971-1972.

4.2 HYDRAULIC EVALUATION

The existing water distribution system has been evaluated based response of system components under various loading conditions defined in Section 2.5. Loading conditions include average day demand (ADD), maximum day demand (MDD), peak hour demand (PHD) and high pressure / reservoir filling demand (night filling demand NFD).

Analysis of system components is provided in the following sections.

4.2.1 Pumping Facilities

For purposes of clarity, the term pumphouse will refer to facilities that contain both booster and circulation pumps. A booster station will contain only booster pumps and circulation station only pumps for circulating flows.

Table 4.1 identifies existing pumphouse ADD and MDD flows and pumping capacities.

Table 4.1 Existing Pumping Requirements

Pumpstation	Demand		Capacity			Back-up Capability	Reservoir MDD Notes
	ADD (L/s)	MDD (L/s)	Pumps Required @ MDD	MDD Capacity (L/s)	Head (m)		
Selkirk	153.92	307.8	2 of 2 duty	341	30.0	1 Booster on Generator	RD drain at 0.3 L/s
2 Mile Hill	75.83	151.7	1 of 3 duty	198	85.6	2 Boosters on Generator	VV drain at 2.0 L/s
McIntyre	34.14	68.3	2 of 2 duty	89.2	37.1	80 l/s Diesel Pump	PC fill at 24.4 L/s
Crestview	3.70	7.4	1 of 1 duty	7.4	28.0	Diesel Pump	No reservoir
Hamilton	13.21	26.4	1 of 1 duty	40.5	43.3	None	HC drain at 18.1 L/s
Copper Ridge	3.57	7.1	1 of 1 duty	39.3	32.0	2 Booster on Generator	CR fill at 32.2 L/s

Table 4.1 does not recognise standby pumps as the purpose of a standby pump is for backup during maintenance or failure and should not be relied upon to meet capacity.

4.2.1.1 Selkirk Pumphouse

The Selkirk Pumphouse is located in Riverdale on the east bank of the Yukon River and distributes water to the entire city. The Selkirk Pumphouse is location by plan in Figure 2.2 and by water system schematic in Figure 2.4.

Water is supplied to the station from Schwatka Lake via a 550 mm gravity intake line and nearby ground water wells. A new siphon above high water line and valve chamber was constructed at Yukon Energy dam in 1999. The two intakes into Schwatka Lake were not reconstructed. In addition to the new siphon and valve chamber, 186 meters of the supply line to Selkirk has been twinned with 500 mm lines.

This station has three Gould horizontal split case 100 hp pumps (2 duty and 1 standby). The MDD capacity of the Selkirk Pump Station is approximately 341 L/s at a total dynamic head (TDH) of 30.0 m. Selkirk Pump Station distributes water to Area 1, provides supply flows to Two Mile Hill and fills the Riverdale Reservoir.

The current average day demand for the city is 152.3 L/s. Selkirk Pumphouse can meet this demand with one pump running at 182.7 L/s.

The current MDD for the city is 307.8 L/s. This station currently has to operate near its maximum potential of 341 l/s with 2 of 3 boosters running to meet maximum day demand conditions.

The 1990 Water and Sewer Study recommended this station be upgraded to accommodate three 150 hp pumps with the capacity of delivering ultimate Maximum Day Demand of 710 L/s with a TDH of 34 m. This work was recommended after completion of the Crosstown Watermain and the Two Mile Hill Booster Station, which have since been completed.

This study recommends replacement of Selkirk Pumphouse at the time of a water treatment plant construction. Currently Selkirk Pumphouse has horizontal pumps that require a suction head. Vertical turbine pumps will likely be required due to the loss of suction head in the treatment process. The current booster pumps are highly dependent on the intake head provided by Schwatka Lake and the well pumps. With a combination of well water (150 L/s) and lake water supply (208 L/s), the suction HGL was modeled at 651.2m. The station ground elevation is 636.19 m.

In 1990 it was also reported that the raw water supply line capacity was 500 L/s at a head loss of 11 m. If this scenario holds true the future pumps will have to overcome a head of approximately 41 m in order to supply the required MDD.

Two other alternatives are available to address this issue. Twining the supply line would reduce the headloss and inlet has already been partially twinned as part of the inlet siphon construction. The other option would be to increase groundwater supply.

If the Selkirk Pumphouse is reconstructed backup power should be provide to meet control system MDD pumping requirements.

In the past the City has experienced very low water pressures in Riverdale, the hospital area and Downtown when two boosters were running at Two Mile Booster with no pumps operating at Selkirk Pumphouse. This situation is not reflected in the water model and further investigation may be warranted. The situation my have been caused by a closed or partially closed valve in the distribution system. If the situation still exists and cannot be remedied, the City should confirm that current SCADA control would not start two boosters at Two Mile Booster unless a booster is running at Selkirk.

4.2.1.2 Two Mile Hill Booster Station

The Two Mile Hill Booster Station is located at the junction of Industrial Road and Two Mile Hill Road just north of Downtown. It conveys water from the Downtown area to the upper area of the City as well as Marwell. Based on the recommendations of the 1990 Water and Sewer Study, this pump station was reconstructed in 1997. At present, the station consists of four 300 hp horizontal split case pumps (3 duty and 1 standby). These pumps have been designed to satisfy the Maximum Day Demand of 345 L/s for the projected population of 20,640 people by Year 2017 based on the existing system head characteristics. The station is sized to accommodate the fifth pump to meet the ultimate Maximum Day Demand of approximately 500 L/s for the projected ultimate population of 39,366 people. The station is also equipped with an emergency generator rated at 750 kW (938 kVA, 600 V, 3ϕ, 3 W, 1800 RPM, 60 Hz), capable of operating two of the stations booster pumps. This provides significant reliability and peak power shaving to the overall system.

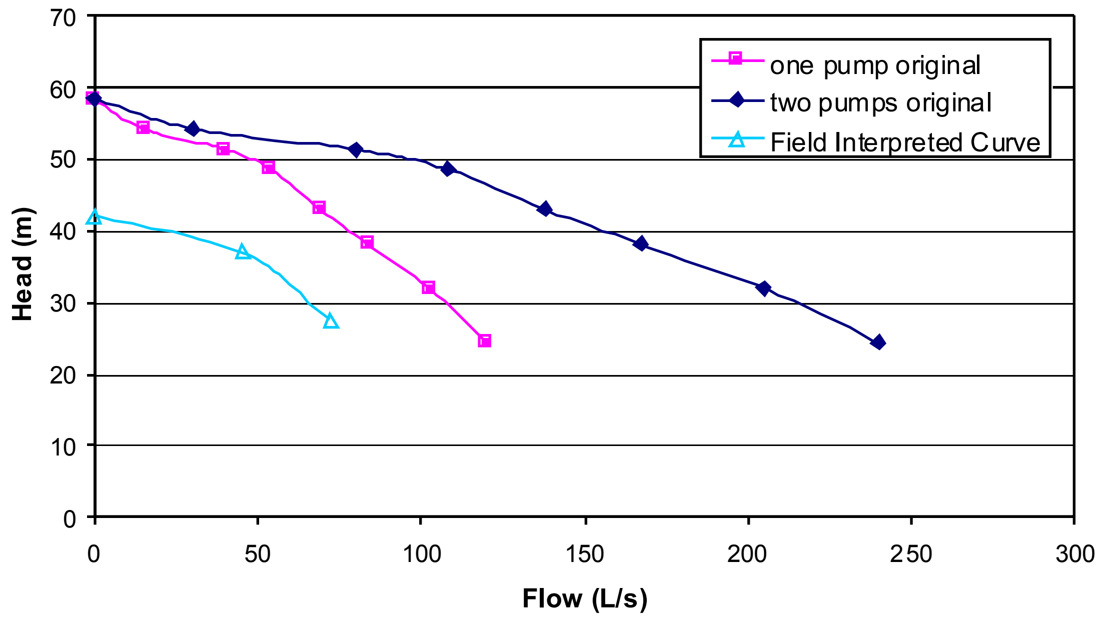
4.2.1.3 McIntyre Creek Booster Station

The McIntyre Creek Booster Station is located between Takhini and Porter Creek. The pumphouse boosts line pressure and delivers water to the Kopper King area and the Porter Creek Reservoir. The pumphouse has two 50 hp horizontal split case duty pumps and one 75 hp standby pump is dedicated for standby use and fire flows. The station is equipped with two boilers for heating water. The capacity of each of the duty pumps according to the published pump curve is approximately 54 l/s at a TDH of 48 m. The diesel drive fire pump is rated at 80 L/s at TDH of 39 m.

Field data indicated that the duty pumps are operating closer to 28.8 m as inlet and outlet pressures of 593 and 876 kPa (86 and 127 psi) respectively were recorded during pumps operation and 634 and 792 kPa (92 and 115 psi) with no pumps on. This may be due to trimming of the pump impellers or inaccurate gauges. The City may wish to inspect and compare impeller dimensions to manufacturer specifications during annual maintenance activities.

Figure 4.9 illustrates the pump curves from the original water model for McIntyre Creek Pumphouse and the interpreted pump curve from field measured data.

Figure 4.9 McIntyre Creek Pumphouse Pump Curves



The pump curve from field interpreted data was adopted for study water modeling.

The MDD for the station is 68.3 l/s. The station has a capacity of 89.17 L/s with both pumps running. As will be identified later the upstream Porter Creek reservoir is under capacity; therefore, it is recommended that hydraulic capacity upgrades be considered.

The 1990 study did not recommend any upgrading, as they were satisfactory with the existing and future requirements. However, the 1999 Pumphouse and Lift Station Audit study recommended that an engineering review of the pumphouse hydraulic capacity and the adequacy of the existing pumps be conducted. It was also recommended that the existing 75 hp diesel standby pump be replaced with a new standby pump with the hydraulic capacity equal to or larger than the existing duty pump. Further, the operations staff have reported 18 hour per day run times during times of high water demand. The operations staff also indicated that although the 75 hp diesel engines has very low hours it may be unreliable due to the age of the equipment and lack of exercising.

Standby power with enough capacity for control system requirements and ultimate maximum day demand pumping should be installed at this station. The standby generator should also be able to meet ultimate averaged day demand requirements with full boiler operation.

Due to some uncertainties raised by operations staff and the 1999 Pumphouse and Lift Station audit, further investigation into actual station demand and capacity may be warranted. It should also be noted that the existing Mag meter used to record station flows should be evaluated for accuracy and recalibrated or replaced as required.

As discussed in Section 4.2.2, fire flow conditions are a concern in some areas of Ponderosa Drive and Grove Street. Even with two pumps running at McIntyre, the residuals pressures are below the minimum standard.

SCADA upgrades and a backup generator installation are planned for 2003 at this station.

4.2.1.4 Ponderosa and Grove Booster Stations

There are currently no known issues with Ponderosa and Grove Booster station performance; however, during fire flow conditions there is a potential for negative system pressures in these areas if there is no booster running at McIntyre Creek Pumphouse. One possible solution is to install pressure sensors in these areas that are directly connected to McIntyre Creek booster control. If McIntyre creek is ever upgraded it may also be possible to interlink Ponderosa and Grove Boosters so that they do not operate when McIntyre Creek Boosters are providing enough pressure to these areas.

4.2.1.5 Crestview Pumphouse

The Crestview Pumphouse is located on Azure Road in the Crestview neighbourhood. The pumphouse boosts operating pressure, provides circulation flow and water tempering by means of a small boiler to the upper area of Crestview. The lower area of Crestview is supplied from the high-pressure side of Crestview boosters via PRV's located in the pumphouse.

There are currently no known performance issues with Crestview Pumphouse. The booster pumps and circulation pumps run in series and may not be as efficient as a single pump that can provide both the required head and volume required for boosting and circulation. According to operations staff the circulation pumps have been run year round in conjunction with the boosters for thermal and water quality circulation. Table 4.1 reports a booster capacity of 7.4 l/s and an MDD requirement of 7.4 l/s. It should be noted that since Crestview does not have an upstream reservoir, pumping capacity will match demand. Also, the circulation pumps greatly improve flow capacity when they are operating.

A distribution system interconnection between the upper and lower area is recommended in Section 4.3.1. If the recommendation is implemented, the City may want to review Crestview pump characteristics and possibly replace the booster pumps and circulation pumps with a duty and standby pump capable of provide enough required head and circulation.

4.2.1.6 Hamilton Boulevard Pumphouse

The Hamilton Boulevard Booster Station is located near the intersection of Hamilton Boulevard and McIntyre Drive and fills the Hillcrest Reservoir using two 40 hp horizontal split case high speed booster pumps. The pumphouse also circulates water through, Arkell-Logan and McIntyre Neighbourhoods using two, 7.5 hp end suction pumps. The operating point of the booster pump is approximately 39 L/s at TDH of 45 m. Each of the circulation pumps circulates approximately 39 L/s at TDH of 9m. Fire flow bypass valve with electric actuator is provided to allow fire flow to each zone. There is addition space provided for one circulation and two booster pumps. The station also contains a circulation pump for Granger neighbourhood that is no longer used since the construction of Granger Booster, which now provides neighbourhood circulation.

There are three boilers (one for McIntyre, one for Granger, Arkell & Logan and one standby for either zone) in this pumpstation for tempering water.

The existing MDD for the station is 26.4 l/s. One duty pump can meet this demand with a capacity is 37 l/s.

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The 1990 Water and Sewer Study estimated the ultimate Maximum Day Demand for the area serviced by the Hamilton Boulevard is approximately 136 L/s at TDH of 56 m. That study proposed to install three 50 hp service pumps and one 50 hp standby pump with the eventual replacement of the existing 40 hp pumps. The extra pumps will be required to ensure adequate night filling demand for the reservoirs. The timing of the installation of the extra pumps was to be coincidental to the construction of the proposed Hillcrest "B" Reservoir (Copper Ridge Reservoir).

The operations staff has reported current long single booster run times. This is especially prevalent when Copper Ridge Reservoir is filling. It should also be noted that Copper Ridge Pumphouse's single booster capacity and Area 4 demand is greater than Hamilton Boulevard's single booster capacity. When one booster is running at Copper Ridge and one booster is running at Hamilton Boulevard, Hamilton Boulevard's Hillcrest reservoir has a net loss of water. Pumping records and reservoir operational dynamics should be more closely examined as Copper Ridge develops. Deep cycling of Copper Ridge Reservoir is currently needed for water quality but may inhibit the ability to fill Hillcrest Reservoir during times of high water demand.

Deep cycling refers to maximizing the depth between pump call and pump stop elevations within the domestic storage in a reservoir. Deep cycling results in better reservoir mixing and change over, however, deep cycling may result in noticeable system pressure fluctuations and can impede reservoir recovery during high demand periods. Deep cycling should not be allowed to operate into emergency or fire storage levels.

Standby power with enough capacity for control system requirements and ultimate maximum day demand pumping should be installed at this station. The standby generator should also be able to at least meet ultimate averaged day demand requirements with full boiler and circulation pump operation. As noted in Section 4.2.2, standby power is required to activate the electrically actuated fire flow valves at Hamilton Pumphouse.

4.2.1.7 Granger Booster Station

The Granger Booster Station is located near the intersection of Thompson Road and Hamilton Boulevard and boosts water into the Granger Neighbourhood. The design pumping capacity is 31.5 L/s at TDH of 15 m. There were some operational problems encountered after the installation of the booster station. It was difficult to find the right PRV set points at Wilson and Thompson PRV stations in order to maintain upstream pressure and provide circulation flow. At present, the Granger Booster is on year round, acting both as a boosting pump and a circulation pump. Subsequent to the commissioning of this facility the circulation pump at the Hamilton pump house was decommissioned.

Field verification of Granger Booster indicated the station inlet at 300 kPa and outlet at 428 kPa. According to the pump curve a head differential of 13.5 m should be equivalent to a flow of approximately 40 L/s.

4.2.1.8 Copper Ridge Pumphouse

The Copper Ridge Pumphouse is located on Falcon Drive near the intersection of North Star Drive and supplies water to the Copper Ridge Neighbourhood and fills the Copper Ridge Reservoir. The station was built to accommodate three booster pumps, three circulation pumps and three boilers, Currently the station has two 25 hp booster pumps, three 15 hp circulation pumps and two boilers. The capacity of each booster pump is 34 L/s at total discharge head (TDH) of 37 m. The pumps are primarily controlled by the levels in the Copper Ridge Reservoir. Station low-pressure sensors will shut down circulation pumps and activate valves to allow for bi-directional flow during a fire flow condition. The capacity of each circulation pump varies between 39 L/s to 45 L/s at TDH of 18 m to 14 m. The boilers are operated automatically to turn on when the return temperature is less than 2 ^{o}C. The 1999 Pumphouse and Lift Station Audit study recommended that the boiler operating temperature be adjusted to a slightly lower set point (potentially 1 or 1.5 ^{o}C).

4.2.1.9 Circulation pumps

The water network contains circulation pumps to maintain minimum velocities in the network to prevent freezing. These facilities are primarily a thermal component and are discussed in more detail in the thermal section of this report. Also, these facilities pose a fire flow concern. Fire flows are discussed in more detail in Section 4.2.2.

4.2.2 Distribution Network

The modeling results provide insights into the existing capacities of the larger diameter transmission lines and distribution systems. The performance of the modeled pipe networks of the existing transmission and distribution mains have been evaluated under various hydraulic conditions with respect to abnormal head loss, residual pressure and flow velocity throughout the system. They are described below.

Operating Pressures

System-wide maps of nodal pressures and flows are presented in Figures 4.1, 4.2, 4.3 and 4.4 for peak hour demand (PHD), maximum day demand (MDD), night filling demand (NFD) and average day demand (ADD) respectively. The figures are used to graphically illustrate areas of concern.

Since low demand periods result in highest system pressures, night filling demand has been reviewed to identify areas of high pressure. As indicated Section 2.8.1, 700 kPa (100 psi) was chosen as the safe high operating pressure. Since EPANET works in metres of pressure, Figure 4.3 nodal pressures are indicated in metres. The general conversion is 1.0 m water equals 9.81 kPa of pressure. Figure 4.3 indicates that Takhini East and the north end of Ponderosa Drive have pressures that are in excess of 70 metres (687 kPa) and therefore all individual services should have pressure reducing valves.

Times of high demand generally represent times of low system pressure. There were no significant areas of low pressure during maximum day demand and full pumping. During peak hour demand with no pumping, Grove Street, Ponderosa Drive, Crestview Neighbourhood and Granger Neighbourhood indicate low pressures as can be expected without localized boosters running. Southern portions of Riverdale were also indicating low pressures but they were just above the acceptable 240 kPa (40 psi) pressure requirement.

Fire Flow Pressures and System Adequacy

The modeling results have also been analyzed to provide estimates of available fire flows at critical model nodes. These quantities have been compared to required fire flows (a function of land use zoning) and deficient areas have been identified in Table 4.2.

Table 4.2 Maximum Day Demand plus Fire Flow Deficiencies

Node	Neighbourhood	Pressure Zone	Required Flow (L/s)	Available Flow (L/s)	Residual Pressure (kPa/psi)	Lowest Zone Pressure (kPa/psi)	Lowest Zone Pressure Node
3520	Downtown	Riverdale	75.3	23.4	140 / 20	225 / 32.7	2000
3540	Downtown	Riverdale	75.4	25.2	140 / 20	225 / 32.6	2000
3560	Downtown	Riverdale	75.5	29.2	140 / 20	224 / 32.5	30220
3570	Downtown	Riverdale	76.0	33.5	140 / 20	222 / 32.2	30220
3580	Downtown	Riverdale	75.6	34.2	140 / 20	222 / 32.2	30220
3720	Downtown	Riverdale	75.0	34.9	140 / 20	185 / 26.8	3730
3530	Downtown	Riverdale	76.5	37.0	170 / 24.6	140 / 20	3520
3490	Downtown	Riverdale	75	38.3	140 / 20	221 / 32.1	30220
3550	Downtown	Riverdale	77.3	39.2	148 / 21.4	140 / 20	3540
3730	Downtown	Riverdale	75	39.5	140 / 20	140 / 20.0	3720
3410	Downtown	Riverdale	75.2	39.9	140 / 20	156 / 22.6	3400
3400	Downtown	Riverdale	75.2	41.6	142 / 20.6	140 / 20	3410
3500	Downtown	Riverdale	75.1	47.4	140 / 20	219 / 31.7	30220
3420	Downtown	Riverdale	75.6	48.1	143 / 20.8	140 / 20	3410
1550	Riverdale	Riverdale	76.2	41.0	140 / 20	175 / 25.4	1560
1560	Riverdale	Riverdale	78.6	46.2	140 / 20	148 / 21.5	1550
1360	Riverdale	Riverdale	75.9	46.4	140 / 20	180 / 26.2	1350
1210	Riverdale	Riverdale	76.3	50.1	140 / 20	168 / 24.3	1200
16780	Porter Creek	Grove	75.4	48.0	140 / 20	155 / 22.5	16790
16790	Porter Creek	Grove	75.4	48.4	140 / 20	159 / 23	16780
14240	Granger	Granger	75	68.5	154 / 22	145 / 21	14250
1425	Granger	Granger	75	66.5	148 / 21	165 / 24	14240
14260	Granger	Granger	75	68.2	169 / 25	145 / 21	14250
14270	Granger	Granger	75	70.6	140 / 20	145 / 21	14250
14280	Granger	Granger	75	72.6	184 / 27	140 / 20	14250
14300	Granger	Granger	75	70.4	141 / 21	186 / 27	14250

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Table 4.2 Maximum Day Demand plus Fire Flow Deficiencies (Continued)

Node	Neighbourhood	Pressure Zone	Required Flow (L/s)	Available Flow (L/s)	Residual Pressure (kPa/psi)	Lowest Zone Pressure (kPa/psi)	Lowest Zone Pressure Node
5100	Marwell	Marwell	150 - 250	93.4	138/20	47/7	5110
5110	Marwell	Marwell	150 - 250	109.1	138/20	138/20	5100
5120	Marwell	Marwell	150 - 250	123.2	22/153	138/20	5110
5090	Marwell	Marwell	150 - 250	124.1	138/20	47/7	5080
5080	Marwell	Marwell	150 - 250	125.6	138/20	173/25	5090
5070	Marwell	Marwell	150 - 250	128.3	138/20	160/23	5060
5060	Marwell	Marwell	150 - 250	130.1	138/20	144/21	5070
5050	Marwell	Marwell	150 - 250	136.4	138/20	161/23	5060
5130	Marwell	Marwell	150 - 250	147.2	138/20	143/21	5110
5040	Marwell	Marwell	150 - 250	150.2	138/20	145/21	5050
5140	Marwell	Marwell	150 - 250	151.1	138/20	138/20	5130
5180	Marwell	Marwell	150 - 250	157.3	138/20	213/31	5170
5150	Marwell	Marwell	150 - 250	170.5	138/20	152/22	5130
5170	Marwell	Marwell	150 - 250	176.2	138/20	144/21	5180
5160	Marwell	Marwell	150 - 250	181.8	138/20	138/20	5150
5030	Marwell	Marwell	150 - 250	195.5	138/20	144/21	5050
5020	Marwell	Marwell	150 - 250	201.8	138/20	138/20	5030
5010	Marwell	Marwell	150 - 250	203.3	139/20	138/20	5030
5000	Marwell	Marwell	150 - 250	300.0	369/54	443/64	5030

Table 4.2 addresses two distinct issues: fire flows and predicted pressure residuals. The acceptable criterion for pressure residual is no less than 140 kPa (20 psi). The fire flow node does not necessarily produce the lowest pressure residual, as restricted pipe sizes or elevations may produces lower pressures elsewhere.

Field flow monitoring should be used to verify model findings and the Fire Department should be notified via system maps of any areas that are in risk of pressures dropping below 140 kPa (20 psi).

Circulation Zones

Prior to this study, previous City of Whitehorse water models were incapable of modeling circulation zones, as the models could not find a flow solution when a pump pulls water from and discharges to the same pressure zone. The fire flow results presented in Table 4.2 assumes that circulation pumps have been bypassed. This assumption can pose a concern, as Transit, Hart, Redwood and Centennial Circulation stations require manual bypass.

Fire flow from nodes both east and west of Transit Circulation station are significantly reduced when the circulation pump is in operation as flows are restricted by the 50 mm pump.

Fire flows near Hart Circulation are already slightly below minimal requirements. The circulation pump further reduces available fire flows a small amount. The 1990 study and the current study recommends construction of a new water main from Selkirk Pumphouse to south and west Riverdale. Hart circulation station may no longer be required after construction of the new watermain due to thermal flow improvements.

The node south of Centennial Circulation station provides marginal fire flows when the circulation pump is in operation but may be acceptable. The node west of the station does not provide adequate fire flows and the node north of the station appears to provide adequate fire flows during circulation pump operation.

Nodes near Redwood Circulation appear to be able to provide adequate fire flow during circulation pump operation.

The model findings need to be confirmed by field testing as modeling fire flows within circulation zones may not be accurate. Areas of concern need to be identified on Fire Department maps with instructions to contact the pumphouse on call operator to manually bypass circulation pumps.

Bypasses with a check valve may be an option to remedy fire flow concerns downstream of a circulation pump but a check valve would not improve upstream flows. The City should install fire flow valves capable of providing flows in both directions when fire flows are a concern upstream and downstream of a circulation pump.

Kopper King Circulation uses small diameter copper lines for circulation and the circulation pump operation should not affect flow results. Circulation pumps located within pumphouses should not be a concern as they either already have fire flow bypasses or fire flow nodes have adequate main sizes and multi directional feed to provide required fire flows.

Downtown

The worst location for meeting fire flows was located in Downtown's west side where fire flows of only 23 to 40 L/s were calculated. This is due to the limited interconnections of the systems and the smaller 150 mm diameter lines. It was noted that the situation worsened if the pumps at Selkirk were not functioning. A connection can be made between the 600 mm Crosstown Watermain and the 150 mm distribution main on Sixth Avenue to improve fire flows on the west end of Main Street until future looping is completed.

Watermain looping and upgrading dead end lines will improve fire flows, frost protection and eliminate free flow bleeders.

Riverdale

Southern and western portions of Riverdale also have significant fire flow deficiencies. The 1990 Water and Sewer Study recommended construction of new watermains from Selkirk Pumphouse to southeast Riverdale to remedy the situation. A similar option was modeled under the current study. The alignment modeled ran south from Selkirk Pumphouse following the existing intake lines. The modeled alignment then ran east following the Yukon Energy right of way on the south side of Riverdale. Connections were made at Boswell Crescent and Hyland Crescent.

Stantec found that not only fire flows improved but also thermal flow conditions improved throughout the neighbourhood. Marginal gains were also noted for system pressures and Riverdale reservoir filling capabilities.

Porter Creek South

Another area showing slight deficiencies was along Grove Street and along Ponderosa Drive. The fire flow residual pressures were marginal with a McIntyre Creek booster in operation and nearly zero if no McIntyre pumps were operating. Stantec modeled the area by minimizing all head losses between the reservoir and the fire nodes and residual pressures improved but were still marginal without McIntyre Pumphouse in operation. Upsizing watermains throughout Porter Creek would be very expensive and would seriously impact thermal conditions. Porter Creek Reservoir alone simply does not provide enough system head to meet fire flow requirements in the higher areas of Grove Street and Ponderosa Drive. Both booster pumps have to be running at McIntyre Creek Pumphouse.

Stantec recommends that pressure sensors be installed in the Grove Street and Ponderosa Drive areas that are directly connected to McIntyre Creek booster pump control. Even with pumps running the residuals on Grove Street and Ponderosa Drive are below the minimum standards during fire flows. McIntyre pumps should be upgraded in the near future. The area surrounding Grove Street and Ponderosa Drive is heavily treed and better than marginal fire flows should be considered.

Field fire flow monitoring at the upper end of Ponderosa Drive and on Grove Street may also be warranted to confirm model results. A fire flow test was conducted at the lower end of Ponderosa Drive as part of the water model calibration. As expected, the fire flow was good as pressures are higher at the lower end of Ponderosa Drive.

Granger

The deficient fire flow data for Granger presented in Table 4.2 assumes only one Granger booster pump in operation. Operating both Grainger boosters during a fire flow event appears to improve fire flows. The control system at Granger Booster should allow for the second pump to start upon a drop in pressure. Water modeling indicates that the pressure does not drop much at Granger Booster during a fire event in the upper area of Granger so second pump operation and fire flow capabilities should be field confirmed. If it is found that Granger booster does not start the second pump on pressure drop, a flow switch or remote pressure sensors may be required to start the second pump.

It should also be noted that fire flows are significantly reduced to the upper area of Granger if Granger booster pumps are not operating during a power fail event. Concerns should be noted on Fire Department maps as careful fire pumper operation will be required during power failures.

Construction of a supply main with PRV from Copper Ridge pressure zone to Granger booster pressure zone will also improve Granger fire flows. A pressure zone interconnection also has the benefit of providing fire flows during a power fail event. This option will not be included in the implementation plan as the option is not likely economically feasible.

It is important that water can back feed through Hamilton Pumphouse during fire flow events in Granger. Hamilton Pumphouse has electrically actuated fire flow valves that open on low pressure. Currently there is no standby power at Hamilton Pumphouse to open fire flow valves during power disruption. The City has budgeted for a standby generator but low fire flows during power failure should be clearly marked on Fire Department maps until the standby generator can be installed. If installation of a standby generator at Hamilton Pumphouse is deferred, consideration should be made for standby batteries to operate the fire flow valves.

Hillcrest

Hillcrest neighbourhood indicated slight fire flow deficiencies in some areas and is generally not a concern unless land uses change. This study recommends a water supply main with pressure reducing valve from to Grainger to Hillcrest to improve thermal conditions. The supply main should also improve Hillcrest fire flows.

Marwell

As was with the 1990 study, this study identified fire flow requirements of 150 L/s to 250 L/s for commercial / industrial lands. The fire flow requirements were calculated as per the Fire Underwriters Survey guidelines provided in the document entitled Water Supply for Public Fire Protection, 1999.

The Quest Engineering Group 2003 Marwell Water System Hydraulic and Thermal Assessment Summary calculated similar requirements for Marwell area as follows:

- Maximum requirements (1,500 sq.m. building, no sprinklers): 267 L/s
- Average requirements (750 sq.m. building, no sprinklers): 167 L/s
- Minimum requirements: 33 L/s

Stantec

The difference between Stantec's and Quest's required fire flows can be attributed to assumptions of building areas or construction materials.

The Quest report recommended upgrade of the existing 250 mm Industrial Road supply main to 400 mm from Two Mile Hill to Quartz Road and upgrading the 250 mm main to 300 mm from Quarts Road to Galena Road.

It appears that the biggest cause of inadequate fire flows in the Marwell area is too high head losses. The watermains are simply too small to meet high fire flow requirements dictated by land use and building types.

Stantec modeled a few options to improve fire flows to Marwell, including Marwell pressure reducing valve (PRV) modifications, a new low pressure supply main from Chilkoot Way, and new low and high pressure supply mains from Two Mile Booster.

Table 4.3 provides fire flow modeling results for the various Marwell fire flow improvements. Fire flow nodes on both sides of Transit Circulation station were modeled as they represented the locations with greatest head losses.

Table 4.3 Marwell Fire Flow Improvements

Improvement	Node 5080 Flow (L/s)	Node 5080 Residual (kPa)	Node 5110 Flow (L/s)	Node 5110 Residual (kPa)
Existing System (250 mm PRV @ 481 kPa / 67 psi)	129	142	142	142
Alternative 2 Existing plus increase to 350 mm PRV @ 481 kPa	126	148	140	147
Alternative 3 Existing plus increase PRV to 636 kPa (92 PSI)	150	148	168	143
Alternative 4 Replace main from TMB to Quartz (PRV @ 481 kPa)	158	145	188	142
Alternative 4 Replace mains from TMB to Galena (PRV 481 kPa)	164	143	210	145
Alternative 4 Replace mains TMB to Galena (PRV at 636 kPa)	192 (3.2m/s)	248	248 (3.6m/s)	147
Alternative 5 Supply Marwell from Chilkoot Way and Two Mile Booster low pressure side	135	145	150	143
Alternative 5 Supply Marwell from Chilkoot Way with existing TMB high pressure as alternate supply (PRV @ 481 kPa)	155	138	172	142
Alternative 5 Supply Marwell from Chilkoot Way with existing TMB high pressure as alternate supply (PRV @ 636 kPa)	161	144	180	145
Existing System with PRV bypassed (1,100 kPa during fire flow)	295 (5.4m/s)	14.4	318 (4.2m/s)	14.6

Alternative 1 – Reduce Fire Flow Requirements

An alternative to providing increased fire flows to Marwell would be to reduce fire flow requirements. One possible solution would be to consider mandatory building fire suppression systems.

The Fire Underwriters survey suggests that a building's fire flow requirements may be reduced by up to 50% if the building contains a fully supervised automatic sprinkler system with full building coverage and adequate hydrant supply. If all buildings were fully compliant it may be possible to reduce fire flow requirements for Marwell to 125 L/s assuming 50% of the 250 L/s criteria.

All new buildings or renovated buildings that do not have adequate fire flow at adjacent hydrants should be required to have internal fire suppression systems. The City may also consider implementing a Bylaw that requires existing buildings to upgrade. A grant program may be considered if it can defer Capital expenditure for water system upgrade.

Alternative 2 – Increase PRV Size

Increasing the size of the PRV did not make any difference to the fire flows in Marwell as the downstream pressure setting restricts flows through the PRV. The existing PRV settings at Marwell reduce pressures from approximately 1,150 kPa (166 psi) to 481 kPa (70 psi) at maximum day demand.

Alternative 3 – Increase PRV Pressure

The easiest and most cost effective method of modestly increasing fire flows to Marwell is to allow higher pressure flows through the PRV at Two Mile Booster during fire flow events. Under normal operation a PRV maintains a set downstream pressure by limiting flow through the valve. Since head losses can be great between the Marwell PRV at Two Mile Booster and a high demand fire flow in Marwell, the pressure will not drop enough on the immediate downstream side of the PRV to allow adequate flow through. Control can be introduced to allow higher pressures across the PRV during fire flow events; however, increasing flow across the PRV should only be carefully considered as increasing downstream pressures may cause damage to the system. If the PRV is set to 636 kPa (92 psi), as modeled, system pressures at night filling demand (NFD) are as high as 751 kPa (109 psi), so any pressure increase should only be allowed during fire flow events. System pressures are normally as high as 594 kPa (86 psi) at NFD with the current PRV setting of 481 kPa (67 psi).

Very high fire flows can be achieved by bypassing the Marwell PRV completely during a fire event but system pressures and velocities will likely cause damage.

Alternative 4 – Replace Watermain from Two Mile Booster to Galena Road

Fire flows are greatly improved with upgrade of the existing 250 mm Industrial Road supply main to 400 mm from Two Mile Hill to Quartz Road and upgrading the 250 mm main to 300 mm from Quarts Road to Galena Road as was identified in the Quest report.

Increasing pressure across the Marwell PRV during a fire flow event in conjunction with new watermains provides additional fire flow improvement. More detailed modeling is recommended to determine PRV setting and optimum main sizes.

A modified option to upgrading from Industrial Road and Quarts Road to Galena along Industrial Road may be to provide a new watermain loop from Industrial Road and Quartz Road east. The new loop can pass through the Government of Yukon Grader Station property and the North of 60 property adjacent to the Yukon River. This option will provide servicing to future development lands, improve thermal flow conditions and provide additional watermain supply capabilities to the future Riverdale Expansion area. A secondary feed to Riverdale Expansion area may be incorporated into a future bridge crossing of the Yukon River near the end of Industrial Road.

Alternative 5 – Supply Marwell from Low Pressure Zone

Supplying Marwell from the low pressure mains on Industrial Road and from Chilkoot Way does not provide adequate fire flows to Marwell without upgrading all watermains to 300 mm minimum. Slight improvements are gained over existing conditions if Marwell is supplied from Chilkoot way and if the existing Marwell PRV and the Industrial Road main are used as an alternate supply during fire flow.

Alternative 6 - Increase Watermains to 300 mm Minimum

Fire flows in excess of 250 L/s can be achieved if watermains on Quartz Road, Copper Road, Tlingit Road and Galena Road loop were upgraded to 300 mm minimum in addition to upgrading from Two Mile Booster to Quartz Road. Construction costs would however be prohibitive. Circulation station upgrades may also be required at Transit.

Alternative 7 – Replace Watermains from Two Mile Booster to Transit

Alternative 7 is similar to Alternative 4 but also includes an additional 300 mm water main along Calcite, through private property with a connection on Tlingit Road near the Transit building. This option will provide required fire flows in excess of 250 L/s but may be cost prohibitive and requires development through private lands. Alternative 7 however would be attractive if the private lands were ever developed and require additional servicing. Circulation station upgrades may also be required at Transit.

Marwell Fire Flow Improvement Recommendation

Alternatives 1 and 3 consisting of mandatory sprinkler systems and increased PRV pressure may be the most cost effective method to balance fire flow requirements and fire flow capabilities in Marwell.

One draw back to only implementing Alternatives 1 and 3 is that this solution may not provide enough fire flow for future larger buildings and watermains may require upgrade anyway to service future development lands. Further, increasing pressures across Marwell PRV requires careful consideration in order to avoid system over pressuring.

Alternatives 6 and 7 consisting of watermain replacement were the only alternatives that provided flows that meet the 250 L/s fire flow requirements but may be cost prohibitive.

The final solution to Marwell fire flows will be dependent upon development. If minimal development is anticipated and some risk can be assumed, Alternative 1 and 3 may be the best solution. If development were anticipated Alternative 1 and 7 would be desirable. A more detailed analysis of building fire flow requirements, potential sprinkler upgrades and a cost benefit analysis are required.

A possible future secondary feed to Riverdale Expansion area via a new bridge near the end of Industrial Road may also be considered when sizing any new mains in Marwell; however, Riverdale Expansion will likely not be required within the next 20 years.

Slight fire flow improvements can also be achieved if a 250 mm bypass was provided outside Transit Circulation station. The mains are currently restricted to 150 mm lines running to the facility. As fire flows are a concern when the circulation pumps are in operation a fire bypass station can be provided on Tlingit Road. Consideration should be made for a surface level or shallow burry station as ground water elevations are shallow.

The Quest report also posed the question of the whether or not the Fire Department had equipment and adequate fire hydrants spacing to use the required fire flows. The existing fire equipment is rated at the following flows:

- 2 pumpers @ 64 L/s (840 IGPM) each
- 1 pumper @ 78 L/s (1050 IGPM)
- 1 pumper @ 95 L/s (1250 IGPM)
- 2 portable pumps @ 47 L/s (625 IGPM) each

Current Marwell fire flow capabilities are adequate for one pumper truck operation; however, during large fire events the Fire Department may wish to connect fire fighting equipment to more than one hydrant. Hydrant spacing in the vicinity of high fire flow demand buildings should be reviewed, as hydrant infilling may be required.

In accordance with the Fire Underwriters Survey, a hydrant should be provided for every 9,500 square meters of land area when fire flows of 16,000 litres per minute (267 L/s) are required; however, the City may wish to adopt their own standards when multiple hydrants are required to service one building. For example, enough hydrants should be available within a specified distance from the building to connect enough equipment to meet calculated fire flow requirements.

4.2.3 Water Storage Reservoirs

Table 4.4 details the water storage requirements based on the criteria outlined in Section 2.10.

Each storage facility has been examined to determine the total required storage and existing surplus / deficit in storage. Effective Capacity does not account for operational level and assumes

Effective Capacity accounts for dead storage resulting from piping configuration and vortexing. Votexing is discussed in detail in Section 3.5.1. The Effective Capacity does not account for operational levels and assumes maximum capacity is to overflow level.

Table 4.4 Adequacy of the Existing Storage Facilities

Reservoirs	**Fire** (m^3)	**Supply** (m^3)	**Peak** (m^3)	**Emergency** (m^3)	**Total Required** (m^3)	**Total Existing Capacity** (m^3)	**Effective Capacity** (m^3)	**Surplus / Deficit** (m^3)
Copper Ridge	3,150	307	153	46	3,656	7,285	6,449	2,793
Hillcrest	3,150	826	413	124	4,513	6,058	6,058	1,545
Porter Creek	3,150	2,985	1,493	448	8,076	5,716	5,190	-2,886
Riverdale	3,150	6,511	3,255	977	13,892	13,406	12,076	-1,816
Valleyview	3,150	4,011	2,006	602	9,768	5,508	5,032	-4,736
					39,906		**34,805**	**-5,101**

Reservoir evaluation assumes that MDD capacity will be met by the supply system and additional storage is not required for 5-day and 7-day maximum demands.

This assessment indicates that the Valleyview, Porter Creek and Riverdale reservoirs are undersized. The Riverdale reservoir is undersized but due to the proximity of the Selkirk pumpstation it is not a large concern but consideration needs to be made for upgrading of Selkirk Pumphouse to ensure the station meets future maximum day demand. If the vortexing potential is addressed or determined not to be a concern, the reservoir will be adequately sized.

The areas within the Valleyview reservoir service area highly rely on the Two Mile Hill Booster Station to provide adequate flows. If for any reason, the Two Mile Hill Booster Station were out of service there would only be enough storage to last for approximately 8 hours under average day demands.

The ability of the distribution system to cascade flows down from the higher reservoirs (Copper Ridge and Hillcrest) in an emergency situation lessen the criticality of the reservoir expansion but still does not make up the existing shortfall in volume. Copper Ridge Pumphouse, Hamiliton Pumphouse and Two Mile Booster have manual cascade valves that can be operated upon low reservoir level alarms in lower zones. Automation is not recommended, as operator intervention should be required anytime a reservoir requires cascading.

Since Copper Ridge neighbourhood is still in development water demands are low and Copper Ridge Reservoir is maintained at half capacity during summer months for water quality. In order to increase thermal mass the reservoir level is increased in winter and deep cycled. A cascade valve in Copper Ridge Pumphouse is also left partially open to cycle reservoir water more quickly.

The Porter Creek reservoir is the most critical in terms of expansion. The remote location of this reservoir makes expansion a priority. Further, Porter Creek is a single cell reservoir that cannot be easily shut down for maintenance.

The computer model has been run with no pumps in operation to examine the adequacy of the reservoirs in terms of hydraulic capacity. The model run simulated a peak hour demand (PHD) with all supply pumps off. Each reservoirs available volume was determined by subtracting the total fire storage volume from the calculated volumes. Table 4.5 indicates the time each reservoir can safely operate at PHD without booster re-supply.

The implementation of SCADA has greatly improved the ability to adjust reservoir set points at Riverdale, Valleyview and Copper Ridge Reservoirs. Porter Creek and Hillcrest Reservoir still need to be incorporated into the SCADA system. Reservoir dynamics is very complicated as system demand, reservoir turnover rates and chlorine residuals need to be taken into account. Any automated control logic would be complicated and prone to error. Operator intervention is recommended to make adjustments to reservoir set points as system dynamics dictates. SCADA can provide very good data trending to aid in adjustment of reservoir level set points.

Table 4.5 Adequacy of Hydraulic Capacity of Reservoirs

Facility	Peak Hour Flow (L/s)	Volume available (m^3)	Time (hrs)
Copper Ridge	10.7	3299	85.6
Hillcrest	29	2908	27.9
Porter Creek	102	2040	5.6
Riverdale	237	8926	10.5
Valleyview	84.5	1882	6.2

Existing overall reservoir dynamics was analysed for inflows and outflows during various run conditions. Table 4.6 indicates overall inflow and outflow dynamics.

Table 4.6 Reservoir Dynamics[15]

Reservoir	NFD Full Pumping Inflow /Outflow (L/s)	ADD Typical Pumping Inflow /Outflow (L/s)	ADD Full Pumping Inflow /Outflow (L/s)	MDD Full Pumping Inflow /Outflow (L/s)
Riverdale	74.27	-89.89	49.03	-15.73
Valleyview	52.78	92.84	40.03	8.23
Porter Creek	66.81	37.27	54.98	24.42
Hillcrest	-4.71	-9.64	-8.59	-18.05
Copper Ridge	37.09	-3.57	35.69	32.16
System Balance	266.24	27.01	171.01	31.03

The requirement for reservoir filling at night filling demand (NFD) is that the reservoirs fill at 1.8 to 2.0 times ADD filling. The criteria was found to be too basic in a complex reservoir and pumping system. Alternately, if all booster stations are capable of supplying maximum day demand (MDD), night filling is only required to top up reservoirs that were at the bottom of their boost cycles at the end of the day. As

[15] Full pumping in Table 4.6 is defined as pumping conditions required to meet MDD demand. Typical pumping only has one booster running at Selkirk Pumphouse, Two Mile Booster Station and McIntyre Creek Pumphouse. Hamilton Boulevard Pumphouse and Copper Ridge Pumphouse are not boosting.

indicated in Table 4.1, although some are marginal, all stations are currently capable of supplying MDD demand.

Table 4.6 indicates that all reservoirs have good night filling characteristics even under MDD upstream pumping except Hillcrest Reservoir. This should not be a concern. Hillcrest Reservoir only has a net loss of water when Copper Ridge Pumphouse is boosting water. Since demands are far below design capacity at Copper Ridge Pumphouse long boost times are not expected unless Copper Ridge Reservoir is deep cycled. Deep cycling of any reservoir during times of high demand is not recommended.

Table 4.6 also indicates that long single booster run times can be expected at Selkirk Pumphouse during ADD if only one booster is in operation. This is currently critical during times of high Schwatka Lake turbidity when only one booster can be operated to increase well water blending.

4.3 THERMAL EVALUATION

4.3.1 General

The calibrated thermal model was applied to assess the thermal performance of the existing water distribution system under the design winter condition of 1971 – 1972, which provided a design ground temperature of –5 °C. The following hydraulic loading conditions have been modeled:

- Winter Design Condition with Winter Night Filling Demand - No facilities on
- Winter Condition with Winter Night Filling Demand – All facilities on

The simulation runs have been carried out by assuming that all the circulation pumps and bleeders are not working. This condition indicates which of the circulation pumps or bleeders need to be on and which of could be abandoned. This scenario also depicts the influence of only Selkirk Pump house and water wells on the entire system. The second simulation with the network in full winter operation mode outlines the influence frost protection stations on the entire system.

Thermal modeling has indicated that the night filling demand condition is the most critical condition throughout the City because water flow through the system is at its slowest. Night filling demand was chosen to be 0.3 times ADD as opposed to 0.6 times ADD (0.3 times MDD) used for reservoir filling analysis. The rational is that winter night filling demands may be expected to be lower than summer night filling demands and represents a worst-case scenario. The following summarizes the findings of this thermal evaluation.

Figure 4.6 and Figure 4.7 presents nodal temperatures found throughout the network for these two design conditions.

There are six significant areas that indicate deficient thermal conditions. These are:

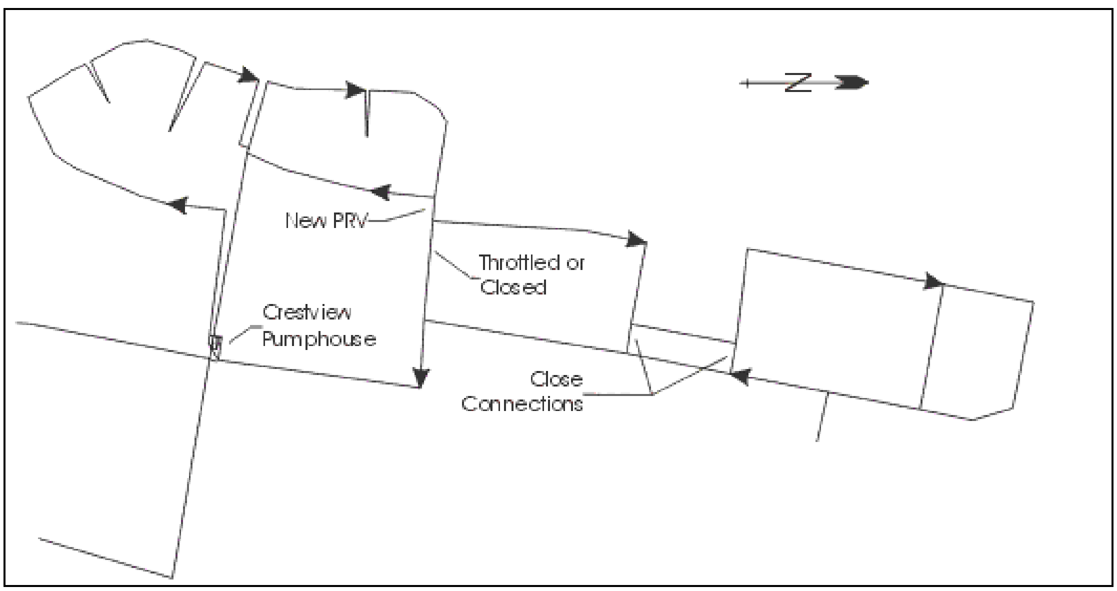
1) Riverdale - outer extremities of the system to the south and east.
2) Hart Circulation Station
3) Downtown west side (bleeders)
4) Airport
5) Hillcrest Neighbourhood
6) Range Road North (trailer parks and Crow Street)

Improvement recommendations for these deficiencies are discussed in Section 4.3.2.

In an attempt to continue to reduce water consumption the lower area in Crestview was examined for a method to eliminate Kathleen and Klukshu bleeders. The 1999 pumpstation audit suggested that alternative means should be investigated.

Currently water flows into the lower Crestview area via a PRV within the Crestview booster station. With minimal demands in the area, two bleeders are required to maintain flow and prevent freezing of the mains. Upon review of the distribution system a potential alternative was conceived. A control valve (PRV) can be constructed in a vault located at the intersection of Klukshu and Rainbow Roads. A normally closed valve currently separates the two loops. The PRV would cascade warmer water from the upper zone into the lower zone and flow will travel around the loop and discharge at the suction side of the Crestview booster station.

The following figure shows the concept.



A more detailed analysis will be required to fully assess thermal and fire flow impacts.

In order to maintain flows and boost pressure in the Crestview upper area, booster pumps and circulation pumps are used in series. It may be possible to replace the pumps with duty and standby pump capable of providing both pressure and flow to the neighbourhood. The City may want to review pump configuration at the time of proposed looping design.

4.3.2 Evaluation of Thermal Improvement Alternatives

Under this task potential freeze prevention improvements have been identified under existing demands and design winter conditions. The following are some of the potential measures that have been evaluated to prevent potential freeze ups:

- Modification of the distribution system
- Modification and/or addition of circulation facilities
- Providing direct heat, i.e. localized boiler
- Modification and/or addition of thermostatically controlled bleeders

The most effective and beneficial method of providing increased thermal protection throughout the City is to improve source temperatures by increasing groundwater supply capacity or using heat from a geothermal heat pump. Geothermal heat pumps to warm reservoir or circulation loops may also be considered but warrant further investigation beyond the scope of this study.

Figure 4.8 represents network conditions with proposed thermal improvements. The most likely feasible alternatives for different areas are discussed below.

Downtown – Westside bleeders

Alternatives to mitigate low temperatures in Downtown are presented below.

Alternative 1 - increase the bleeder flow rate thereby ensuring freeze protection at the extremities. Though effective this alternative does not address long-term water conservation initiatives.

Alternative 2 – install TCB at key locations. This alternative potentially reduces the total volume of water required by Alternative 1.

Alternative 3 – introduce looping and circulation on the west extents of the Downtown distribution system. Distribution piping installation of 700 m of 200 mm minimum would be required. A circulation station could be introduced at the end of Strickland Street. The station could utilize the three mains to the south to draw warmer water.

Alternative 3 is recommended.

Hillcrest

Alternatives to mitigate low temperatures in Hillcrest are presented below.

Alternative 1 – introduce a circulation station near Summit Drive. This will force the water to circulate around the neighbourhood. Possibly include a boiler as a source of heat to the system.

Alternative 2 – introduction of TCB in strategic locations. Though effective this would impact long-term water conservation initiatives.

Alternative 3 – Install a supply line to Hamilton Boulevard Pumphouse. The flow can be controlled at 15-20 l/s to draw warmer water from Hamilton Boulevard. The line could operate even with the booster station off, as the circulation pump for McIntyre neighbourhood will induce a flow around the loop. The supply line would have the added benefit of improving marginal fire flows in Hillcrest.

Alternative 3 is recommended.

Airport

Alternatives to mitigate low temperatures at the Airport are presented below.

Alternative 1 – construct watermain looping and possibly introduce a circulation station. This solution would not be cost effective unless a system upgrade is already planned for the area.

Alternative 2 – introduction of TCB in strategic locations. Alternative 2 is recommended.

Takhini North

Takhini North is currently being studied for an area development scheme. Thermal requirements should be addressed during this work.

Alternative 1 – introduction of TCB in strategic locations

Alternative 2 – Reconstruct Ortona Pressure Reducing Station to a circulation station. The distribution system can be designed to only have Elvin's Pressure Reducing Station supply water to Takhini North under normal operating conditions. The proposed Ortona Circulation Station could pull water through the neighbourhood and discharge flow to the McIntyre Creek Pumphouse supply line. A fire flow valve should also be installed to reverse flow if required.

Alternative 3 – Eliminate Ortona and Elvin's Pressure Reducing Stations and install residential pressure reducing valves if required. Alternative 3 improves flow through the neighbourhood while providing increased supply to the McIntyre Creek supply lines.

Alternative 3 was provided by Quest Engineering. Quest Engineering was a Sub-Consultant on the Takhini North Planning and Pre-Design study by Inukshuk Planning. At the time of writing of this report, the planning was not yet complete. Quest Engineering indicated that Alternative 3 was the most likely recommendation.

Riverdale

Alternative 1 – increase heat source at Selkirk by means of increased ground water supply or introduction of boilers.

Alternative 2 – introduce circulation in the outer fringe areas. This entails the construction of two circulation stations.

Alternative 3 – introduction of TCB in strategic locations.

Alternative 4 – install alarmed temperature monitoring stations and bleed water manually if required. For instance a thermister can be installed on the water main near Hyland Lift. Hyland lift can be used to provide thermister alarming.

Alternative 1 is preferred and should be done in conjunction with Alternative 4. Alternative 4 can be implemented in the short term. Since the elimination of free flow bleeders in Riverdale there have been no reported frozen watermains, therefore bleeding may only be required under worst conditions.

Hart Circulation Station (Riverdale)

Hart circulates water around two loops at a rate of 15 L/s. With the proper source of heat this station should function effectively. From the thermal assessment the source of heat is the main problem in this area. While the circulation pumps moves the water the low demand in this area does not introduce sufficient warmer water to add heat to the system. Some of the potential solutions under the previous Riverdale project address this heat source problem.

New watermains from Selkirk Pumphouse to south and east Riverdale was recommended by the 1990 study and the current study to remedy low fire flows. These new watermains also provide thermal benifits by increasing flow throughout the neighbourhood. If these watermains were constructed Hart Circulation station could likely be abandoned.

4.4 SERVICING NEW DEVELOPMENT AREAS

4.4.1 Lower Porter Creek

In the 1990 study two alternatives were examined to service Lower Porter Creek Bench.

Alternative 1 - Porter Creek reservoir via Wann Road

Alternative 2 - Valleyview Reservoir via Range Road.

A report entitled Lower Porter Creek concept plan, 1994 utilized both these concepts for servicing the area.

A third alternative, a new reservoir site was considered as part of this study upon request from the City.

With an average ground elevation of 680 m, the Lower Porter Creek Bench will require a HGL in the range of 740 m to 760 m. A new reservoir would have to be built on a contour of approximately 750 m. A review of the topography in the area would suggest that the closest location is the Porter Creek Reservoir. This would not be desirable due to the additional cost in pumping. A second location could be on the slopes west of Takhini but this would provide no benefit over using Valleyview Reservoir.

The 1990 study concluded that Alternative 2 would be the most desirable. Water mains within Porter Creek have been upgraded and a sufficient transmission main has been constructed along Range Road currently terminating at the Northlands Trailer Park.

Water supply and storage for the lower bench is recommended primarily from the Valleyview Reservoir with a secondary feed from Porter Creek.

Modifications to the Range Road PRV will be required as flows increase. The ultimate demand for Lower Porter Creek is estimated at 44.6 L/s at average day demand and 89.1 L/s at maximum day demand.

A thermal and circulation system similar to that in Copper Ridge will be required. The exact layout will highly depend on the layout and staging of the neighbourhood.

4.4.2 Porter Creek Extension

The Porter Creek Extension is located adjacent to Mountainview Drive immediately south of Porter Creek and has elevations in the range of 700 to 725m. The area can be serviced off the exiting mains in Porter Creek with circulation considerations. The area can be serviced from a 300 mm water main located on Mountain View Drive / Hickory Street.

With a peak HGL of 775 m, pressure reduction may be required. Pressure may be more effectively controlled via individual service lines depending on the extent of the high-pressure area. Further, controlling the pressure at each lot may make the circulation system easier to design.

4.4.3 Beyond Copper Ridge

Beyond Copper Ridge was referred to as McLean Lake in the 1990 study. A servicing study was performed for this area in 1999 in a report entitled Beyond Copper Ridge Feasibility Study. The area contains up to 355 ha available for new development. Based on a range of "appropriate' urban residential densities the area may accommodate up to 2027 new residential units. There will also be the accompanying commercial, institutional and recreational facilities.

Water supply alternatives to feed the area included the following:

Alternative 1 - A new "twinned" transmission main following the existing alignment of Hamilton Boulevard.

Alternative 2 - An independent supply line and pump station from Schwatka Lake

Alternative 3 - Independent ground water wells

Alternative 4 - A new transmission main along the Alaska Highway from Two Mile Hill to Robert Service Way. The Hillcrest / Airport water supply main has recently been twinned with a new 500 mm line. Since there are likely pressure limitations due to topography a new pumphouse may be required. This option also provides servicing for commercial development along the Alaska Highway.

A possible line from Hamilton Pumphouse to Hillcrest for thermal improvements may provide backup supply or a secondary connection from Copper Ridge Reservoir to Beyond Copper Ridge may be considered for supply security.

Alternative 1 was chosen for the ultimate analysis in order to assess the impact of Beyond Copper Ridge on existing systems but more detailed study will be required to determine the best servicing alternative.

4.4.4 Tank Farm Expansion

The old Whitepass Tank Farm is a potential infill area between the Valleyview Neighbourhood and the Hillcrest Neighbourhood.

This area can be serviced from the Valleyview Reservoir in Zone 2. A connection to the new airport transmission main with secondary connection to the transmission main on Hamilton Boulevard can be made. A detailed design including a thermal analysis will be required once the layout is determined.

4.4.5 Riverdale Expansion

The Riverdale Expansion can be serviced with the installation of a 300 mm supply line from the 350 mm Riverdale Reservoir line. This neighbourhood would become part of Area 1. As was discussed in Section 4.2.3 on storage, the capacity of Riverdale Reservoir has to be considered. A secondary feed would ultimately be required. A secondary feed can be provided by a connection to the Marwell distribution system via a future bridge at the end of Industrial Road. Alternately, water wells can developed in the Riverdale Expansion Area.

Circulation systems will be required depending on the layout of the neighbourhood.

5.0 Modeling Ultimate Water Supply System

5.1.1 General

The EPANET computer model has been expanded to examine the behaviour of the future infrastructure networks required to supply water to planned expansion areas.

Various servicing alternatives have been examined. These improvements have been tested as to their benefit, with non-functional improvements being discarded and promising alternatives being the subject of further development. The final step in concept development is the preparation of order of magnitude cost estimates for each proven idea and ultimately develop a recommended infrastructure upgrading program. In order to accomplish this, the following activities have been carried out.

Based on the findings of the modeling results of the future water system, various feasible upgrading alternatives have been investigated including existing water main upgrades, additional pump stations and reservoir requirements, PRV requirements etc. In developing various alternatives for the water distribution system, the following key parameters have been taken into account:

- Implication of reduction of water usage
- Adequate capacity
- Water supply pressure
- Fire flows
- Circulation of water distribution for frost protection
- Blending of groundwater with surface water to avoid heating supply and improving quality
- Ability of the system (reservoirs, pump stations, pipelines and pressure zone arrangements) to provide reliable service for the City's long term plan
- Economic feasibility

The updated computer models have been applied to assess the hydraulic performance of the various alternatives. A cost benefit analysis has been carried out for each alternative.

5.2 DESIGN CRITERIA

The following table presents the design parameters used in the design of the future water distribution system. They are in accordance with the City of Whitehorse Design Standards.

Table 5.1 Design Parameters

Variable	Value
Population Density	3.0 people/dwelling (min)
Multi-Family Residential	80 people/hectare
Residential Average Day Demand (ADD) Downtown and Riverdale Other Areas	 600 Lpcd 500 Lpcd
Non Residential Rate - Downtown & Riverdale Non Residential Rate - Others	30 m^3 /day/hectare 10 m^3 /day/hectare
Maximum Day Demand (MDD)	2 X ADD
Peak Hour Demand	3 X ADD
Minimum Ground Level Pressure at peak flow	280 kPa (40 psi)
Minimum Ground Level Pressure at MDD plus Fire flow	140 kPa (20 psi)
Maximum ground level pressure to existing services	550 kPa (80 psi)
Maximum ground level distribution system operating pressure	700 kPa (100 psi)
Maximum velocity	3 m/s
Minimum velocity	0.15 m/s
Hazen-Williams roughness coefficient	120
Minimum depth of bury – uninsulated	3.0
Reservoir size	As per Section 4.2

Table 5.2 provides a review of ultimate build out capacity system demands.

Table 5.2 Ultimate Flows by Area

Area	Served Pop	Composite Residential Demand	Composite Residential ADD	Non Residential Area	Non Residential Rate	Non Residential ADD	Total ADD	Total MDD
		lpcd	l/s	ha	m3/ha/day	l/s	l/s	l/s
Area 1								
Riverdale	5411	600	37.6	30.8	30	10.7	48.3	96.5
Riverdale Expansion	1500	500	8.7	10	10	1.2	9.8	19.7
Downtown	1250	600	15.6	63.5	30	22.0	37.7	75.3
Hospital	40	600	0.3	19	30	6.6	6.9	13.8
Area 1 Totals	9201		62.2	123.3		40.5	102.7	205.3
Area 2								
Marwell	80	500	0.5	105.5	10	12.2	12.7	25.3
Takhini	1163	500	6.7	84	10	9.7	16.5	32.9
Hillcrest	697	500	4.0	9.4	10	1.1	5.1	10.2
Valleyview	168	500	1.0	0	10	0.0	1.0	1.9
McIntyre	1605	500	9.3	8.6	10	1.0	10.3	20.6
Trailer Parks	1608	500	9.3	0	10	0.0	9.3	18.6
Lower Porter Creek	7200	500	41.7	25	10	2.9	44.6	89.1
White Pass Tank Farm	1835	500	10.6	10	10	1.2	11.8	23.6
Airport	0	500	0.0	61.8	10	7.2	7.2	14.3
Area 2 Totals	14356		83.1	304.3		35.2	118.3	236.6
Area 3								
Porter Creek	4649	500	26.9	63	10	7.3	34.2	68.4
Crestview	799	500	4.6	0	10	0.0	4.6	9.2
Kopper King	322	500	1.9	19	10	2.2	4.1	8.1
Porter Creek Extension	1416	500	8.2	0	10	0.0	8.2	16.4
Area 3 Totals	7186		41.6	82.0		9.5	51.1	102.2
Area 4								
Granger	1046	500	6.1	7.5	10	0.9	6.9	13.8
Arkell	579	500	3.4	0	10	0.0	3.4	6.7
Logan	451	500	2.6	0	10	0.0	2.6	5.2
Area 4 Totals	2076		12.0	7.5		0.9	12.9	25.8
Area 5								
Copper Ridge	3132	500	18.1	14.5	10	1.7	19.8	39.6
Beyond Copper Ridge	6000	500	34.7		10	0.0	34.7	69.4
Area 5 Totals	9132		52.8	14.5		1.7	54.5	109.1
Ultimate System Totals	41951		251.7	531.6		87.8	339.4	678.9

5.2.1 Results of Ultimate Water Supply System Analysis

5.2.1.1 Water Supply

Under the ultimate build out capacity loading condition an average day demand (ADD) of 339 L/s and a maximum day demand (MDD) of 677 L/s are estimated to be required by the City. The proposed water treatment plant and Selkirk Pumphouse will ultimately be required to provide enough treated and sufficiently warmed water to meet MDD.

Upgrade phasing will be dependent on the rate of population growth and the effectiveness of any water consumption reduction program.

The City should continue to pursue expansion of groundwater supplies, however, with the addition of the twinned supply line, sufficient capacity will exist to convey flows from Schwatka Lake to Selkirk Pumphouse if required. A small portion of the supply line has been twinned in 1999 during the construction of the supply line siphon at the Yukon Energy Dam.

The development of a treatment facility to ensure security of source will be required in the short term. A staged implementation plan should be adopted to accommodate the existing and future needs for this facility.

5.2.1.2 Thermal Modeling of Ultimate Water System

The thermal modeling parameters have been incorporated for the new areas and the hydraulic modeling results have been utilized for assessing the thermal performance of the future water system. Detailed configuration for the new areas has not been performed as part of this study. Thermal protection measures will likely be required within new development areas. The thermal modeling has been carried out for the average day demand and night filling demand.

Expansion of the heat source at Selkirk Pumphouse is required. Ultimately the provision of heat to a total of 600 L/s at 3 degrees will be required to meet winter demands. This is equivalent to a power input of approximately 10,000 kW. Treatment efficiency may also be obtained from warming of the supply water.

Tempering water with boilers would be very expensive in comparison to operating ground water wells.

5.3 HYDRAULIC EVALUATION

5.3.1 Pumping Facilities

Over time replacement of pumps will be required in all of the major pump stations. Consideration to the ultimate size must be made to ensure future performance. Typically an upgrade will be staged in order to accommodate the 10-year and ultimate build out capacity flow projections. Table 5.3 is a recap of the existing pumping requirements for the service areas of the respected pump stations.

Table 5.3 Existing Pumping Requirements[16]

Pump Station	Demand		Capacity		
	ADD (L/s)	MDD (L/s)	Pumps Required @ MDD	MDD Capacity (L/s)	Head (m)
Selkirk	153.92	307.8	2 of 2 duty	341	30.0
2 Mile Hill	75.83	151.7	1 of 3 duty	198	85.6
Mc Intyre	34.14	68.3	2 of 2 duty	89.17	37.11
Crestview	3.70	7.4	1 of 1 duty	7.4	28.0
Hamilton	13.21	26.4	1 of 1 duty	40.5	43.3
Copper Ridge	3.57	7.1	1 of 1 duty	39.3	32.0

The ultimate build out capacity demand condition is depicted in Table 5.4.

Table 5.4 Ultimate System Demand Requirements

Pump Station	ADD (L/s)	MDD (L/s)
Selkirk	339.4	678.9
Two Mile Hill	236.7	473.6
Mc Intyre	51.1	102.2
Crestview	3.70	7.4
Hamilton	67.4	134.9
Copper Ridge	54.5	109.1

5.3.1.1 Selkirk Pumphouse

The current pumps at Selkirk Pumphouse will not support any significant growth within the city without using the standby pump to meet maximum day demands. Table 2.5 indicates that upgrading at population increase of 4,075 people (25.9 l/s) occurs in year 10 of a high growth rate scenario. The current MDD for Selkirk Pumphouse is 307.8 L/s and the capacity of Selkirk Pumphouse with two (2) booster running is 341 L/s. It is recommended that Selkirk Pumphouse be replaced during

[16] Table 5.3 does not recognise standby pumps as the purpose of a standby pump is to provide backup for maintenance or failure and should not be relied upon for capacity

water treatment plant construction. Two 150 hp duty pumps and one standby will be required to meet near future demands.

Ultimate flow requirements will be 678.9 L/s. A total of three (3) 150 hp duty pumps and one standby would be required to meet this demand. Consideration should be made for a fifth booster slot during booster station design in order to meet any unforeseen capacity requirements.

5.3.1.2 Two Mile Hill Booster Station

Two Mile Hill booster station performs adequately under ultimate flow estimates with the addition of the fifth pump (total of 4 duty pumps and one backup). Ultimate flows of 473.6 L/s are estimated which is slightly less than the original design of 493 L/s.

5.3.1.3 McIntyre Creek Booster Station

The current pumps at McIntyre will be adequate to a maximum day demand increase of 21 L/s. However fire flow analysis suggests upgrades in order to accommodate fire flows along Ponderosa Drive and Grove Street. Upgrading should be considered within the next two years. Ultimate flow requirements will be approximately 100 L/s at a head of 50 m.

5.3.1.4 Crestview Pumphouse

No ultimate change required, however, it may be beneficial to replace existing boosters and circulation pumps with a duty and backup capable of producing pressure and flow requirements.

5.3.1.5 Ponderosa and Grove Street Booster Stations

No ultimate change, however, there is concern with fire flows that can be address through booster upgrade and control at McIntyre Creek Pumphouse. New Pressure sensors at Pondersa Drive and Grove Street should be incorporated to ensure McIntyre Creek booster pumps are operating during a fire flow.

5.3.1.6 Hamilton Boulevard Pumphouse

Hamilton Boulevard Pumphouse capacity should be adequate to a population increase of 1,000 people (14 L/s @ MDD) with the stations one 40 hp duty pump in operation. Beyond a 1,000 person increase in population, 2 duty pumps and one stand by will be required. Installation of a three (3) 50 hp pumps is recommended. The station has an existing empty slot to accommodate the third pump. Significant population increases in Copper Ridge have already occurred since populations were established for this study.

Ultimately pumps capable of producing 147 L/s at a TDH of 77 m will be required to supply Beyond Copper Ridge. Due to the age of the station complete replacement will likely be required by the time Beyond Copper Ridge is developed. Improvement of the transmission network described in Section 5.3.2 can reduce the TDH requirement to 51 m.

5.3.1.7 Copper Ridge Pumphouse

This facility should be adequate to a population increase of 4,500 people (30 L/s). This is well beyond the build out population of the Copper Ridge area. At full capacity, including Beyond Copper Ridge, the station has been simulated at 74 L/s at a TDH of 34 m. Ultimately pumps capable of producing 118.5 L/s at a TDH of 36 m would be required.

5.3.2 Distribution Network

Several upgrades are required to accommodate the ultimate build out capacity demands in the system.

The upgrades are as follows:

1) Twin pipe along Alaska Highway from Elvins to Hamilton Boulevard with 270m of 400 mm diameter water transmission main.
2) Twin pipe along Hamilton Boulevard east of the Hamilton Boulevard Booster station with 900m of 300 mm diameter water transmission main.
3) Twin pipe along Hamilton Boulevard southwest of the booster station to Falcon Drive with 570 m of 300 mm diameter water transmission main.
4) Twin pipe along Hamilton Boulevard from Falcon Drive to the Granger booster station with a 680m of 300 mm diameter water transmission main.

The 1990 study proposed reversing circulation flows through McIntyre and Granger neighbourhood to increase supply for Beyond Copper Ridge. Reversing circulation flows through McIntyre neighbourhood is still possible to improve supply to Hamilton Pumphouse but will require modifications at the pumphouse. Reversing flow though Granger neighourhood to increase supply upstream of Hamilton Pumphouse will be more difficult since the construction of Granger Booster. A new booster would be required near Thompson or Wilson PRV stations. Granger Booster would have to be replaced with a pressure reducing station. The PRV station near the new booster may be abandoned.

The 1990 Water and Sewer Study identified that replacement of watermain above McIntyre Creek Booster Station. This was required in order to ensure MDD and NFD of the Porter Creek Reservoir.

Supply line modifications at McIntyre Creek Pumphouse by the City improved MDD flows since the 1990 Study. The McIntyre Creek Pumphouse currently has a 200 mm and a 250 mm discharge line. Stantec determined that even under ultimate build out capacity conditions with upgraded pumps at McIntyre Creek Pumphouse running at peak hour demand (PHD) the watermain flow characteristics were adequate. Head losses were high but velocities of 1.5 L/s were acceptable. A fire flow conditions at MDD near Grove Street and Ponderosa Drive produced slightly less velocities than normal (PHD). Generally the maximum acceptable system velocity is 3.0 m/s.

Under Ultimate MDD conditions with only existing planned watermain upgrades on Wann Road the Porter Creek reservoir had a net gain of 2 L/s which was considered marginal but acceptable. A significant increase in reservoir filling rates was not achieved by modeling a larger watermain from McIntyre Creek to Grove Street and 11 Avenue under Ultimate MDD conditions. Modeling larger supply mains from McIntyre Creek to Grove Street and 14 Avenue increased reservoir filling by about 30 L/s. Watermain upgrading from McIntyre Creek to 14 Avenue and Grove Street was not included in the Implantation Plan as the benefits were marginal and not considered necessary. Construction costs would also be high. Increasing the size of the McIntyre Creek discharge main also had a negative effect on thermal conditions in Porter Creek.

5.3.3 Water Storage Reservoirs

The existing condition assessment noted a shortfall in the storage capacity at the Valleyview and Porter Creek Reservoirs. Consideration must be given to the ultimate requirements for storage to properly size the immediate needs.
Table 5.5 depicts the ultimate storage requirements for the City.

Table 5.5 Ultimate Storage Requirements

Ultimate	Fire (m^3)	Supply (m^3)	Peak (m^3)	Emergency(m^3)	Total (m^3)	Existing (m^3)	Surplus / Deficit (m^3)
Copper Ridge	3,150	5,114	2,557	767	11,588	6,449	-5,139
Hillcrest	3,150	1,270	635	191	5,246	6,058	812
Porter Creek	3,150	4,343	2,172	651	10,316	5,190	-5,126
Riverdale	3,150	6,370	3,185	956	13,661	12,076	-1,585
Valleyview	3,150	9,968	4,984	1,495	19,597	5,032	-14,565
					60,408	34,805	-25,603

By far the Valleyview Reservoir requires the largest expansion. This is primarily due to the growth within its service area. Growth centres including the Tank Farm Expansion, Lower Porter Creek Bench and infill / redevelopment in Hillcrest, Takhini and McIntyre. The Valleyview Reservoir zone also supplies McIntyre Creek and Hamilton Boulevard Pumphouses.

The Porter Creek Reservoir expansion will be required within the next five years in order to support growth in the region. An additional 5,500 m^3 should be added to this facility. Expansion will also allow for some future flows into the Lower Bench during peak demand periods. It is recommended that Lower Porter Creek Bench be primarily supplied through the Valleyview Reservoir zone. Flow from Porter Creek Reservoir zone should be limited due to the cost of pumping.

Copper Ridge reservoir will require an additional 5200 m^3 primarily due the development of the Beyond Copper Ridge area. As the timing of Beyond Copper Ridge is questionable this will likely not be required within a 20-year time frame.

5.4 WATER SYSTEM IMPLEMENTATION PLAN

The Implementation Plan for water system upgrades outlines Capital timing and priority based on growth projections.

Concept level cost estimates have been provided for each project identified in the Implementation Plan. More detailed cost estimates are recommended for budgetary purposes.

Since growth centers will trigger specific project priority, only the 0 to 5 Year Implementation Plan has been prioritized based on need or required project sequencing. The Implementation plans are presented in Tables 5.6 to 5.9. Proposed upgrade areas are depicted in Figure 5.1.

Table 5.6 Water System Implementation Plan (0-5 Years)

No.	Project	Item	2003 Cost ($)
1	Ground Water Investigation	• Construct test wells and report on findings	250,000
2	Selkirk Pumphouse Flow Meter	• Install flow meter external to Selkirk Pumphouse	65,000
3	Flow Meter Audit and Improvements	• Audit existing metering system and recalibration ($25,000) • Replacement of meters and installation of new meters ($225,000)	225,000
4	McIntyre Creek Pumphouse and Backup Power Improvements	• Upgrade 3 booster pumps ($150,000) • Installation of backup generator ($175,000). Install pressure sensors in Porter Creek (50,000)	375,000
5	Hamilton Pumphouse Backup Power	• Install backup generator at Hamilton Pumphouse	175,000
6	Water Treatment Plant Investigations[17]	• Water Quality Management Plan including operating procedures ($100,000) • Schwatka Lake thermal and quality profiling ($25,000) • Infiltration gallery pilot project ($50,000) • Water treatment pilot project ($200,000)	375,000
7	Ground Water Well Development [16]	• Develop new ground water wells (2)	750,000
8	Water Treatment Plant[16]	• Construct a water treatment plant at Selkirk • Costing from 2001 Epcor Study including engineering was $9,901,081	10,000,000
9	Downtown Westside Thermal & Fire Flow Improvements	• Construct watermains to complete looping ($600,000) • Construct circulation station ($130,000)	730,000

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[17] Work required is contingent upon the findings of the Ground Water Investigation

Table 5.6 Water System Implementation Plan (0-5 Years)

No.	Project	Item	2003 Cost ($)
10	Circulation Facility Fire Flow Improvements	• Install fire flow valves at Transit, Hart and Centennial circulation stations	150,000
11	Porter Creek Fire Flow Deficiencies	• Install pressure sensors along Grove Street and Ponderosa Drive with pump control connection to McIntyre Pumphouse	50,000
12	Marwell Fire Flow Improvements	• Install New Watermain from Two Mile Booster to Galena Road during Industrial Road Reconstruction	350,000
13	Water and Sewer Consumption Audit and Demand Management Strategy	• Audit water and sewer consumption by area after better data is available from flow meter upgrades • Update models • Establish Demand Management Strategy to reduce water demand.	100,000
14	Porter Creek Reservoir Expansion	• Construction of a new 5,500 m^3 cell	2,500,000
15	Riverdale Thermal Deficiencies	• Install thermisters on water main and provide alarming (temporary solution)	75,000
16	Reservoir Flow Monitoring	• Upgrade PLC logic and SCADA logic to calculate reservoir inflow and outflow from level at Riverdale, Valleyview and Copper Ridge	10,000
17	Crestview Thermal Improvements	• Construct circulation station and modify piping	200,000
18	Hillcrest Thermal Improvements	• Construct supply main from Hamilton Pumphouse to Hillcrest	500,000
19	Airport Thermal Improvements	• Installation of TCB at strategic locations (2)	75,000
20	Takhini North Thermal Improvements	• Decommission Elvin's and Ortona PRV Stations. Work to be completed during Takhini North redevelopment	-
		0-5 Year Total	**16,955,000**

Table 5.7 Water System Implementation Plan (5-10 Years)

No.	Project	Item	2003 Cost ($)
21	Riverdale Thermal and Fire Flow Improvements	• Construct new water mains from Selkirk to Boswell Crescent and Hyland Crescent – 3,500 m of 400 mm or 300 mm	2,000,000
22	Valleyview Reservoir Expansion Phase I	• Construction of a new 7,000 m^3 cell	3,000,000
23	Hamilton Booster Upgrade	• Upgrade 3 booster pumps	150,000
24	Alaska Highway Watermain Improvements	• Twin pipe along Alaska Highway from Elvins to Hamilton Blvd – 270m of 400 mm diameter water transmission main.	230,000
25	Porter Creek Extension Transmission Main	• Construct transmission main to service expansion area	550,000
26	Tank Farm Expansion Transmission Main	• Construct transmission mains to service expansion area	1,500,000
		5-10 Year Total	**7,430,000**

Table 5.8 Water System Implementation Plan (10-20 Years)

No.	Project	Item	2003 Cost ($)
27	Selkirk Booster Upgrade	• Upgrade Booster 1 Pump	150,000
28	Lower Porter Creek Bench Expansion	• Construct transmission main	2,800,000
		10-20 Year Total	**2,950,000**

Table 5.9 Water System Implementation Plan (20+ Years)

No.	Project	Item	2003 Cost ($)
29	Valleyview Reservoir Expansion Phase II	• Construction of a new 8,000 m^3 fourth cell	3,200,000
30	Beyond Copper Ridge	• Upgrade Copper Ridge booster pumps ($270,000) • Expansion of Copper Ridge Reservoir ($2,200,000) • Transmission Main ($3,000,000) • Reconstruct Hamilton Boulevard Pumphouse (3,000,000)	8,470,000
31	Two Mile Booster Station Booster Upgrade	• Install new booster pump	250,000
32	Riverdale Expansion Transmission Main	• Construct transmission main from Riverdale Reservoir ($1,500,000) • Construct transmission main from Downtown ($1,500,000)	3,000,000
33	Hamilton Boulevard Watermain Improvements (Beyond Copper Ridge)	• Twin pipe along Hamilton blvd. East of the Hamilton Blvd. Booster station - 900m of 300 mm water transmission main. ($630,000) • Twin pipe along Hamilton Blvd. Southwest of the booster station to Falcon Drive- 570 m of 300 mm diameter water transmission main. ($410,000) • Twin pipe along Hamilton Blvd. From Falcon Drive to the Granger booster station – 680m of 300 mm diameter water transmission main. ($480,000)	1,520,000
		20+ Year Total	**16,440,000**

6.0 Water System Conclusions and Recommendations

6.1 WATER SYSTEM CONCLUSIONS

Proposed Studies and Investigations

1. A very important alternative approach to water and sewer infrastructure upgrade due to increased demand is demand side management. The City of Whitehorse currently has high per capita water usage and should develop a water demand management strategy that encompasses: leak detection and repair, bleeder reduction, education, metering, rate structuring, economic incentive, regulation, politics, and plumbing fixtures. A water usage audit may also be incorporated into water demand management strategy. (Section 2.6.10 Water Demand Management)

2. The City may wish to consider an audit of all commercial services to identify type of installation, wastage of water and types of fixtures. An audit can also identify storm to sanitary cross connections concerns and water cross connection concerns. Data can be recorded on the City's GIS system. This project has not been identified in the Capital Implementation Plan as it can be done with internal resources. (Section 2.6.10 Water Demand Management)

3. Based on information from operations staff, some flow meter results may be suspect. The City currently does not have a structured meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement, calibration or data collection. Some work has been recently undertaken to install new meters and get non-functioning or improperly function meters operating but the metering program still needs improvement. The City needs to conduct a metering audit and implement a structured metering program. (Section 2.6.4)

4. Drinking water quality has become and important issue in recent years. Development of an all-encompassing water quality management plan may be beneficial to provide plan management direction and corporate priority. The City currently only has a few official critical operating procedures documented. This work needs to be completed. (3.3.2 Water Quality)

5. The City should conduct further investigations into locating additional ground water sources due to the importance of the thermal gains, lack of turbidity and source security. Even if new aquifers are found to be difficult to treat for mineral content and hardness the City should investigate the potential use of heat pumps to temper surface water. (Section 3.3.3 Ground Water Wells)

6. The need for a water treatment plant has been recognised. Before commencing detailed design of a water treatment plant the City should undertake further

study and pilot projects to ascertain the best treatment process. (Section 3.3.6 Treatment Plant Options)

7. Further investigation into the water supply sources should be conducted. Water quality and temperature profiling of Schwatka Lake may indicate that intake relocation will be beneficial. Additional groundwater source and quality investigation is important. Increased ground water supply will decrease the requirement for turbidity removal and improve system thermal characteristics during cold water periods. (Section 3.3.6 Treatment Plant Options)

8. The City may also want to investigate the potential use of a large-scale infiltration gallery to reduce Yukon River turbidity. (Section 3.3.6 Treatment Plant Options)

9. The City currently only has a few official critical operating procedures documented. This work needs to be completed and there may not be enough internal resources to complete the task. The City may want to consider including this work in its capital plan to either hire more staff or hire a consultant to complete the work. (Section 3.3.2 Water Quality)

10. An assessment of reservoir operating levels was not part of the study scope. Operating reservoirs at lower levels than dictated by storage requirements requires an assessment of risk as both storage and pressures can be impaired. An assessment of reservoir operating levels is recommended and the results should be posted at relevant pump stations, reservoirs and SCADA systems. This project has not been identified in the Capital Implementation Plan as it can be done with internal resources. (Section 3.5.1)

11. An assessment of reservoir mixing was not part of the study scope. Many water system operators in recent years have begun to introduce measures to improve reservoir mixing in order to improve reservoir thermal and chlorine residuals. An assessment of reservoir mixing and remediation measures is recommended. This project has not been identified in the Capital Implementation Plan as it can be done with internal resources. (Section 3.5.1)

Hydraulic System Deficiencies

1. A water system implementation plan has been presented in Section 5.4 that identifies required upgrades, estimated cost of upgrades and phasing. System deficiencies are noted as follows:

2. Improving ground water supply and construction of a water treatment plant should be considered the City's first priority for water and wastewater system upgrades.

3. Since ultimate build out capacity modeling was based on a potential capacity of 41,951 persons instead of a predicted population a detailed review of population projections and project phasing needs to be conducted prior to expansion of major facilities. (Section 2.4 Population Growth Scenarios)

4. It is critical that a new meter be installed at Selkirk Pumphouse to determine accurate total system demand prior to the design of a new treatment plant. (Section 2.6.4 Existing Average Day Demand)

5. If the City wants good future data on Peak Hour Demand, accurate flow meters will be required at all booster stations and reservoirs. It may be possible for the City's SCADA system to calculate nearly instantaneous reservoir inflow and outflow rates based on change in reservoir elevation. (Section 2.6.8 Peak Hour Demand)

6. Selkirk Pumphouse should be replaced as part of the water treatment plant construction. (Section 4.2.1.1 Hydraulic Evaluation Selkirk Pumphouse)

7. Long booster run times can be expected at Selkirk Pumphouse during ADD if only one booster is in operation. This is currently critical during times of high Schwatka Lake turbidity when only one booster can be operated to increase well water blending. (Section 4.2.3 Water Storage Reservoirs)

8. In the past the City has experienced very low water pressures in Riverdale, the hospital area and Downtown when two boosters were running at Two Mile Booster with no pumps operating at Selkirk Pumphouse. This situation is not reflected in the water model and further investigation may be warranted. The situation my have been caused by a closed or partially closed valve in the distribution system. If the situation still exists and cannot be remedied, the City should confirm that current SCADA control would not start two boosters at Two Mile Booster unless a booster is running at Selkirk. (Section 4.2.1.1)

9. Porter Creek reservoir is under capacity; therefore, it is recommended that hydraulic capacity upgrades be considered at McIntyre Creek Pumphouse. (4.2.1.3 Hydraulic Evaluation McIntyre Creek Pumphouse)

   - Standby power with enough capacity for control system requirements and ultimate maximum day demand pumping should be installed at this station.
   - The standby generator should also be able to meet at least ultimate averaged day demand requirements with full boiler operation.
   - It should be noted that the existing Mag meter used to record station flows should be evaluated for accuracy and recalibrated or replaced as required.
   - Fire flow conditions are a concern in some areas of Ponderosa Drive and Grove Street. Even with two pumps running at McIntyre, the residuals pressures are below the minimum standard. Pump upgrade is recommended.

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10. Standby power with enough capacity for control system requirements and ultimate maximum day demand pumping should be installed at Hamilton Boulevard Pumphouse. The standby generator should also be able to at least

meet ultimate averaged day demand requirements with full boiler and circulation pump operation. (4.2.1.6 Hydraulic Evaluation Hamilton Pumphouse)

11. Fire flows are a concern near Transit, Hart and Centennial Circulation stations when circulation pumps are in operation. The model findings need to be confirmed by field testing as modeling fire flows within circulation zones may not be accurate. Areas of concern need to be identified on Fire Department maps with instructions to contact the pumphouse on call operator to manually bypass circulation pumps. Fire flow bypass valves should be installed. (Section 4.2.2 Distribution Network)

12. Fire flow deficiencies were observed in Downtown's west side where fire flows of only 23 to 40 L/s were modeled. The deficiency is due to the limited interconnections of the systems and the smaller 150 mm diameter lines. Further, looping and upgrading dead end lines will improve frost protection and eliminate free flow bleeders. (Section 4.2.2 Distribution Network)

13. A watermain can be constructed from Selkirk Pumphouse to south and east Riverdale to improve fire flows and thermal flows. (Section 4.2.2 Distribution Network)

14. Pressure sensors should be installed at Ponderosa Drive and Grove Street with interconnection to McIntyre Pumphouse control to ensure McIntyre Pumphouse booster pumps operate during fire flows. (Section 4.2.2 Distribution Network)

15. Field fire flows need to be confirmed in Granger to ensure that both booster pumps will start on fire flow. (Section 4.2.2 Distribution Network)

16. The existing 250 mm Industrial Road supply main should be upgraded to 400 mm from Two Mile Hill to Quartz Road and upgrading the 250 mm main to 300 mm from Quarts Road to Galena Road to improve fire flows in Marwell. The City may also wish to investigate reducing demand requirements by making automatic building fire suppression systems mandatory. Hydrant spacing should also be reviewed near buildings with high fire flow demand requirements. (Section 4.2.2 Distribution Network)

17. It is important that water can back feed through Hamilton Pumphouse during fire flow events in Granger. Hamilton Pumphouse has electrically actuated fire flow valves that open on low pressure. Currently there is no standby power at Hamilton Pumphouse to open fire flow valves during power disruption. Standby power is required at Hamilton Pumphouse. (Section 4.2.2 Distribution Network)

18. Valleyview, Porter Creek and Riverdale reservoirs are undersized. (Section 4.2.3 Water Storage Reservoirs)

    - The Riverdale reservoir is undersized but due to the proximity of the Selkirk pumpstation it is not a large concern but consideration needs to be made for upgrading of Selkirk Pumphouse boosters to ensure the station meets future maximum day demand.

- The Porter Creek reservoir is the most critical in terms of expansion. The remote location of this reservoir makes this expansion a priority.
- Porter Creek is a single cell reservoir that cannot be easily shut down for maintenance.

Thermal Deficiencies

1. In order to eliminate City TCB bleeders in Crestview, a control valve (PRV) can be constructed in a vault located at the intersection of Klukshu and Rainbow Roads. The PRV would cascade warmer water from the upper zone into the lower zone and flow can travel around the loop and discharge at the suction side of the Crestview booster station. Some additional piping will have to be constructed to complete the loop. (Section 4.3 Thermal Evaluation)
2. In order to improve Downtown Westside thermal conditions watermain looping on the west extents of the Downtown distribution system is required. Distribution piping installation of 700 m of 200 mm minimum would be required. A circulation station could be introduced at the end of Strickland Street. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
3. In order to improve Hillcrest thermal conditions a supply line to Hamilton Boulevard Pumphouse should be installed. The circulation pump for McIntyre neighbourhood will induce a flow around the loop. The supply line would have the added benefit of improving fire flows to Hillcrest. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
4. TCB should be installed in strategic locations at the airport to improve thermal requirements. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
5. Ortona Pressure Reducing Station and Elvin's Pressure Reducing Station will likely be decommissioned when Takhini North is reconstructed. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
6. The heat source at Selkirk should be increased and alarmed temperature monitoring stations should be installed in the western extents of Riverdale to improve thermal requirements. Water can be bled manually from abandoned bleeders if required. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
7. Hart Circulation Station loop does not meet thermal requirements. An increased source heat at Selkirk will improve deficiency. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)
8. The most effective and beneficial method of providing increased thermal protection throughout the City is to improve source temperatures by increasing ground water supply capacity or using heat from a geothermal heat pump. (Section 4.3.2 Evaluation of Thermal Improvement Alternatives)

Servicing New Development Areas

1. Water supply and storage for the Lower Porter Creek Bench is recommended primarily from the Valleyview Reservoir via Range Road with a secondary feed from Porter Creek via Range Way. (Section 4.4.1 Lower Porter Creek Bench)

2. Porter Creek Extension can be serviced off the existing watermains in Porter Creek with circulation considerations. The area can be serviced from a 300 mm water main located on Mountain View Drive / Hickory Street. (Section 4.4.2 Porter Creek Extension)

3. Servicing of Beyond Copper Ridge was chosen through existing Copper Ridge for the ultimate analysis in order to assess the impact of Beyond Copper Ridge on existing systems. Determination of the best servicing option requires further development analysis. (Section 4.4.3 Beyond Copper Ridge)

4. The Tank Farm Expansion area can be serviced from the Valleyview Reservoir. A connecting to the new airport transmission main with secondary connection to the transmission main on Hamilton Boulevard can be made. (Section 4.4.4 Tank Farm Expansion.

5. The Riverdale Expansion can be serviced with the installation of a 300 mm supply line from the 350 mm Riverdale Reservoir line. The capacity of Riverdale Reservoir has to be considered. A secondary feed would ultimately be required, which can potentially be provided by a connection to the Marwell distribution system. Alternately, water wells can developed in the Riverdale Expansion Area. (Section 4.4.5 Riverdale Expansion)

Future and Ultimate Deficiencies

1. Under the ultimate build out capacity loading condition an average day demand (ADD) of 339 L/s and a maximum day demand (MDD) of 677 L/s are estimated to be required by the city. The proposed water treatment plant and Selkirk Pumphouse will ultimately be required to provide enough treated and sufficiently warmed water to meet MDD. (Section 5.2.1.1 Supply)

2. Two Mile Hill booster station performs adequately under ultimate flow estimates with the addition of the planned fifth pump. (Section 5.3.1.2 Two Mile Booster Station)

3. The current pumps at McIntyre will be adequate to a moderate population increase. Moreover fire flow analysis suggests upgrades in order to accommodate fire flows along Ponderosa Drive and Grove Street. Upgrading should be considered within the next two years. Ultimate flow requirements will be approximately 100 L/s at a head of 50 m. (Section 5.3.1.3 McIntyre Booster Station)

4. Hamilton Pumphouse should be adequate to a population increase of 1,000 people (14 L/s), which could be in the very near future as Copper Ridge has

seen significant population increase since study populations were established. Upgrading to three (3) 50 hp pumps is recommended. The station will require further upgrade and likely replacement if Beyond Copper Ridge is developed. (Section 5.3.1.6 Hamilton Boulevard Pumphouse)

5. Copper Ridge Pumphouse should be adequate to a population increase of 4,500 people (30 L/s). This is well beyond the build out population of the Copper Ridge area. At full capacity, with Beyond Copper Ridge, the station has been simulated at 74 L/s at a TDH of 34 m. Ultimately pumps capable of producing 118.5 L/s at a TDH of 36 m would be required to supply Beyond Copper Ridge. (Section 5.3.1.7 Copper Ridge Pumphouse)

6. Twinning the pipe along Alaska Highway from Elvins to Hamilton Boulevard with 270m of 400 mm diameter water transmission main is ultimately required. (Section 5.3.2 Distribution Network)

7. Twinning the pipe along Hamilton Boulevard east of the Hamilton Boulevard Booster station with 900m of 300 mm diameter water transmission main is ultimately required. (Section 5.3.2 Distribution Network)

8. Twinning the pipe along Hamilton Boulevard southwest of the booster station to Falcon Drive with 570 m of 300 mm diameter water transmission main is ultimately required. (Section 5.3.2 Distribution Network)

9. Twinning the pipe along Hamilton Boulevard from Falcon Drive to the Granger booster station with a 680m of 300 mm diameter water transmission main is ultimately required. (Section 5.3.2 Distribution Network)

10. Valleyview Reservoir requires the largest expansion. This is primarily due to the growth within its service area. Growth centres including the Tank Farm Expansion, Lower Porter Creek Bench and infill / redevelopment in Hillcrest, Takhini and McIntyre. (Section 5.3.3 Water Storage Reservoirs)

11. The Porter Creek Reservoir expansion will be required within the next five years in order to support growth in the region. An additional 5,500 m^3 should be added to this facility. Expansion will also allow for some future flows into the Lower Bench during peak demand periods (It is recommended that Lower Porter Creek Bench be primarily supplied through the Valleyview Reservoir zone). (Section 5.3.3 Water Storage Reservoirs)

12. Copper Ridge reservoir will require an additional 5200 m^3 primarily due the development of the Beyond Copper Ridge area. As the timing of this is questionable this will not occur within the 20-year time frame. (Section 5.3.3 Water Storage Reservoirs)

Operations and Maintenance Programs

1. The City has recently undertaken a multi-year Supervisory Control and Data Acquisition (SCADA) implementation program. Because of the benefits gained by SCADA, including vastly improved system knowledge, efficiency, reliability and accountability, the City should continue with its SCADA implementation plan and budgeting. (Section 2.2.2 Existing Operation and Maintenance Activities)

2. In 2002, the City of Whitehorse implemented a unidirectional watermain flushing program developed by Epcor. The unidirectional flushing program allowed the City to systematically and efficiently flush watermains to remove turbidity settlement. The unidirectional flushing program is beneficial to water quality and preventative maintenance should be continued. (Section 2.2.2 Existing Operation and Maintenance Activities)

3. The City has in recent years actively strived to reduce water bleed and boiler operation times for frost protection. The City manually and automatically monitors water system temperatures and ground frost conditions to optimize when to turn on and off frost protection systems. This practice should be continued as apposed to automatically turning on and off frost protection systems on specific dates. (Section 2.2.2 Existing Operation and Maintenance Activities)

4. In order to continue flow reduction strategies, the City should consider implementation of residential flow metering. Since water metering is politically difficult to implement, the City could create an incentive for homeowners to switch to metering. (Section 2.6.2 Historical Water Demand)

5. The City of Whitehorse has in recent years allowed Kathleen Bleeder to flow free during summer months in order to refresh water in the area when chlorine residuals get too low. It is recommended that the City investigate improved circulation, re-chlorinization or use only intermittent bleeding for water quality. (Section 2.6.3 Bleeders)

6. There have been a very small percentage of TCB failures resulting in customer service freezing and free flowing bleeders. (Section 2.6.3 Bleeders)

   - Solutions to problems should continue to be found in order to maintain customer confidence in the units.
   - TCB's can bleed much more water than the original free flow bleeders they replaced if costumers force them into free flow.
   - Services highly susceptible to freezing should have TCB's replaced with flow rated free flow bleeders.
   - Implementation of a residential water metering program would ensure that customers are operating bleeders properly.

7. The City has recently begun installing new thermisters at strategic locations during regular digging operations. This practice should be continued. Also, permanent thermisters should be installed during new developments. (Section 3.6.4 Review Temperature Monitoring Data)

8. Field fire flow monitoring should be used to verify fire flow model findings and the Fire Department should be notified via system maps of any areas that are in risk of pressures dropping below 140 kPa (20 psi). (Section 4.2.2 Distribution Network)

9. Due to upstream pumping and reservoir dynamics, deep cycling of any reservoir during times of high demand is not recommended. (Section 4.2.3 Water Storage Reservoirs)

10. An assessment of the physical condition or actual operating conditions of infrastructure was not part of the scope of this study; however, it is very important to a capital replacement or upgrade plan. The City of Whitehorse should improve system condition documentation in its preventative maintenance programs. This may be achieved by good field checklist procedures and record keeping forms and/or seasonal information gathering and documentation sessions with operations staff. It is recommended that Engineering staff be involved in any documenting procedures so that Engineering gets information they require for capital planning.

11. The City has in recent years actively strived to reduce water bleed and boiler operation times for frost protection. The City manually and automatically monitors water system temperatures and ground frost conditions to optimize when to turn on and off frost protection systems. This practice should be continued as apposed to automatically turning on and off frost protection systems on specific dates.

6.2 WATER SYSTEM RECOMMENDATIONS

The following recommendations are made based on the findings of this study

1. The City should implement the 5-year Implementation Plan as outlined in this report.

2. The City should continue long-term planning with regard to neighbourhood development and resulting water system upgrades.

3. Drinking water quality has become and important issue in recent years. The City should develop and all encompassing Water Quality Management Plan that includes an in house sewer and water task force, a back flow prevention program, staff training, water license revision, watershed and well head management, and development of operating procedures.

4. A water treatment plant is necessary to provide security to the city's drinking water quality. Pilot programs and further investigations are required to identify the best treatment options for the City.

5. As part of a water treatment plant design, the City needs to do more investigation into source water including, expansion of the groundwater supplies and possible relocation of surface water intakes.

6. The City should periodically recalibrate the hydraulic and thermal models based on the future flow and temperature data.

7. The benefits of SCADA are recognized in the form of vastly improved system knowledge, efficiency, reliability and accountability. The City should continue with its SCADA implementation program.

8. The City should conduct a Metering Audit and the City should develop a structured meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement, calibration or data collection.

9. An assessment of the physical condition or actual operating conditions of infrastructure is very important to a capital replacement or upgrade plan. The City should improve system condition documentation in its preventative maintenance programs.

7.0 Basic Information and Criteria-Sanitary Sewer System

7.1 SEWER SYSTEM SERVICED AREA

The service area for the City of Whitehorse sanitary system encompass a wide variety of land uses spread over a large area. Over the years, the City has expanded to several distinct topographical areas resulting in three independent sanitary sewerage systems servicing the urban areas. The rural or "country" portions of the city remain un-serviced. The three sanitary sewerage systems are as follows:

1. Marwell Lift Contributing Area
2. Porter Creek Sewerage System
3. Crestview Sewerage System

Figure 7.1 shows these three sewerage systems.

The City of Whitehorse storm system consists of an extensive sewer network and Strickland Storm Lift in the Downtown area and lesser storm systems in other neighbourhoods. With the exception of Riverdale and Downtown, the majority of storm systems consist of small collection areas with surface discharge. An evaluation of the storm system was not part of the scope of this study.

7.1.1 Marwell Lift Contributing Area

The Marwell Lift Contributing Area is the largest sanitary sewerage system and services the urban areas of Whitehorse south of Porter Creek. The serviced areas include Downtown, Riverdale, Marwell, Hillcrest, Airport, Takhini, Valleyview, Kopper King, McIntyre, Granger, Copper Ridge, Arkell and Logan. Within this system, a network of gravity sewers in conjunction with three major lift stations, collect the sewage and transport it into the Livingston Trail Environmental Control Facility (LTECF) located on the east bank of the Yukon River to the northeast of the Porter Creek area. The system services approximately 13,340 people.

Within the Marwell Lift Contributing Area there are several drainage areas. All sewage from the Riverdale subdivision is collected at Lift Station # 3 located on the east side of the Robert Campbell Bridge adjacent to the Yukon River and transported under pressure to the Downtown collection network.

Flows from Downtown, the Airport area, and old Hillcrest drain to Lift Station # 1, located at Second Avenue and Ray Street in the northwest corner of Downtown. From Lift Station # 1, the sewage is pumped via a forcemain to the Marwell Lift Station.

Collection trunks terminating at the Marwell Lift Station service Marwell, Valleyview, Takhini, Kopper King, McIntyre, Granger, Arkell, Logan and Copper Ridge. Sewage from the Marwell Lift Station is pumped under the Yukon River via forcemain to the LTECF.

There are several minor lift stations located within the Marwell Lift Contributing Area as follows:

- Alsek minor lift station and Hyland lift station are located within the Riverdale subdivision.
- Quartz Lift, a small station, is located at Quartz Road and conveys flows from the Chilkoot Centre and the Waterfront Place Developments on the north end of Downtown.
- Mountainview Place is a minor lift station in the Range Road North Area.
- Crow Street is a minor lift station in the Range Road North Area and is operated by the Kwanlin Dun First Nation and services the Crow Street residential neighbourhood. The City may assume operation of Crow Street Lift Station in 2004.
- A minor lift station on the south end of Range Road is operated by Northwestel and services the commercial area developed from the lands formally owned by Northwestel. The City never assumed operation of this minor lift, as it does not meet City standards.
- There are several private grinder pumps along Copper Road, which discharge into a shallow buried 50 mm insulated forcemain.
- There is a lift station located in an undeveloped area of McIntyre Neighbourhood that has never been commissioned. The lift station has been extensively vandalised and all equipment has been removed.

7.1.2 Porter Creek Sewerage System

The next largest sewage collection system is the Porter Creek Sewerage System. This system has three minor (Tamarack, 11th & Pine, 9th & Fir) lift stations and one major lift station (Clyde Wann Lift). All sewage from the system is conveyed to the Porter Creek flush tank located at the northeast corner of Porter Creek near Larch Road and Oak Street. From the flush tank, the sewage flows through the gravity siphon/forcemain across the Yukon River into the LTECF. The flush tank uses an automated retention and release system in order to provide flushing velocity through the Yukon River siphon crossing. The existing serviced population of this sanitary system is approximately 4,100 people.

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7.1.3 Crestview Sewerage System

A minor sewage collection and disposal system services the Crestview Neighbourhood. This system collects and transports sewage to the Crestview

Lagoons situated on the west side of the Yukon River below Crestview. This system serves 800 people.

7.1.4 Un-Serviced County Residential Areas

County Residential and un-serviced areas within the urban and rural portions of the City of Whitehorse operate private disposal systems or transport sewage to the Whitehorse system. The populations of these areas are not included in the sanitary sewer calculations.

7.2 SEWAGE SYSTEM ULTIMATE AREA

As outlined in Section 2 of this report, the three City sewerage systems are to be extended to infill the existing urban portions of the City of Whitehorse, as well as service proposed neighbourhoods, for an ultimate build out capacity population of 41,951.

7.2.1 Ultimate Marwell Lift Contributing Area

The Marwell Lift Contributing Area can expand in all existing contributing areas. The future Beyond Copper Ridge development may also serviced to Marwell Lift. The built out population for the Marwell Lift Contributing Area is approximately 27,900.

7.2.2 Ultimate Porter Creek Sewerage System

The Porter Creek Sewerage System can be expanded to collect sewerage from the Porter Creek Extension Area and the proposed 5,000 person Lower Porter Creek Bench area, located on the bench below Porter Creek. The proposed ultimate build out capacity population of the Porter Creek area is 13,300.

7.2.3 Ultimate Crestview Sewerage System

The population of the Crestview Sewerage system will remain at about 800 people. There are no major expansion plans for the Crestview System.

7.2.4 County Residential

There are no plans to service the County Residential portions of the city.

7.3 DEFINITION OF SEWER SYSTEM

The terminology used for sanitary sewer analysis is similar to that used for water supply, with some minor changes and additions. The terminology is as follows:

CONTRIBUTING AREA is defined as the areas or portions of the area serviced by each sewerage system. Within each contributing area is the serviced population of the systems. It should be noted that the contributing areas are not necessary related to the water servicing areas as detailed in the Water portions of this study.

MINOR LIFT STATION is any sewage lift station or pump station equipped with wet well installed submersible pumps. For example, the Alsek Lift Station in Riverdale is a minor lift station.

MAJOR LIFT STATION is any sewage lift station or pump station that has both a wet well and a dry well. These stations tend to be significantly larger than minor lift stations and service a greater area. An example of a major lift station is Lift Station # 1.

7.4 CONTRIBUTING AREAS AND SEWAGE GENERATION RATES

7.4.1 General

As previously discussed, the City of Whitehorse sewerage system comprises three separate systems and, therefore, not all the water delivered to a certain "water area" becomes sewage for that area. For example, water delivered to water distribution Area 3 is collected by Porter Creek and Crestview sewer systems. A breakdown of sewage generation by area is shown in Table 7.1.

Table 7.1 Summary of Existing Sewage Generation Contributing Areas

Contributing Area	Neighbour-hoods within Contributing Area	Water Area	Main Discharge Facility	Down-stream Discharge Area	Discharge Facility
Riverdale	Riverdale Whitehorse Hospital	Area 1 Area 1	Lift Station #3	Downtown	LTECF
Downtown	Downtown	Area 1	Lift Station #1	Marwell	LTECF
Hillcrest	Hillcrest Airport	Area 2 Area 2	Hillcrest Outfall	Downtown	LTECF
Granger	Granger	Area 4	McIntyre Outfall	McIntyre	LTECF
McIntyre	McIntyre	Area 2	McIntyre Outfall	Takhini	LTECF
Takhini	Takhini Yukon College Valleyview Kopper King	Area 2 Area 2 Area 2 Area 3	Takhini Outfall	Marwell	LTECF
Marwell	Marwell	Area 1	Takhini Outfall	Marwell	LTECF
Arkell/Logan	Arkell/Logan	Area 5	McIntyre Outfall	McIntyre	LTECF
Copper Ridge	Copper Ridge	Area 5	McIntyre Outfall	McIntyre	LTECF
Porter Creek	Porter Creek	Area 3	Porter Creek Outfall	Porter Creek Flush Tank	LTECF
Crestview	Crestview	Area 3	Crestview Outfall	Crestview Lagoon	CV Lagoon

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7.4.2 Generation Assumptions for Sewage Collection and Pumping

The total sewage flow from a contributing area can be calculated by adding the individual components of sewage flow. The basic components of sewage flow are as follows:

- Residential portion of the sewage flow
- Commercial/industrial/institutional portion of the sewage flow
- Public lands portion of the sewage flow
- Watermain bleeder flow
- Residential/commercial/public lands bleeder flow
- Ground water Infiltration into the system
- Inflow from cross connections between the sanitary and storm systems and through manholes into the system

Water lost from the water system does not contribute to sanitary sewage flow. Water can be consumed by lawn watering or car washing and not be returned to the sewerage system. Water can also be lost due to leakage. A portion of sewage collected can also be lost by system exfiltration.

The sewage generated in a collection system can be expressed as follows:

Total Sewage = Water Consumption – Water Loss + Inflow + Infiltration - Exfiltration

As accurate exfiltration data is seldom available, infiltration generally represents the net flow of actual infiltration less exfiltration.

7.4.3 Analysis of Flow Monitoring Data

The following flow meters were used to obtain flow data for the sewer collections systems:

1. Marwell Contributing Area flow to the LTECF is monitored by a flow meter at Marwell Lift Station.
2. Porter Creek flow to the LTECF is monitored at the Porter Creek Flow Meter located on the Porter Creek Gravity Forcemain down the hill from the Porter Creek Flush Tank.
3. Sewage flow to the Crestview Lagoons, was monitored semi-annually at data collection station WH5 (influent into Crestview Lagoon Primary Cell A) but was discontinued in 2002 due to flow inconsistencies and inaccuracies.

Based on the analysis of the available recorded data at various pumping stations and the treatment facilities, the total sewage generated can be separated into two components: residential sewage and non-residential sewage. Both sewage components include Inflow/Infiltration, bleeder flows and any other sources that are not accounted for.

A water system to sewage system recovery factor of 0.85 was established by the 1990 Water and Sewer Study and was adopted as a starting point for modeling in the current study. The recovery factor can be used to compute sewage volumes based on water consumption in each water service areas as follows:

$$Sewage\ Volume = 0.85\ x\ Water\ Consumption$$

The recovery factor can also be expressed as follows:

$$R_F = \frac{WaterConsumption - WaterLoss + Infiltration - Exfiltration}{WaterConsumption}$$

The 1997 Ground Water Infiltration study did not attempted to estimate infiltration to mains that were 300 mm and larger as accurate data for the larger mains was not obtained.

As there was insufficient data available to the current study to specifically estimate the sewage volumes that could be attributed to infiltration throughout the City, an equal volume was distributed by area. The distribution ratio was then adjusted in the model until the total sewage volumes based on consumption matched the measured sewage volumes. The estimated residential and non-residential flow rates have been presented in Table 7.2.

Table 7.2 Sewage Generation Volumes under Existing Development Conditions

See Excel Sheet

The total measured annual sewage volume at the Marwell Lift Station for the year 2000 is 3,473,000 m^3 including the volume of inflow/infiltration whereas the total annual water consumption for the services area drained into the Marwell Lift Station is 3,883,636 m^3. The gross recovery rate is approximately 0.9. The annul sewage volume generated from the Porter Creek area is 752,100 m^3 and the annual water consumption volume for this area is 801,941 m^3 resulting in a recovery rate of 0.94. There was no information available on the total annual sewage volume generated from the Crestview area.

The existing composite sewage generation rate is about 655 Lpcd. The 1990 Water and Sewer Study reported an average sewage generation rate of approximately 1100 Lpcd.

The LTECF facility was designed to fulfil the City's sewerage treatment needs until year 2012 on the assumption that the City's sewage flow generation has to be reduced to 570 Lpcd by the year 2012. These figures were based on the Klohn Leonoff / Nova Tec Phase III Report produced in 1992. The Marwell Lift Station has also been designed on the same assumption. As the generated sewage volume is continuously decreasing, the City of Whitehorse needs to continue their flow reduction strategy to achieve the desired goal. Based on the total areas served by each lift station and the treatment facilities, Table 7.3 presents the average daily volume of flows to each facility.

Table 7.3 Inflow Volumes to Various Facilities under Existing Conditions

Facilities	Non-residential area (ha)	Population	Total Estimated Sewage (m^3/day)	Monitored Sewage in 2000 (m^3/day)	Average Composite sewage rate (Lpcd)
Lift Station # 3	49.0	5,354	3982	N/A	744
Lift Station # 1	134.1	8,246	7054	N/A	855
Marwell	335.6	13,341	9575	9,515.0	718
Clyde Wann	4.5	2,049	913.0	N/A	445
Porter Creek Flush Tank	28	4,139	2,060	2,060.0	498
Total to LTECF	**363.6**	**17,480**	**11,635**	**11575.0**	**666**
Crestview Lagoon	0	799	312.0	N/A	390
Total	**363.6**	**18,279**	**11,947**		**654**

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These average sewage generation rates and the daily volume of generated sewage have been used to analyze the hydraulic response of the existing sanitary sewer system.

The analysis of the flow data monitored in September 2000 at the Dogwood Sanitary Sewer in Porter Creek results in a peak flow ratio (ratio of peak flow to average flow) of about 1.35 during the dry weather flow conditions. Also, the analysis of flow data monitored in 1996 at the Marwell Sewer Trunk at Tlingit Street results in a peak flow ratio of about 1.3 during the dry weather flow conditions. The peak flow ratio in the sewer trunk of that magnitude during dry weather flow conditions is typical in urban areas. However, the peak flow ratio during wet weather flow events varies significantly from place to place. This wide variation is due to the rainfall induced inflow/infiltration and depends on the intensity and duration of rainfall events, soil type, pipe material, system condition, ground water level, cross-connections, etc.

Generally, experience elsewhere indicates that the peak flow ratio during wet weather events could vary from 3 to 4 under normal conditions. Realistically, the peak flow ratio for a particular location needs to be established by implementing a flow monitoring program that collects and analyzes flow and rain data during wet and dry weather periods.

There was no wet weather flow data available for the City of Whitehorse. As a result, the hydraulic response of the sewer system has been evaluated by utilizing a peak flow ratios of 3 and 4 as discussed in further detail in Section 8.2.

7.4.4 Storm to Sanitary Cross Connections

It has been the practice in the City of Whitehorse to not provide private storm servicing. Building down spouts and property surface drains are generally not directly connected to the City sanitary or storm sewer. Basement sumps are not common in Whitehorse with the expectation of Downtown. A number of commercial buildings Downtown have basement sumps connected to the sanitary sewer services. City staff have witnessed some Downtown commercial sumps generating significant flows; however, this information has not been documented.

If City servicing information is ever converted to GIS, unusual service conditions can easily be documented when encountered. The City may also wish to consider an audit of all commercial services to identify type of installation, sanitary cross connections concerns, water cross connection concerns and wastage of water.

The 1997 Sanitary Sewer Main Ground Water Infiltration Study identified a number of storm to sanitary cross connections in the Manhole Repair Lists. The cross connections are presented in Table 7.4.

Table 7.4 Storm to Sanitary Cross Connections

Sanitary Manhole	Street	Neighbourhood	Comment
145	Alsek Road near Lewes	Riverdale	CB cover
146	Alsek Road near Lewes	Riverdale	CB cover
83	Lewes at Klondike	Riverdale	Yes
175A	Lewes fronting F.H. Collins School	Riverdale	Yes
66	Centennial at MacDonald Road	Porter Creek	2
14	Larch and Redwood	Porter Creek	Possible
156	Black Street	Downtown	Yes
178	Fifth Avenue and Wheeler Street	Downtown	Yes
8	Taylor Street west end	Downtown	2
57	Lambert Street west end	Downtown	Yes
151	Alexander Street and Fourth Avenue	Downtown	Yes
220	Fourth Avenue near Lambert	Downtown	Yes
165	Fourth Avenue at Black Street	Downtown	Yes
168	Black Street at Third Avenue	Downtown	Yes
213	Baxter Street west end	Downtown	Yes
215	Fourth Avenue and Baxter Street	Downtown	2
194A	Ogilvie Street at Sixth Avenue	Downtown	Yes
196	Ogilvie Street at Fourth Avenue	Downtown	Yes
22	Lane adjacent to Robert Service Way	Downtown	Yes
60	Lambert Street at Third Avenue	Downtown	Yes
53	Second Avenue and Hanson Street	Downtown	Yes
155	Second Avenue and Alexander Street	Downtown	Yes

The cross connections at manholes 145 and 146 consist of barrier curb catch basin covers on sanitary manholes. The Catch Basin covers are not required for storm drainage. The catch basin covers were installed because barrier curb type sealed manhole covers are not available.

The cross connection at manhole 156 at the end of Black Street is operated seasonally. A storm system that drains the gulch at the end of Black Street is switched to the sanitary system in winter because the storm system freezes.

The existence and type of each cross connection needs to be confirmed as the field investigations during the 1997 study were conducted by permanent and temporary staff with various levels of experience. It is expected that some locations listed are not actual cross connections as some are adjacent to storm drainage systems. Only Downtown manholes 21, 156, 213 and 215 were identified on overall storm sewer maps. This information needs to be confirmed and updated.

The cross connections listed in Table 7.4 were not specifically identified in the model as they would require additional storm analysis beyond the scope of this study.

7.5 SEWAGE COLLECTION AND NETWORK PIPE INFORMATION

7.5.1 Trunk Main Details

The sewage collection and disposal system for the City of Whitehorse starts at the individual residential or commercial service and terminates at the outfalls of the each of the three systems. As the sewage progresses through the pipe network, the combined flows result in progressively larger pipe diameters. The sewage emanates from each source typically through a 100 mm service pipe. The service pipe connects to a lateral or local pipe, typically 150 or 200 mm in diameter. Several laterals join to form sub-trunks and trunks, from which sewer mains carry the flow. For the purpose of this report, we shall define any pipe 200 mm and smaller in diameter as a local, or lateral pipe and any pipe larger than 200 mm as a sewer trunk or main. Trunk capacity is the focus of the sewer study.

Trunk details used in the computer simulation models were obtained from the as-built drawings of the sewer networks within each contributing area and local subdivision. The key information required is manhole numbering, pipe diameter, pipe length, pipe material, pipe material, network path flows, land uses and pipe/manhole inverts. The as-built drawings and sub-division plans were utilized to obtain the required pipe information.

7.5.2 Manning Roughness Coefficient, "n"

The published values for Manning "n" used in the flow calculations range from 0.013 for newer concrete pipes to 0.017 for older rougher pipes. With the exception of newer neighbourhoods the City predominantly has asbestos cement pipe for sanitary sewers. A conservative value of 0.015 was selected for all city pipes to reflect the age of the system and to account for any unforeseen flow limiting restrictions.

7.6 PUMPING CAPACITY

7.6.1 Minor Lift Stations

There are several minor lift stations located throughout the sewer network. These lift stations are: Hyland, Alsek, 9th Avenue and Fir Street, 11th Avenue and Pine Street, Tamarack Drive, Mountainview Place, and Clyde Wann.

The 1999 Pumphouse and Lift Station Audit report recommended that all pumps at Alsek, 9th Avenue and Fir Street, 11th Avenue and Pine Street and Tamarack Drive be replaced as they are approaching their life expectancy. The study also recommended that the Dogwood infiltration condition be assessed in order to evaluate its impact on the hydraulic capacity of the Clyde Wann lift station. The salient features of these lift stations are presented below.

7.6.1.1 Alsek Lift Station

Alsek Lift station is located in the Riverdale neighbourhood as shown in Figure 7.1.

The station consists of a small structure, which houses electrical and control panel, mounted directly on top a wet well. The wet well contains one 7.5 hp and one 10 hp Flygt submersible pump. The pumps have historically run more frequently during the winter months than the summer due to bleeder operation. The 1999 Pumphouse and Lift Stations Audit report recommended that these pumps be replaced in the near future due to service life.

As there is no as-built information available, an estimate of pumping capacity for the station has been made by assuming that the pumps lift sewage through a 20 m long 100 mm pipe with a static head of 4 m. Based on these assumptions, the capacities of this lift stations were estimated at 28 L/s and 34 L/s when one pump and two pumps are working, respectively.

7.6.1.2 Hyland Lift Station

Hyland Lift station is located in Riverdale neighbourhood as shown in Figure 7.1. The station consists of a steel wet well and separate control kiosk. The station has two-2 hp, single-phase, submersible Flygt pumps. The lift station discharges through a 100 mm 200 m long forcemain. The capacities of the lift station are estimated to be 5 L/s and 6 L/s when one pump and two pumps are working, respectively. The pump characteristics and system curves are graphically presented in Appendix B1.

The station has been problematic due to the fact that only single-phase power is available. The pumps experience frequent pump jamming, as single-phase power does not provide good pump starting power.

7.6.1.3 Tamarack Lift Station

Tamarack Lift station is located in the Porter Creek neighbourhood as shown in Figure 7.1.

The station consists of a small structure, which houses electrical and controls and ventilation equipment, mounted directly on top a wet well. The wet well contains two 9.4 hp Flygt submersible sewage pumps. The lift station discharges through a 150 mm 340 m long forcemain. The capacity of this location is estimated to be 19 L/s and 22 L/s when one pump and two pumps are working, respectively. The pump characteristics and system curves are graphically presented in Appendix B1. The 1999 Pumphouse and Lift Stations Audit report recommended that these pumps be replaced in near future.

7.6.1.4 9th and Fir Lift Station

This station is located near 9th Avenue and Fir Street in the Porter Creek neighbourhood as shown in Figure 7.1.

The station consists of a small structure, which houses electrical and controls and ventilation equipment, mounted directly on top a wet well. The wet well contains two 9.4 hp Flygt submersible sewage pumps. The lift station discharges through a 150 mm 370 m long forcemain. The capacity of this location is 21 L/s and 25 L/s when one pump and two pumps are working, respectively. The 1997 Ground Water Infiltration Study reported a one pump draw down rate of 9.8 L/s. Since there is a significant discrepancy, the pumping capacity should be confirmed. The pump characteristics and system curves are graphically presented in Appendix B1. The 1999 Pumphouse and Lift Stations Audit report recommended that these pumps be replaced in near future.

7.6.1.5 11th and Pine Lift Station

This station is located near 11th Avenue and Pine Street in the Porter Creek neighbourhood as shown in Figure 7.1.

The station consists of a small structure, which houses electrical and controls and ventilation equipment, mounted directly on top a wet well. The wet well contains two 9.4 hp Flygt submersible sewage pumps operating on 208 VAC, 3-phase power. The lift station discharges through a 150 mm 260 m long forcemain. The capacity of this location is 19 L/s and 25 L/s when one pump and two pumps are working, respectively. The pump characteristics and system curves are graphically presented in Appendix B1. The 1999 Pumphouse and Lift Stations Audit report recommended that these pumps be replaced in near future.

The pumping rates and operating head for each of the minor lift stations are presented in the following table:

Table 7.5 Pumping Rate and Operating Heads of Minor Lift Stations

Minor Station	1 Pump Capacity (L/s)	2 Pump Capacity (L/s)	Operating Head (m)
Alsek Lift	28	34	5
Hyland Lift	5	6	5
9th & Fir Lift	21	25	16
11th & Pine Lift	19	25	16
Tamarack Lift	19	22	17

7.6.1.6 Quartz Road Lift Station

Quartz Road Lift is a new station installed in 2002. The lift consists of a Flygt Pre-packaged wet well and control kiosk. A Flygt controller provides control.

7.6.1.7 Mountainview Place Lift

Mountainview Place Lift Station is a new station installed in 2000. The lift consists of a steel wet well and control Kiosk. A Flygt controller provides control.

7.6.2 Major Lift Stations

As indicated in Figure 7.1, the major lift stations in Whitehorse are:

- Lift Station # 1
- Lift Station # 3
- Marwell Lift Station
- Clyde Wann Lift Station.

A brief description of these lift stations is provided as follows:

7.6.2.1 Lift Station # 1

Sewage Lift Station # 1 pumps sewage collected from the Downtown and Riverdale neighbourhoods to Marwell Lift Station. Currently, there are three 35 hp dry pit mounted Flygt submersible pumps. Existing average inflows into the station are approximately 80 l/s with normal daily peaks in the order of 160 L/s. The pump characteristics and system curves are graphically presented in Appendix B1.

The facility was upgraded with PLC control in 2002.

7.6.2.2 Lift Station # 3

Lift Station # 3 is located immediately southeast of the Robert Campbell Bridge on Lewes Boulevard. The station has three 20 hp Flygt submersible pumps installed in a dry well at this location. A Polysonics ultrasonic flow meter located under the bridge abutment structure monitors discharge flow from the station. The lift station pumps sewage into a Downtown trunk through a 350 m long 400 mm forcemain. No problems have been reported with this forcemain. The estimated capacity of the lift station is 100 L/s and 190 L/s when one pump and two pumps are working, respectively. The pump characteristic curves along with the developed system curve are graphically presented in Appendix B1.

The City of Whitehorse operations staff reported that there was a cross–connection from the Lewes Boulevard storm main to the Lewes Boulevard sanitary trunk. The cross-connection was due to a blockage in the Lewes Boulevard storm outfall line and a damaged sanitary manhole channel. The operations crew felt that a majority, if not all, of the storm water might have been entering the sanitary sewer for a number of years.

7.6.2.3 Marwell Lift Station

The Marwell Lift Station is located in the Marwell Industrial Subdivision as shown in Figure 7.1. The station pumps sewage collected from all areas south of Porter Creek to the Livingston Trail Environmental Control Facility. The station contains two 600 hp electric variable frequency drive (VFD) vertical pumps and two 685 hp diesel engine driven vertical pumps. The station collects sewage from the Lift Station # 1 forcemain and from the 600 mm diameter Marwell gravity trunk system.

The system curves were calculated based on the elevation of water in the wet well and the frictional losses in the forcemain. The pump curves are superimposed on a chart with the estimated flow derived from the intersection of the two curves. The flow rates obtained form this analysis was then utilized in the hydraulic model. The pump characteristics and system curves are graphically presented in Appendix B1.

The two diesel pumps provide both electrical peak shaving and standby pumping. The sequencing of the four pumps is VFD lead, diesel lag, diesel standby, then VFD standby. The capacity of the station is 502 L/s when two pumps are running and 350

L/s when one pump is running. The actual pumping rates will vary depending on the total pumping head.

Marwell Lift also has a truck dump facility for commercial and City trucked effluent dumping. A channel grinder was installed in 2000 to reduce effluent solids.

7.6.2.4 Clyde Wann Lift Station

Clyde Wann Lift Station is the largest sewage pumping facility in the Porter Creek area as shown in Figure 7.1. The lift station consists of two-30 hp Smith & Loveless horizontal sewage pumps located in a dry well. The lift station discharges through a 250 mm diameter 240 m long forcemain. The capacity of the station is 63 L/s and 100 L/s when one pump and two pumps are working, respectively. The pump characteristics and system curves are graphically presented in Appendix B1.

Based on information from the operations staff, the pumps at Clyde Wann Lift may be obsolete and are non-matching. It is believed that the pumps were designed to spin in opposite directions in order to fit in a smaller space. As a result, parts may be harder to obtain in the future for a non-standard pump. The wet well capacity is also too small but depends on acceptable supply trunk surcharge level. As was indicated in the 1997 Infiltration study, the inlet main is susceptible to infiltration. The building is susceptible to spring flooding. Upgrading of this facility may be considered from an operational standpoint.

7.6.2.5 Pump Characteristics

The pumping rates and operating head for each of the major lift stations are presented in the following table:

Table 7.7 Pumping Rate and Operating Heads of Major Lift Stations

Lift Station	Pumping Capacity (L/s)	Operating Head (m)
Lift Station # 3	245.0 (3 of 3 pumps)	10.0
Lift Station # 1	315.0 (3 of 3 pumps)	14.0
Marwell	502 L/s (2 of 4 pumps)	90.0
Clyde Wann	100 L/s (2 of 2 pumps)	21.0

7.6.3 Forcemains

Four major forcemains are incorporated into the City of Whitehorse Sewerage System.

Forcemain #1 runs parallel to the Yukon River from the Lift Station # 1 to the Marwell Lift Station. This 2300 m long 560 mm diameter shallow buried line has had no known structural or capacity problems.

Forcemain #2 runs from the Marwell Lift Station into the LTECF. This 7,893 m long forcemain consists of 5195 m of newly installed section and 2698 m of the old steel section with 600 mm diameter. The new section consists of 4285 m of 600 mm diameter ductile iron, 900 m of 700 mm diameter HDPE, and 10 m of 600 mm diameter steel.

Forcemain # 3 runs from Lift Station # 3 across the Yukon River via the Robert Campbell Bridge to the south end of Downtown on Trunk 1. No problems have been reported with this forcemain.

Forcemain # 4 (Gravity pressure) runs from the Porter Creek Flush Tank to the LTECF.

7.6.4 Porter Creek Flush Tank

The Porter Creek Flush Tank receives sewage from the Porter Creek neighbourhood and discharges to the LTECF through a gravity forcemain. The location of this facility is shown in Figure 7.1.

The facility consists of a storage tank on Larch Street in Porter Creek, and an underground valve chamber and control kiosk located down stream on the Lower Porter Creek Bench. The facility controls sewage flow to the LTECF by providing storage tank retention and flushing. The main function of the facility is to provided self-cleansing velocity through the gravity forcemain pipeline. The valve chamber houses a sonic flow meter, a 400 mm Rotork actuated plug valve with a manual bypass, and associated piping. The control valve operates on automatic mode based on the level in the upstream Flush Tank. The tank level and flows are continuously monitored and indicated at the valve chamber control station.

7.7 TREATMENT FACILITIES

There are four wastewater treatment facilities within the City of Whitehorse sanitary system. The facilities are the Livingston Trail Environmental Control Facility (LTECF), Whitehorse Lagoons, Porter Creek Lagoons and the Crestview Lagoons.

7.7.1 Livingston Trail Environmental Control Facility

The sewage treatment facility, the Livingston Trail Environmental Control Facility (LTECF), completed in 1996 was built to replace Whitehorse and Porter Creek Lagoons and treat sewage from all serviced areas except Crestview. The facility consists of two primary non-aerated lagoons with ultimate retention time of fifteen days; four secondary-non-aerated lagoons with ultimate retention time of sixty days and a long term storage impoundment with once per year discharge (1997 Sanitary Sewer Main GWI study) into the pothole lake.

The design for this facility was based on the assumption that the City's sewage generation rate has to be reduced from 828 Lpcd, in 1992 to 570 Lpcd by the year 2012. The facility was designed to fulfil the City's sewerage treatment needs until approximately 2012. The 2002 sewage generation rate was 666 Lpcd. A detailed assessment of this facility was not part of the scope of this study.

Provision has been made for future construction of a fifth secondary treatment cell. Depending on Water License requirements, provision for a discharge to the Yukon River has been identified.

7.7.2 Crestview Lagoons

The Crestview Lagoons are located on the bench northeast of Crestview adjacent to Yukon River and it has been used for sewage disposal from the Crestview area. The lagoon system has two primary treatment cells and a large secondary cell. A second, secondary cell was originally constructed but has never been used. The Crestview lagoon does not require discharge and relays upon evaporation/seepage.

7.7.3 Whitehorse Lagoon

The Whitehorse Lagoons have not been used since the construction of the LTECF and the Marwell Forcemain Extension. The lagoon system is maintained for emergency treatment. Annual top up the lagoon is required in order to maintain effluent cover over the ultra violet sensitive liners.

7.7.4 Porter Creek Lagoon

The Porter Creek Lagoon has not been used to treat effluent since the construction of the Porter Creek Gravity Forcemain and Flush Tank during the LTECF upgrades. The lagoon was initially maintained for emergency treatment and for dumping of sump and catch basin waste by both City and private eductor trucks. The Porter Creek gravity forcemain has been operating without concern. Complete abandonment of the facility may be considered before Lower Porter Creek Bench development.

7.7.5 Porter Creek Gravity Forcemain Operations Lagoon

A small lagoon was constructed as part of the Porter Creek Gravity Forcemain project. The lagoon was intended for operational requirements to flush the Yukon River crossing; however, due to the requirement of having to treat or truck effluent from the lagoon, river crossing flushing operations have never been attempted.

In order to secure against potential river crossing blockage or capacity reduction due to debris build up, a grinder station should be installed at the Porter Creek gravity forcemain flush tank.

8.0 Sewer Model Development and Calibration

8.1 GENERAL

Stantec conducted the analysis of the sewage collection network with respect to line capacity with the aid of a RTSWMM computer model. Input to the model includes pipe information, land use information and sewage flows. The sewage collection network model for the City of Whitehorse consists of three separate model components. One of the components terminates at the Marwell Lift Station, one component terminates at the Porter Creek Flush Tank and the third component terminates at the Crestview Lagoons.

The loading and demand on each pipe was designated to originate from two sources: residential and non-residential. These loadings include domestic and commercial sewage and inflow/infiltration.

8.2 LOADING SCENARIOS

The 1990 Water and Sewer study performed seven loading scenarios on the Riverdale subdivision in order to establish the worst case loading condition. The seven runs were:

1. Peak flow based on Harmon Peaking Factor
2. Average day flow + a 1 in 2 year rainfall event with 10% infiltration
3. Average day flow x 1.25 + a 1 in 2 year rainfall event with 10% infiltration
4. Maximum day flow + a 1 in 2 year rainfall event with 10% infiltration
5. Average day flow + a 1 in 5 year rainfall event with 10% infiltration
6. Average day flow x 1.25 + a 1 in 5 year rainfall event with 10% infiltration
7. Maximum day flow + a 1 in 5 year rainfall event with 10% infiltration

It is noted that 10% infiltration assumes that 10% of the total rainfall over a certain area enters the sanitary sewer system due to inflow. From these seven runs, The 1990 Water and Sewer study established that Scenario 1, the peak flow based on Harmon Peaking Factor, as the worst case followed by Scenario 4 and Scenario 7.

Computing peak flow using the Harmon Peaking factor, which depends on the population only, generally results in peaks of 2 to 3 times of average flow. Using Harmon Peaking Factor alone does not account of wet weather flows. An accurate assessment of contributing wet weather inflow or infiltration could not be determined from available data; therefore, separate dry weather and wet weather analysis was not conducted.

It was decided that simple peaking factors would be utilized for model analysis since good wet weather data was not available. The following two peak flow multipliers were used by this study:

1. Peak flow equals to three times of average flow
2. Peak flow equals to four times of average flow

Experience elsewhere indicates that combined peak dry weather sewage flow and inflow/infiltration during wet weather events is generally 3 to 4 times greater than the average dry weather peak flow.

The City should implement a more detailed and accurate sanitary flow monitoring program. When more accurate flow and rainfall event data becomes available, the City can update inflow and infiltration rates in the sewer model. It should also be noted that more accurate peak flow data is required prior to commitment of infrastructure upgrade.

8.3 MODEL DEVELOPMENT

The hydraulic assessment of the sewage collection system was done using the RT-SWMM computer model developed by Stantec. The RT-SWMM model was developed from open source code provided by the popular United States Environmental Protection Agency SWMM model. The RT-SWMM model provides Arcview GIS interface and improved sanitary analysis.

The model was constructed and calibrated as accurately as was possible with existing data. Flows attributed to residential lands and the non-residential lands such as commercial/industrial lands and public lands were inputted as manhole nodal contributing flow in the sewer model. More detailed flow monitoring will be required in the future to provide more accurate model calibration and analysis.

Each of the three Whitehorse sewer systems, Marwell Contributing Area, Porter Creek and Crestview, were developed as separate models. Each of these models was developed using the assembled data by importing the system inventory databases into the model. The created sewer system schematic was graphically overlayed and compared to the latest system maps. In a similar fashion to the water model, a functional GIS was developed to analyse drainage area delineation, pipe networks, base parcels, air photos, etc.

All major lift stations were modeled from as-built drawings to reflect the physical system. The minor lift stations were not included in the model; however, the capacity of each of the minor lift stations was assessed individually to identify any improvement requirements.

8.4 MODEL CALIBRATION

The model was calibrated against the daily volume of flows measured at various pumping stations and the treatment facilities. Accurate hourly data was not available for this study. Further implementation of SCADA and improved flow monitoring will provide better data. The City should conduct a metering audit to assess existing meter condition and make recommendation for improvements. If accurate pump start and start times and corresponding wet well levels are available, flow volumes can be estimated without a flow meter.

When more detailed data is available, the models can be recalibrated to reflect better understanding of peak flow data.

8.5 HYDRAULIC LOADING RATIO

A Hydraulic Loading Ratio was used to evaluate the performance of the system. A Hydraulic Loading Ratio of less than one indicates that the sanitary trunk has reserve capacity. A Hydraulic Loading Ratios of greater than one indicates that a system is surcharged and might pose a concern.

A sanitary trunk operating under surcharge condition has greater capacity than a non-surcharged trunk. The hydraulic head created by the surcharge causes the trunk to function like a forcemain and drives increased flow through the pipe.

Normally trunk sewers can operate under some degree of surcharge, but a detailed assessment of hydraulic grade lines and building serving would be required to identify a safe operating level for each trunk.

Based on input from the City, a maximum hydraulic loading ratio of 1.2 instead of 1.0 was chosen to flag trunks that require additional flow monitoring and assessment. As discussed in Section 8.2, peak flows were assessed at three and four times that of average flow.

9.0 Existing Sanitary System Hydraulic Evaluation

9.1 GENERAL

The focus of this study was to assess the hydraulic adequacy of the existing sanitary system, identify potential system bottlenecks and assess the impact of future development.

For the purposes of the study, several basic assumptions were made in order to arrive at the upgrading requirements as follows:

- All pipes are assumed to be structurally adequate
- All pumps have a design life greater than the study horizons. Replacement of older or worn out pumps is assumed to be a function of maintenance
- Peak Flow Factors of 3 and 4 will be analysed
- A maximum acceptable hydraulic loading ratio (design flow / capacity) of 1.2 will be used to flag for further flow monitoring and analysis.

Modeling can only be used as a tool to identify areas of concern. Generally pipes do not require upgrading if they are only a small amount over capacity; however, any model result indicating a pipe over capacity should be investigated further due the potential for error in flow monitoring or modeling data. Further, some specific field conditions may limit pipe capacity or be more susceptible to overflow.

9.2 SENSITIVITY CHECK ON INPUT DATA

9.2.1 General

As previously mentioned, the sewage generation rates and pipe information used for the modeling were based on the available pump records, past studies, as-built drawings and on site investigations. Where data was not available, established engineering principles were used to supplement the data in order to analysis the system.

One method of determining the accuracy of a model is to compare model results against know conditions. The City of Whitehorse flow monitoring data is too limited to make and accurate assessment of model accuracy. Based on information from operations staff; some flow meter results may also be suspect. The City currently does not have a structured meter replacement and maintenance program that addresses existing meter installation, condition, accuracy, obsolescence, critical placement or calibration. Some work has been recently undertaken to get non-functioning or improperly functioning meters operating. The metering program needs improvement. Further, the City currently only does a limited amount of portable flow monitoring to investigate suspect lines.

Flow meters at all major facilities should be checked for accuracy and upgraded as required. Data should be recorded at 5 to 15 minute intervals maximum in order to predict peak flows. Both facility inflow and outflow should be monitored to assess upstream and downstream flow conditions. If flow meters are not provided for both inflows and outflows, flows can be estimated with accurate pump start and stop times and corresponding wet well levels.

An annual summer flow-monitoring program should be implemented with temporary flow monitors to assess flow conditions at any trunks with capacity concerns. Accurate rainfall data is also required to assess wet weather flows. Temporary flow monitoring data should be collected at a maximum of 5 to 15 minute intervals.

9.2.2 Lowering of Overall Sewage Flows

The sewage system analysis assumed that the per capita residual sewage flow rates and commercial sewage flow rates would remain constant for the duration of the study period. This assumption does not take into account any City flow reduction initiatives to defer system upgrade and promote better environmental management. In sewer systems where residential water consumption makes up a large portion of sanitary flow, even small reductions in residential demand can make a significant deference.

While bleeder and groundwater infiltration flow may not be as critical to pipe network peak flows as consumption flow and wet weather flow, they generally represent a base demand that impacts lift station operations and treatment volumes. If bleeder flows or infiltration make up a significant portion of total flow through a lift station, the lift station must operate more frequently resulting in higher operations and maintenance costs.

By reducing sanitary flows with demand side management strategies including public awareness campaigns, repair and rehabilitation programs and policy changes, the City could save a significant amount of operating costs and defer capital expenditures.

As was discussed in Section 2.6.10, a very important alternative approach to infrastructure upgrade due to increased water demand and sewage generation is demand side management. The City of Whitehorse should develop a demand management strategy. In addition to water consumption reduction objectives, inflow and infiltration reduction can be added to the strategy to reduce sewage generation rates.

9.3 EXISTING SYSTEM UPGRADING REQUIREMENTS

Modeling results for existing conditions are shown in Figure 9.1 and sewer segments indicating capacity restrictions are presented in Table 9.1.

Table 9.1 Hydraulic Loading on Existing System with Existing Conditions

Location	PIPE	Length (m)	Diameter (m)	Design Capacity (L/s)	Peak Flow = 4 X Average Flow		Peak Flow = 3 X Average Flow	
					Flow (L/s)	Ratio	Flow (L/s)	Ratio
Downtown	2024	100	0.500	103.0	262.0	2.54	134.0	1.3
	2036	85	0.500	107.0	262.0	2.45	139.0	1.3
Riverdale	3039	112	0.460	102.0	130.0	1.27	104.0	1.02
	3038	111	0.460	118.0	144.0	1.22	104.0	0.88
	3154	90	0.250	26.0	34.0	1.31	26.0	1.00

The findings of the existing sewer system hydraulic analysis are described by neighbourhood as follows:

Riverdale Neighbourhood

As identified in Figure 9.1, Trunk 1 in Riverdale has approximately 225 meters of 450 mm pipe with a hydraulic loading ratio greater than 1.2 as was modeled with Peak Flow Factor 4. The same pipes have hydraulic loading ratios of no greater than about 1.0 with Peak Flow Factor 3.

Trunk 2 in Riverdale has a 90 m length of 250 mm pipe on Green Crescent with a hydraulic loading ratio of 1.31 as was modeled with Peak Flow Factor 4. The same pipe has a hydraulic loading ratio 1.0 with Peak Flow Factor 3.

An improved flow-monitoring program should be implemented to establish accurate flow rates through these sewer segments and reassess hydraulic loading conditions.

Trunks 2, 3 and 4 have no capacity concerns.

Downtown

As identified in Figure 9.1, Trunk 1 Downtown has approximately 185 meters of 500 mm pipe with a hydraulic loading ratio of about 2.4 as was modeled with Peak Flow Factor 4. The same pipes have hydraulic loading ratios of no greater than about 1.3 with Peak Flow Factor 3.

It should be noted that peak flows from Lift Station #3 significantly influences the flow through Trunk 1. Additional flow monitoring is required and a review of Lift # 3 operations is recommended in order to assess upgrade requirements.

Trunks 2, 3, 4, 5 and 6 have no existing capacity concerns based on model results.

Hillcrest Neighbourhood

Trunks 1 and 2 have no capacity concerns based on model results.

Granger Neighbourhood

Granger has no capacity concerns based on model results.

McIntyre Neighbourhood

McIntyre Neighbourhood has no capacity concerns based on model results.

Takhini Neighbourhood

Takhini has no capacity concerns based on model results.

Marwell Area

Marwell has no capacity concerns based on model results.

Porter Creek Neighbourhood

Porter Creek has no capacity concerns based on model results.

Crestview Neighbourhood

Crestview has no capacity concerns based on model results.

9.3.1 Minor Lift Stations in the Marwell Lift Contributing Area

The maximum flows to minor lift stations in the Marwell Lift Contributing Area and pumping capacities have been calculated and are presented in the following table.

Table 9.2 Existing Minor Lift Station Flows

Location	Modeled Inflow (L/s)		Capacity (L/s)	
	Peak Flow = 3X Average Flow	Peak Flow = 4X Average Flow	1 Pump	2 Pump
Alsek Lift	13.5	18	28	34
Hyland Lift	3.6	4.8	5	6

The 1 pump capacity at Alsek Lift and Hyland Lift exceeds the modeled peak inflows and does not require upgrade due to capacity.

9.3.2 Major Lift Stations in Marwell Lift Contributing Area

The maximum theoretical flows into the major lift stations and pumping capacity of these lift stations are presented in the following table.

Table 9.3 Flows into Major Lift Stations in the Marwell Lift Contributing Area

Location	Average Inflow (L/s)	Modeled Peak Inflow (L/s)		Calculated Peak Loading Ratio	Capacity (L/s) (Pumps)
		Peak Factor 3	Peak Factor 4		
Lift Station # 3	46.0	129	167	2.8-3.6	190 (2 of 3)
Lift Station # 1	82.0	211	280	2.6-3.4	270 (2 of 3)
Marwell	111.0	285	370	2.6-3.3	502 (2 of 3)

The calculated peak loading ratio at major lift stations ranged from 2.6 to 3.4. Peak flow factors of 3 or 4 was used to model the entire system. Lower peaking factors can be expected at downstream lift stations than from the distribution system since the entire system flow does not peak at the same time due to various system travel times. It should be noted that calculated peaks were established from available data and may be refined when better data is compiled in the future.

None of the lift stations need to be upgraded due to hydraulic requirements; however, if there are any significant local inflow/infiltration problems, structural or operational concerns, replacement or remediation may be considered.

Lift Station # 1 Operational Concerns

The City has reported operational concerns at Lift Station # 1 as follows:

1. The overflow to the river is not in operating condition and may have hydraulic concerns
2. Flow metering is not operational
3. There are wetwell and dry well entry safety issues
4. Station cycle times were reported to be approximately 6 minutes

The requirement for an overflow to the Yukon River needs to be assessed in conjunction with local environmental authorities and is beyond the scope of this study. It may be more desirable to allow a local overflow at station than an overflow to the river. A more detailed assessment of potential overflow impacts needs to be conducted, including overflow to private properties, services and storm systems.

The flow meter at Lift Station #1 should be replaced as part of the Flow Meter Audit and Improvements. Any wet well and dry well entry safety issues need to be addressed but are also beyond the scope of this study.

The hydraulic assessment of this facility identified poor velocities in the forcemain and minimal wet well volumes.

The Forcemain runs parallel to the Yukon River from the Lift Station # 1 to the Marwell Lift Station. This 2300 m long 560 mm diameter shallow buried line has had no known structural or capacity problems. The pump station and forcemain analysis of this location is summarized in the following table.

Table 7.6 Pumping Rates of Lift Station # 1

Pumps Running	Flow (L/s)	Velocity in Forcemain (m/s)
One pump	180	0.76
Two pumps	270	1.13
Three pumps	315	1.32

Existing average inflows into the station are approximately 82 L/s with normal daily peaks on the order of 160 L/s. This flow regime should only activate one pump, which results in low flushing velocities.

The small wetwell volume could be the primary cause of the frequent cycle times. The current wet well volume has been estimated at approximately 6 m^3.

Upon City request, several potential alternatives has been presented to identify and remedy operating issues:

1. Increase operating elevations
2. Increase the wet well size to reduce the pump cycle time
3. Change the operational cycle time by adjusting control sewage levels
4. Install a grinder upstream of the lift station to minimize solids deposit in forcemain
5. Flush / pig the forcemain as a regular maintenance activity

These alternatives are described in more detail as follows:

1. Increase Operating Elevations

Increasing the distance between pump start and pumps stops effectively increases the wet well volume which results in longer pump cycles. The wet well level can be allowed to surcharge into the facility supply trunk as long as it does not overflow upstream customers.

2. Increase the Wet Well Size

Based on the hydraulic assessment at a peak flow of 180 L/s (one pump on) and a cycle time of 20 minutes the station should have an active wet well capacity of 54 m^3. For a 10 minute cycle time, the active wet well volume would be 27 m^3. It is recommended that a minimum cycle time of 10 minutes be maintained in order to minimize wear on mechanical equipment. Construction of a new wet well would be expensive and may be difficult due to local ground water conditions. Consideration may be made to twin the wet well by converting the existing dry well into additional wet well storage. More analysis is required.

3. Change the Operational Cycle Time

The pump control could be programmed to occasionally let the inlet piping surcharge to a safe level. Once this level was obtained two pumps could engage simultaneously inducing flushing velocities of up to 1.13 L/s in the forcemain.

4. Install a Grinder on the Upstream Inlet

Installation of an inline grinder will effectively reduce the particle size of the wastewater reducing the risk of pump jamming and improving effluent partical suspension characteristics. There is limited room within and outside the station to install a grinder. An inline grinder on a new inlet line may be the only solution. Any flooding of the inlet main due to facility operating levels needs to be incorporated into the grander system design.

5. Flush / Pig the Forcemain

The forcemain can be cleansed of any accumulated debris by routinely launching a pigging device. The draw back of this option is that costly pigging facilities would have to be built into the existing forcemain ($200,000 or more) and there is a risk of getting the pig stuck.

Lift Station No 1 Recommendation

The Lift Station No 1 wetwell capacity should be increased. Additional capacity may be available by surcharging the inlet main but an assessment of storage potential is required. Potential debris deposits and periodic flushing velocities may be a concern for the forcemain. Installation of a grinder and control changes to improve flushing frequency is recommended; however, further operational assessment is required to clarify, confirm and remedy all issues. Lift No 3 and Lift No 1 pump sequencing to minimize peak flows should also be included in this assessment.

9.3.3 Porter Creek Sanitary Sewerage System

It is evident from Figure 9.1 that all the modeled sewer segments in Porter Creek have adequate conveyance capacity and there is no need for upgrade with respect to hydraulic capacity. The capacity of each of the lift stations taken from Stantec (1990) and the modeled inflow into these stations are presented in Table 9.4

Table 9.4 Evaluations of Lift Stations in Porter Creek

Location	Modeled Inflow (L/s)		Capacity (L/s)	
	Peak Flow = 3X Average Flow	Peak Flow = 4X Average Flow	1 Pump	2 Pump
9th and Fir	5.0	6.0	21	25
11th and Pine	12.0	16.0	19	25
Tamarack	7.0	9.0	19	22
Clyde Wann	33.0	45.0	63	100

It is evident that the existing pumping facilities in Porter Creek have adequate capacity to handle flows.

The 1990 Water and Sewer Study reported the maximum capacity of 57 L/s for each of Fir and Tamarack lift stations. However, this study computed the maximum capacity of each lift station is only 25 L/s when two pumps are running. The 1997 Ground Water Infiltration study reported even lower one pump wetwell draw down rates. Particularly, 9th and Fir was reported at 9.8 L/s. Although, all report flow rates have adequate capacity, this information should be confirmed. A flow-monitoring program should be implemented to verify inflow rates.

10.0 Future Sanitary Sewer System

10.1 GENERAL

As per the definitions identified in Section 2.5, future is a general time frame between the existing conditions and the ultimate build out capacity. This time frame is left ambiguous for general discussion. Specific timing will be dependent on development phasing.

Ultimate build out capacity refers to total population capacity for all existing and future development areas. Ultimate can also refer to non-tangible items such as demand rates at the time of total system build out.

Based on the growth scenarios discussed in the water sections of this report, the sanitary sewer models were updated to incorporate the future infilling or growth areas.

Under the ultimate build out capacity development condition, the City of Whitehorse significant developments are as follows:

- Tank Farm Expansion – Marwell Lift Contributing Area
- Porter Creek Extension Area – Porter Creek System
- Lower Porter Creek Bench – Porter Creek System
- Riverdale Expansion – Marwell Lift Contributing Area
- Beyond Copper Ridge – Marwell Lift Contributing Area

These locations are shown in Figure 10.1. Possible development phasing was presented in Figure 2.6. The servicing options for future development areas are discussed below.

10.1.1 Tank Farm Expansion

A detailed topographical analysis beyond the scope of this study would be required to determine the most cost effective method of serving the Tank Farm Expansion. There are three alternatives for sanitary servicing.

- Alternative 1 would be to provide servicing to the Hillcrest / Airport sanitary trunk system.
- Alternative 2 would be to provide servicing to the Takhini trunk system via the Alaska Highway.
- Alternative 3 would be to provide servicing to the Takhini trunk system via Hamilton Boulevard.

Alternative 1 would not be in the best interest of the City, as it would impact capacities through the Hillcrest, Airport and Downtown trunk system and capacity at Lift Station #1.

Alternative 1 and 2 may be acceptable but will require a detailed analysis to determine which is most cost effective. Downstream modeling was based on these alternatives.

10.1.2 Porter Creek Extension

Due to topography a lift station will be required to service the Porter Creek Extension area. There are two alternative forcemain discharge locations as follows:

- Alternative 1 would be to discharge to Tamarack Lift Station
- Alternative 2 would be to discharge to Pine Lift Station

Alternative 1 was chosen for analysis as it only impacts one downstream lift station. Alternative 2 impacts both Pine Lift Station and Clyde Wann Lift Stations.

10.1.3 Lower Porter Creek Bench

There is potential for development of a 7,200 person neighbourhood located in the Lower Porter Creek Bench. If the existing Porter Creek gravity force main has adequate capacity at the time of development the following lift station and force main options are available:

- Alternative 1 would be to construct a forcemain that discharges to the Porter Creek Flush Tank
- Alternative 2 would be to construct a forcemain that connects to the existing Porter Creek Gravity Forcemain near the Porter Creek Lagoon
- Alternative 3 would be to construct a force main that connects at the twinned Yukon River crossing of the Porter Creek Gravity Forcemain.

If a forcemain tie in is possible near the Porter Creek Lagoon, the new lift station discharge will have to be timed so that it does not discharge at the same time as the Porter Creek Flush Tank. The City may consider temporarily re-commissioning the Porter Creek lagoon to facilitate construction.

If the Porter Creek Gravity Forcemain does not have adequate capacity at the time of development a forcemain may have to be constructed to the LTECF. Further analysis is required to determine the best servicing option.

10.1.4 Riverdale Expansion Area

The potential Riverdale Expansion development may service a population of 1,500 people. The Riverdale Expansion area is located on the east side of the Yukon River northeast of Downtown. Due to its geographical location, servicing the land via the existing Riverdale subdivision is impractical. Also, all the major lift stations (Lift Station # 3, Lift Station # 1, Marwell Lift Station) and forcemains would need to be upgraded to accommodate additional flows. Three other possible alternatives have been conceptually evaluated to service this area as follows:

- Alternative 1 considered constructing a new lift station and forcemain that discharged upstream of the Marwell Lift Station.
- Alternative 2 considered constructing a new lift station and forcemain that discharged to the existing Marwell forcemain downstream of the Marwell forcemain river crossing.
- Alternative 3 considered the option of constructing a new lift station and forcemain that discharged directly into the LTECF.

Alternative 1 would require Marwell Lift Station and forcemain to be upgraded in order to accommodate the additional flow from the Riverdale Expansion Area.

Alternative 2 would likely ultimately require the upgrade of the Marwell forcemain to accommodate flows but may provide initial Capital deferment. Control would have to be introduced to ensure that the Riverdale Expansion station does not discharge at the same time as Marwell Lift. Additional storage may be required at the Riverdale Expansion lift station to ensure that pumping does not occur during Marwell peak pumping.

Alternative 3 would require the construction of a separate forcemain from the Riverdale Expansion area to the LTECF facility.

10.1.5 Beyond Copper Ridge Area

The study carried out by Yukon Engineering Services in 1999 utilized the concept of servicing the Beyond Copper Ridge area by constructing a sanitary sewer main that tied into the Downtown trunk system. The Yukon Engineering Service option was analysed by the current study but detailed site servicing was considered too far in the future and too conceptual to provide order of magnitude costing.

Since pump station and trunk main infrastructure upgrading to service Beyond Copper Ridge is substantial, a new lagoon system may be an alternative but further investigation is required.

10.2 ULTIMATE SEWAGE VOLUMES

The following table presents the estimated future population and the volume of sewage generated from the City of Whitehorse.

Table 10.1 Generated Daily Sewage Volumes under Ultimate Development Conditions

See Excel sheet

It is seen from the above table that a total population of 41,951 will generate a composite sewage volume of 25,826 m^3 per day. This yields a composite sewage generation rate of approximately 615 Lpcd for the overall City of Whitehorse. Similarly, the composite sewage generation rate of approximately 685 Lpcd for the areas serviced by the Marwell Lift Station can be derived from the above table.

10.3 ULTIMATE UPGRADING OF EXISTING TRUNK NETWORK

Examining the hydraulic modeling results assesses the performance of the existing system under the ultimate build out capacity loading scenario. The hydraulic response of the system is graphically presented in Figure 10.1 and the proposed upgrading is shown in Figure 10.2. The hydraulic adequacy of the system is described in the following sections.

10.3.1 Marwell Lift Contributing Area - Trunk Upgrading

The ultimate modeling results are graphically presented in Figure 10.1. It can be inferred from these figures that several sewer segments located in various locations need to be upgraded to provide the adequate conveyance capacity of the sewer system. The sewer segments with limited capacity are presented in the following tables by neighbourhood:

Table 10.2a Hydraulic Loading on Existing System with Ultimate Development Condition-Downtown Area (Downstream of Lift Station # 3)

Location	Pipe	Length (m)	Diameter (m)	Design capacity (L/s)	Peak Flow = 4Xaverage Flow		Peak Flow = 3Xaverage Flow	
					Flow (L/s)	Ratio	Flow (L/s)	Ratio
Downtown	2024	100	0.500	103.0	262.0	2.58	134.0	1.3
	2036	85	0.500	107.0	262.0	2.47	139.0	1.3
Total Length (m)		185						

Pipe segments 2024 and 2036 were recommended for additional flow monitoring and possible upgrading under existing conditions.

Table 10.2b Hydraulic Loading on Existing System with Ultimate Development Condition- Airport to Downtown

Location	Pipe	Length (m)	Diameter (m)	Design capacity (L/s)	Peak Flow = 4X Average Flow		Peak Flow = 3X Average Flow	
					Flow (L/s)	Ratio	Flow (L/s)	Ratio
Downtown	2156	100	0.350	49.0	160.0	3.26	120.0	2.46
	2157	55	0.350	48.0	160.0	3.26	120.0	2.50
	2158	65	0.350	50.0	160.0	3.20	121.0	2.43
	2170	30	0.350	52.0	136.0	2.61	102.0	1.96
	2171	72	0.350	47.0	136.0	2.89	120.0	2.55
	2172	50	0.350	51.0	138.0	2.70	121.0	2.38
	2175	92	0.350	49.0	143.0	2.91	121.0	2.47
	2183	13	0.350	51.0	144.0	2.82	126.0	2.47
	2184	116	0.350	49.0	144.0	2.94	127.0	2.59
	2185	15	0.380	58.0	144.0	2.48	126.0	2.17
	2186	68	0.500	105.0	144.0	1.37	127.0	1.21
	2187	97	0.500	100.0	147.0	1.47	128.0	1.28
	2178	100	0.500	98.0	148.0	1.51	128.0	1.31
	2164	121	0.525	144.0	160.0	1.11	138.0	0.96
	2165	78	0.525	158.0	191.0	1.21	161.0	1.02
	21681	85	0.525	151.0	191.0	1.27	162.0	1.07
	2168	116	0.525	155.0	191.0	1.24	162.0	1.04
Total Length (m)		1273						

Table 10.2c Hydraulic Loading on Existing System with Ultimate Development Condition-Riverdale Area

Location	Pipe	Length (m)	Diameter (m)	Design capacity (L/s)	Peak Flow = 4 X Average Flow		Peak Flow =3 X Average Flow	
					Flow (L/s)	Ratio	Flow (L/s)	Ratio
Riverdale	3039	112	0.455	102.0	130.0	1.27	104.0	1.02
	3038	111	0.455	118.0	144.0	1.22	104.0	0.88
	3154	90	0.250	26.0	34.0	1.31	26.0	
Total Length (m)		313						

Table 10.2d Hydraulic Loading on Existing System with Ultimate Development Condition-Takhini Area

Location	Pipe	Length (m)	Diameter (m)	Design capacity (L/s)	Peak Flow = 4 X Average Flow		Peak Flow =3 X Average Flow	
					Flow (L/s)	Ratio	Flow (L/s)	Ratio
Takhini	80101	96	0.150	9.0	14.0	1.56	11.0	1.25
	8008	21	0.150	9.0	14.0	1.56	11.0	1.25
Total Length (m)		117						

The computer models are then updated to incorporate the proposed pipe sizes to increase the conveying capacity of these sewer segments and the hydraulic results are reassessed. Figure 10.2 shows the hydraulic loading condition of the sewer systemwith the improvement in place. It is seen that all the modeled pipes are running within the full design capacity. The proposed upgrading is described by neighbourhood as follows:

Downtown - Downstream of Lift Station # 3 Project

Approximately 185 m length of the sewer segments in the Downtown area receiving sewage from the Lift Station # 3 (Table 10.2a) needs to be upgraded to 600 mm in diameter. These segments were recommended for further flow monitoring and possible upgrading under existing conditions.

Airport / Downtown Project

Approximately 1,275 m length of the sewer segments in the Downtown area receiving sewage from the Airport/Hillcrest area (Table 10.2b) needs to be upgraded under the ultimate build out capacity development conditions including the Beyond Copper Ridge Development.

Riverdale Project

Approximately 225 m length of the sewer segments in the Riverdale area (Table 10.2c) needs to be upgraded to 500 mm in diameter to convey flows in the ultimate build out capacity development condition. There is a 90 m length of 250 mm trunk main on Green Crescent that needs to be upgraded. These sewer segments were recommended for further flow monitoring and possible upgrade under existing conditions.

Takhini Project

Approximately 120 m length of the sewer segments in the Takhini area (Table 10.3e) needs to be upgraded to 200 mm in diameter under the ultimate build out capacity development conditions.

10.3.2 Porter Creek Sanitary Sewerage System - Trunk Upgrading

Under the ultimate build out capacity development condition, it is assumed that there will be some infilling and a potential development of Porter Creek Extension area servicing about 1,400 people. The hydraulic loading condition is presented graphically in Figure 10.2. Some of the sewer segments have a hydraulic loading ratio of 1.0 to 1.1. The need for upgrading should be reassessed some time in future by implementing a flow monitoring program. The pumping capacity of Fir, Pine and Clyde Wann Lift Stations are adequate to convey flows under the ultimate build out capacity development conditions.

10.3.3 Crestview Sanitary Sewerage System

As there is no future growth expected in this part of the City other than minor infilling, there are no upgrading requirements.

10.4 UPGRADING REQUIREMENTS OF PUMPING FACILITIES

The Alsek and Hyland Lift Stations are hydraulically adequate to convey flows under the ultimate build out capacity development conditions. However, pumps at these lift stations need to be replaced due to age based on the findings of the 1999 Pumphouse and Lift Stations Audit report.

The hydraulic loading conditions of the three major lift stations, Lift Station # 3, Lift Station # 1, and Marwell Lift Station were assessed to determine the upgrading requirements. Table 10.4 shows the daily volume of inflows into these lift stations under the ultimate build out capacity development conditions with the Beyond Copper Ridge development in place.

Table 10.3 Inflow Volumes to Major Facilities under Ultimate Development Condition

Service Area contributing to	Non-residential Area (ha)	Population	Total Sewage (m^3/day)
Lift Station # 3	49.8	5,451	4,306
Lift Station # 1	184.5	14,398	10,672
Marwell Lift Station	433.6	26,387	18,072
Riverdale Expansion	10.0	15,00	727
Porter Creek Area	88.0	13265	6,649
LTECF	531.6	41,951	25,826

The estimated sewage volume of the Beyond Copper Ridge development is 2,581 m^3/day. If this development does not proceed, the daily total volume of sewage at Lift Station # 1 will be 8,091 m^3/day for a population of 8,398. Consequently, the volume of sewage at the Marwell Lift Station will be reduced to 15,491 m^3 for a population of 20,387.

10.4.1 Ultimate Marwell Contributing Area Lift Stations

Hyland Lift Station does not require ultimate upgrade. Quartz Road, Crow Street and Mountainview Place lifts were not assessed as they are new developments.

The upgrading requirements for the major lift stations in the Marwell Contributing areas are assessed as follows.

Lift Station # 3

The ultimate modeled peak inflow into the Lift Station is approximately 183 L/s and the existing peak inflow is about 167 L/s. As noted in Section 7.6.2.2 there was a significant storm cross-connection that has since been corrected that would have significantly impacted peak flow calculations.

Using the above data, two pumps operating at a combined capacity of 185 L/s will be required to meet peak station inflows. The station has an overall capacity of 245 L/s with three pumps operating. In order to reduce the downstream impact to Lift Station #1 and Marwell Lift station, it is recommended that Lift Station # 3 control system not allow a third pump to operate without operator intervention. A variable frequency drive operating mode may also be adopted to maintain a automatic pumping rate of no greater about 200 L/s.

Lift Station # 1

The peak pumping rate of the Lift Station # 3 influences the inflow into Lift Station # 1. The inflow into the Lift Station # 1 comprises the peak pumping flow of Lift Station # 3 plus daily domestic sewage generated by approximately 8,947 people and non-residential sewage from an area of 135 ha area from Downtown, Airport and Hillcrest areas. Under Ultimate conditions, Lift Station #1 may also include the development of Beyond Copper Ridge servicing a population of 6,000.

The following table presents the ultimate inflow rates into the Lift Station # 1 depending on the number of pumps running at Lift Station # 3 and including the Beyond Copper Ridge development.

Table 10.4 Peak Inflow into Lift Station # 1 under Ultimate Development Condition

Pumps on at Lift #3	Pumping Rate Lift #3 (L/s)	Peak Inflow to Lift #1 Less Lift # 3 Contribution (L/s)		Total Inflow to Lift #1 (L/s)	
		Peak Factor 3	Peak Factor 4	Peak Factor 3	Peak Factor 4
1	100	220	295	320	395
2	185	220	295	405	480
3	245	220	295	465	540

It is evident that the inflow into the Lift Station # 1 and consequently, the pumping rate of this station is significantly influenced by the operation of the Lift Station # 3.

The capacity of the Lift Station # 1 is 280 L/s when two pumps are running and 315 L/s when all the pumps are in operation. Thus it is evident that in order to service the area known as 'Beyond Copper Ridge' upgrading to the Lift Station # 1 is required.

Table 10.5 shows the pumping requirements of the Lift Station # 1 without receiving inflows from the Beyond Copper Ridge development. The local peak inflow from Downtown, Airport and Hillcrest area is then estimated to 175 L/s.

Table 10.5 Peak Inflow into Lift Station # 1 under Ultimate Development Condition without Beyond Copper Ridge

Pumps on at Lift #3	Pumping Rate Lift #3 (L/s)	Peak Inflow to Lift #1 Less Lift # 3 Contribution (L/s)		Total Inflow to Lift #1 (L/s)	
		Peak Factor 3	Peak Factor 4	Peak Factor 3	Peak Factor 4
1	100	130	175	230	275
2	185	130	175	205	360
3	245	130	175	375	420

Even if the pumping capacity of the Lift Station # 3 is maintained at no greater than 190 L/s with variable frequency drives, then there is capacity concern at Lift Station # 1, which can discharge only at 315 L/s (all three pumps running).

The capacity calculation was based on a forcemain Hazen William "C" factor of 100. If the "C" factor were actually 120, the capacity of the pumping facility could be as high as 385 L/s resulting in no upgrade requirement until the beyond Copper Ridge area is developed.

An assessment of the forcemain is required to more accurately estimate the "C" factor and resulting capacity. Lift Station # 1 will ultimately require an additional pump for standby if three duty pumps are required to meet ultimate build out capacity. A new standby pump can be installed in an existing empty slot. If Beyond Copper Ridge is developed, all pumps will require upgrade and a larger wetwell is recommended. An assessment of forcemain adequacy will also be required.

Marwell Lift Station

The peak inflow into this Lift Station is influenced by the peak pumping rate of the Lift Station # 3 and the Lift Station # 1. Under the built out condition, a total population of 26,387 including 6,000 from the Beyond Copper Ridge area and a total of 434 ha of non-residential area my require servicing by this lift station.

The total peak inflow into the Marwell Lift Station will consist of approximately 340 L/s from the Marwell trunk sewer plus the future peak pumping rate from Lift Station # 1. The total peak inflow into the Marwell Lift Station under the ultimate build out capacity including the Beyond Copper Ridge will exceed the existing capacity of 500 L/s.

As noted in Section 7.4.2, the last upgrade of Marwell lift station was based on projected flow data from the Klohn Leonoff / Nova Tec Phase III report with a design capacity to 2012 for servicing a population of 24,000 with a composite sewage rate of 570 Lpcd and a peak flow ratio of 3.2.

Even if Beyond Copper Ridge is excluded from the development plan, the Marwell Lift Station needs to be upgraded to accommodate future growths if the assumed composite sewage generation rate of 685 Lpcd is not reduced to 570 Lpcd.

An expensive option for future consideration is to upgrade Lift Station # 1 to handle Beyond Copper Ridge flows and discharge directly to LTECF instead of Marwell Lift station.

10.4.2 Ultimate Porter Creek Sewage System Lift Stations

The peak inflows into all the pumping facilities in the Porter Creek area under the ultimate build out capacity development conditions are shown in the following table.

Table 10.6 Porter Creek - Evaluation of Lift Stations

Location	Modeled Inflow (L/s)		Capacity (L/s)	
	Peak Flow = 3X Average Flow	Peak Flow = 4X Average Flow	1 Pump	2 Pump
9th and Fir	6.0	8.0	21	25
11th and Pine	16.0	22.0	19	25
Tamarack	29.0	38.0	19	22
Clyde Wann	33.0	45.0	63	100

It is evident that no hydraulic upgrading is required for 9th and Fir, and 11th and Pine lift stations under the ultimate build out capacity development conditions. Tamarack station needs to be upgraded in order to accommodate flow from the 1,400 person Porter Creek Expansion area. The two servicing alternatives were assessed as follows:

- Alternative 1 - Develop land areas that will result in peak flows equal to the total pumping capacity of the Tamarack Lift Station.
- Alternative 2 - Upgrade the lift station to accommodate all potential flows from development area.

Under Alternative 1, the existing inflows into the lift station is about 9 L/s and the maximum pumping capacity is 22 L/s. An additional peak flow of 13 L/s is available. Thus an additional population of about 560 at an average composite rate of 500 Lpcd and a peaking factor of 4 can be serviced.

Alternative 2 results in a peak flow of 38 L/s. The existing lift station needs to be upgraded to accommodate future flows and the downstream sewers will be subject to minor surcharging with a peak flow ratio of 4. The forcemain will have to be checked to ensure it can handle additional capacity.

10.5 SEWER SYSTEM IMPLEMENTATION PLAN

The Implementation Plan for sewer system upgrades outlines Capital timing and priority based on growth projections.

Concept level cost estimates have been provided in the Implementation Plan. More detailed cost estimates are recommended for budgetary purposes.

Since growth centers will trigger specific project priority, only the 0 to 5 Year Implementation Plan has been prioritized based on need or required project sequencing. The Implementation plans are presented in Tables 10.7 to 10.10.

Figure 9.1 indicates existing sewer segments recommended for addition flow monitoring based on calculated flows. Figure 10.1 provides ultimate flow conditions.

Sewer System Implementation Plan 0 to 5 Years

Table 10.7 provides the 0 to 5 Year Implementation Plan for sanitary sewer upgrades.

Table 10.7 Sanitary Sewer System Implementation Plan (0-5 Years)

No.	Project	Item	2003 Cost ($)
1	Flow Meter Audit and Improvements	• Audit and recalibrate existing metering system (Work to be done in conjunction with Water Meter Audit) • Replacement of meters and installation of new meters ($125,000)	125,000
2	Temporary Flow Monitoring	• Purchase equipment for temporary flow monitoring program	50,000
3	Lift Station # 1 Operational Assessment	• Provide a facility and forcemain operational assessment	25,000
4	Lift Station # 1 PLC Programming	• Add logic to PLC to include occasional increased pumping to provide forcemain flushing velocities • Program PLC to calculate wet well inflow and outflow	7,000
5	Water and Sewer Consumption Audit and Demand Management Strategy	• Audit water and sewer consumption by area after better data is available from flow meter upgrades • Update models • Establish Demand Management Strategy to reduce water demand (100,000 included under Water Implementation Plan)	
6	Porter Creek Forcemain Grinder Installation	• Install a grinder at the gravity forcemain flush tank	350,000
7	Lift Station # 1 Grinder Installation	• Install an inline grinder	350,000
		0-5 Year Total	**907,000**

Unconfirmed Projects

The following are possible Capital projects that may or may not be required due to sanitary sewer trunk capacity. Flow monitoring is required to confirm if any upgrade is required and predict possible project timing.

Table 10.8 Unconfirmed Projects

No.	Project	Item	2003 Cost ($)
8	Riverdale Sanitary Trunk 1 Improvements Phase 1	• Construct 225 m of Sanitary Trunk Sewer	225,000
9	Riverdale Sanitary Trunk 1 Improvements Phase 2	• Construct 90 m of Sanitary Trunk Sewer	75,000
10	Downtown Sanitary Trunk 1 Improvements	• Construct 185 m of Sanitary Trunk Sewer	200,000
		Total	**500,000**

Sewer System Implementation Plan 5 to 20 Years

The 5 to 20 Year Implementation Plan is dependent on phasing of the Porter Creek Extension, Tank Farm Expansion, and Lower Porter Creek Bench. Site servicing costs were not assessed, as they would be too conceptual at this stage. Infrastructure upgrade needs and timing will have to be confirmed by flow monitoring.

Table 10.9 Sanitary Sewer System Implementation Plan (5 to 20 Years)

No.	Project	Item	2003 Cost ($)
11	Porter Creek Extension	• Provide site servicing • Construct a lift station and forcemain to Tamarack Lift Station	To be determined
12	Tank Farm Expansion	• Provide site servicing • Construct gravity main to Takhini Sanitary Trunk via Alaska Highway or to Hamilton Boulevard	To be determined
13	Lower Porter Creek Bench	• Provide site servicing • Construct a lift station and forcemain to discharge to Porter Creek Flush Tank, Porter Creek gravity forcemain or Livingston Trail Environmental Control Facility.	To be determined
14	Tamarack Lift Station	• Pump Upgrades	$80,000
15	Takhini Sanitary Trunk Improvements	• Construct 120 m of Sanitary Sewer Trunk (This work will likely be completed under the Takhini area redevelopment $130,000)	
16	Lift Station # 3 – VFD	• Install Variable Frequency Drive and limit pumping to two pumps.	$60,000
17	Lift Station # 1 Standby Pump	• Install Standby Pump	$50,000
		5 to 20 Year Total	**190,000**

Stantec

Sewer System Implementation Plan 20 + Years

The Ultimate Implementation Plan is dependent on phasing of Riverdale Expansion and Beyond Copper Ridge development phasing. Site servicing costs were not assessed, as they would be too conceptual at this stage.

The sewer segments that have restricted capacity are shown in Figure 10.1. Table 10.10 provides the Ultimate Implementation Plan for sanitary sewer upgrades.

Table 10.10 Sanitary Sewer System Implementation Plan (20 + Years)

No.	Project	Item
18	Riverdale Expansion	• Provide site servicing • Construct a lift station and forcemain to discharge to Marwell Lift Station, Marwell Forcemain or Livingston Trail Environmental Control Facility
19	Beyond Copper Ridge	• Provide site servicing • Upgrade or reconstruct Lift Station # 1 and Marwell Lift Station.
20	Airport / Downtown Sanitary Trunk Improvements	• Construct 1,275 m of sanitary sewer trunk depending on Beyond Copper Ridge Development ($1,300,000)
21	Lift Station # 1 Replacement	• Dependent on Beyond Copper Ridge Servicing
22	Marwell Lift Station Twinning	• Can be avoided if composite sewage generation is reduced to 570 Lpcd from 685 Lpcd and Beyond Copper Ridge and Riverdale Expansion are not serviced through Marwell Lift.

11.0 Conclusions and Recommendations – Sanitary Sewer System

11.1 CONCLUSIONS - SANITARY SEWER SYSTEM

The following conclusions are drawn based on the findings of the hydraulic analysis carried out in this study:

Studies and Investigations

1) By reducing sanitary flows with demand side management strategies including public awareness campaigns, repair and rehabilitation programs and policy changes, the City could save a significant amount of operating costs and defer capital expenditures. (Section 9.2.2)

2) Modeling can only be used as a tool to identify areas of concern. Generally pipes do not require upgrading if they are only a small amount over capacity; however, any model result indicating a pipe over capacity should be investigated further due the potential for error in flow monitoring or modeling data. Further, some specific field conditions may limit pipe capacity or be more susceptible to overflow. (Section 9.1)

3) The City of Whitehorse flow monitoring data is too limited to make and accurate assessment of model accuracy and upgrade requirements. Flow meters at all major facilities should be checked for accuracy and upgraded as required. If flow meters are not provided for both inflows and outflows, flows can be estimated with accurate pump start and stop times and corresponding wet well levels. The City also requires more portable flow monitoring to investigate suspect lines. Data should be recorded at 5 to 15 minute intervals maximum in order to predict peak flows. Accurate rainfall data is also required to assess wet weather flows. (Section 9.2.1)

4) The 1997 Ground Water Infiltration Study reported much lower wet well draw down rates for 9th and Fir Lift than was calculated by the current study. The pumping capacity needs to be confirmed and possibly remedied if lower than expected. (Section 7.6.1.4)

5) An operational assessment of Lift Station #1 and forcemain is required to make recommendation with regard to wet well sizing, operating levels and forcemain flushing. (Section 9.3.2)

6) The 1990 Water and Sewer Study reported the maximum capacity of 57 L/s for each of Fir and Tamarack lift stations. However, this study computed the maximum capacity of each lift station is only 25 L/s when two pumps are running. This needs to be confirmed. (Section 9.3.3)

7) The City may also wish to consider an audit of all commercial services to identify type of installation, sanitary cross connections concerns, water cross connection concerns and wastage of water. (Section 7.4.4)

8) The existence and type of each cross connection identified in the 1997 Ground Water Infiltration Study needs to be confirmed. Only a few cross connections were identified on overall storm sewer maps. This information needs to be confirmed and updated. (Section 7.4.4)

System Deficiencies

1. The sanitary sewer system in the Crestview area has adequate hydraulic capacity to convey flows and no upgrading is required. (Section 9.3)

2. The sanitary sewer system in the Porter Creek area has adequate hydraulic capacity under the existing conditions. (Section 9.3)

3. All the lift stations have adequate hydraulic capacity to discharge flows under the existing conditions. (Section 9.3)

4. Based on information from the operations staff, the pumps at Clyde Wann Lift may be obsolete and are non-matching. It is believed that the pumps were designed to spin in opposite directions in order to fit in a smaller space. As a result, parts may be harder to obtain in the future for a non-standard pump. The wet well capacity is also too small but depends on acceptable supply trunk surcharge level. As was indicated in the 1997 Infiltration study, the inlet main is susceptible to infiltration. The building is susceptible to spring flooding. Upgrading of this facility may be considered from an operational standpoint. (Section 7.6.2.4)

5. **Riverdale Neighbourhood** - Trunk 1 in Riverdale has approximately 225 meters of 450 mm pipe with a hydraulic loading ratio greater than 1.2 as was modeled with Peak Flow Factor 4. The same pipes have hydraulic loading ratios of no greater than about 1.0 with Peak Flow Factor 3. Trunk 2 in Riverdale has a 90 m length of 250 mm pipe on Green Crescent with a hydraulic loading ratio of 1.31 as was modeled with Peak Flow Factor 4. The same pipe has a hydraulic loading ratio 1.0 with Peak Flow Factor 3. (Section 9.3)

6. **Downtown Trunk 1** – Trunk 1 Downtown has approximately 185 meters of 500 mm pipe with a hydraulic loading ratio of about 2.4 as was modeled with Peak Flow Factor 4. The same pipes have hydraulic loading ratios of no greater than about 1.3 with Peak Flow Factor 3. (Section 9.3)

7. The wet well at Lift Station #1 is too small. Increasing the distance between pump start and pumps stops effectively increases the wet well volume which results in longer pump cycles. The wet well level can be allowed to surcharge into the facility supply trunk as long as it does not overflow upstream customers. (Section 9.3.2)

8. Installation of an inline grinder will effectively reduce the particle size of the wastewater reducing the risk of pump jamming and improving effluent particle

suspension characteristics. There is limited room within and outside the station to install a grinder. An inline grinder on a new inlet line may be the only solution. Any flooding of the inlet main due to facility operating levels needs to be incorporated into the grander system design. (Section 9.3.2)

Ultimate System Deficiencies

1. An assessment of the Livingston Trail Environmental Control Facility was not part of the study scope. The facility was designed to fulfil the City's sewerage treatment needs until approximately 2012. The design for this facility was based on the assumption that the City's sewage generation rate has to be reduced from 828 Lpcd, in 1992 to 570 Lpcd by the year 2012. The 2002 sewage generation rate was 666 Lpcd. Provision had been made for future construction of a fifth secondary treatment cell. Depending on Water License requirements, provision for a discharge to the Yukon River had also been identified. Requirements for upgrade will be dependent on future water licence requirements, flow generation and treatment effectiveness. (Section 7.7.1)

2. Airport / Downtown Project - Approximately 1,275 m length of the sewer segments in the Downtown area receiving sewage from the Airport/Hillcrest area (Table 10.2b) needs to be upgraded under the ultimate build out capacity development conditions including the Beyond Copper Ridge Development. (Section 10.3.1)

3. Takhini Project - Approximately 120 m length of the sewer segments in the Takhini area (Table 10.3e) needs to be upgraded to 200 mm in diameter under the ultimate build out capacity development conditions. (Section 10.3.1)

4. In order to reduce downstream impact to Lift #1, it is recommended that Lift Station # 3 control system not allow a third pump to operate without operator intervention. Alternately, a variable frequency drive operating mode may also be adopted to maintain a automatic pumping rate of no greater about 200 L/s. (Section 10.4.1)

5. Lift Station # 1 will ultimately require an additional pump for standby if three duty pumps are required to meet ultimate build out capacity. (Section 10.4.1)

6. If Beyond Copper Ridge is developed Marwell Lift Station will require upgrading. Upgrading of Marwell Lift may not be required if Beyound Copper Ridge is not developed and the serviced population is maintained below 24,000 with the composite sewage generation rate of 685 Lpcd reduced to 570 Lpcd. (Section 10.4.1)

7. Tamarack Lift station needs to be upgraded to accommodate future flows from Porter Creek Extension Area and the downstream sewers will be subject to minor surcharging with a peak flow ratio of 4. (Section 10.4.2)

11.2 RECOMMENDATIONS – SANITARY SEWER SYSTEM

The following recommendations are made in this study:

1. A flow monitoring program needs to be implemented to accurately quantify the peak flows and volumes of flows at various locations of the sanitary system.

2. The City should continue their water reduction program to reduce composite sewage inflows to 570 Lpcd.

3. The City should implement the 5-year upgrading plan as outlined in this report.