The Corporation of the
Village of Ashcroft

Village of Ashcroft

Water Master Plan

November 2014

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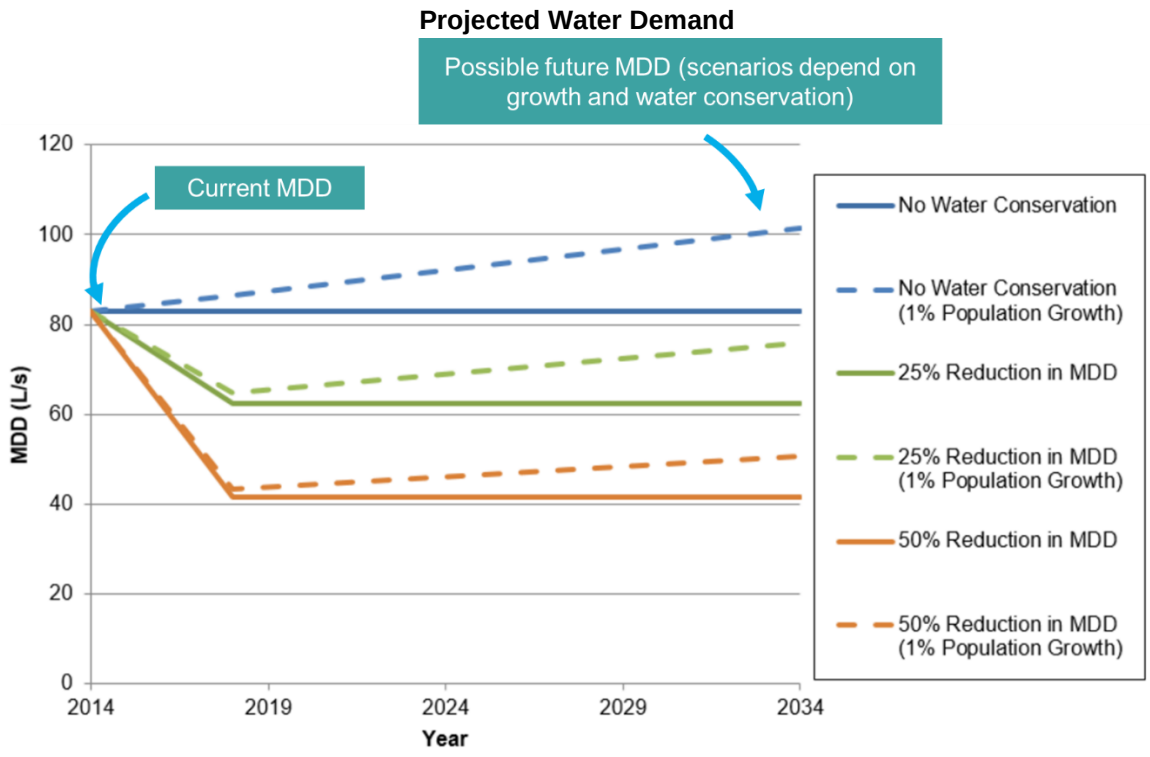
Executive Summary

The Village has worked with Interior Health Authority to identify that the Village's potable water system is worthy of improved water treatment. It is appreciated that the capital and ongoing operations costs associated with such an investment are significant for the Village. While it is essential to determine appropriate treatment considerations and recommended investments, it is important to adopt a holistic approach to considering these investments within the context of the overall, long-term management of the Water Utility. Affordability, long-term investment needs and appropriate asset management activities can be assessed and better accommodated with this additional context.

Population and Related Water Demands

There is some uncertainty associated with future population growth and water use. Based on the per capita demands, projected population growth, and water conservation considerations outlined above, the below figure displays the range of projected future maximum day demands for the Village.



The magnitude of the Village's existing water use indicates that there are a number of opportunities to reduce water use. Since a reduction in MDD will have an impact on capital costs for future treatment infrastructure upgrades, as well as pump station upgrades, it is recommended that the Village select a suitable long-term water conservation factor to account for population growth and water use uncertainties.

It is common practice to supply the MDD within 16 to 20 hours of pumping. The following table outlines the supply and pumping capacity that will be required for some MDD scenarios.

Storage Surplus/Deficiency Summary

MDD Scenario	Pumping Hours	Treatment Plant Capacity (L/s)
83 L/s (Current MDD)	16	125
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An MDD of 125 L/s is conservative and the least optimistic demand as it assumes that minimal water conservation will occur. An MDD of 100 L/s is a reasonable assumption at this stage as it relates to 25% reduction in MDD and also allows for community growth within reasonable pumping best practices. In the short-term, if minimal water conservation is actually realized, MDD can still be supplied with 20 hours of pumping at 100 L/s.

At this stage it is not deemed prudent to assume that the Village would experience more than 25% reduction in the short-term, before the proposed treatment plant commissioning.

Primary Source Review

M. Miles & Associates completed an assessment of the Thompson River in the vicinity of Ashcroft's existing intake and infiltration gallery as part of the Water Master Plan exercise. A number of items were assessed, including the suitability of the existing location in terms of quantity and quality of water, the reliability of the existing location, and whether there is a more suitable location for a future intake if required.

The existing Village primary water intake is in a suitable location on the Thompson River. The original emergency intake, located directly upstream of the primary intake, serves as a suitable backup should the primary intake be out of service due to damage or pump replacement. The infiltration gallery, while not overly productive, does not justify capital expense to repair it. It should, however, be allowed to operate until it no longer produces any water as it does not cost the Village anything to use. Investments in maintenance or rehabilitation are not viewed as worthwhile investments, especially since the two other intakes operate reliably.

Secondary Source Review

The Village relies on the Thompson River as the sole water source for the community. It is common to consider the development of a supplementary source, if practical, in case the primary source is no longer usable.

The risk of not having a secondary water source can be minimized by having a robust primary supply with multiple intakes, maintaining infrastructure in good condition, supplying backup power where appropriate, and having suitable emergency response procedures. The Master Plan involved considering the Bonaparte River and groundwater wells as secondary sources. However, applying the Village's limited funds to identify and develop a secondary water source and related system upgrades, which would likely come at the

expense of treatment and reliability improvements of the Thompson River supply and treatment system, are not deemed to be as high a priority as other water system investments in the next 20 years.

Treating the Thompson River Water

Treating surface water often involves the following the process steps:

1. Filtration - To reduce turbidity and to remove microorganisms and suspended particles from the raw water. This also increases efficacy of UV disinfection and chlorination.
2. UV disinfection - for inactivation of protozoa (*Giardia* and *Cryptosporidium*). This is a second barrier to filtration, and is needed depending on source water quality (i.e. log reduction targets) and the filtration technology used.
3. Chlorination – for both primary disinfection of viruses, and secondary (residual) disinfection to protect the distribution system from microorganisms present or introduced into it.

Review of regulatory requirements and Thompson River raw water quality also results in these treatment stages being employed for the Village's proposed water treatment plant. It is recommended that the plant be located adjacent to the existing pump station and intake system.

Filtration techniques were reviewed with direct filtration being viewed as the leading option. Membrane filtration was also considered but a multi-account evaluation promoted direct filtration. Ultraviolet (UV) and chlorine disinfection are also incorporated into the conceptual cost estimates. A baseline cost of $8.62 million was developed for a 125 L/s design flow. For comparison purposes, a capacity of 100 L/s would result in the treatment plant costing $7.96 million. The 100 L/s cost was applied for capital planning.

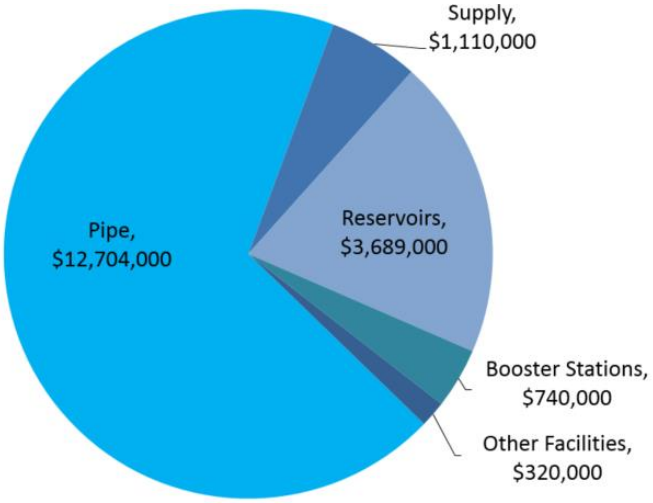
Distribution System and Reservoir Upgrades

Performance of the water system was assessed under normal and fire flow scenarios. From this analysis and related water modeling it was determined that there are four major upgrades for the Village to consider to improve system performance and reliability. These upgrades are presented in Figure ES1.

Replacing Ageing Infrastructure

At a current replacement value of approximately $18.5 million, a substantial investment in water infrastructure has been made. In order to ensure that this investment is maximized, it will be critical that proactive rehabilitation and replacement of assets be undertaken. This will require fiscal resources to be allocated towards maintaining existing levels of service.

Summary of Replacement Values

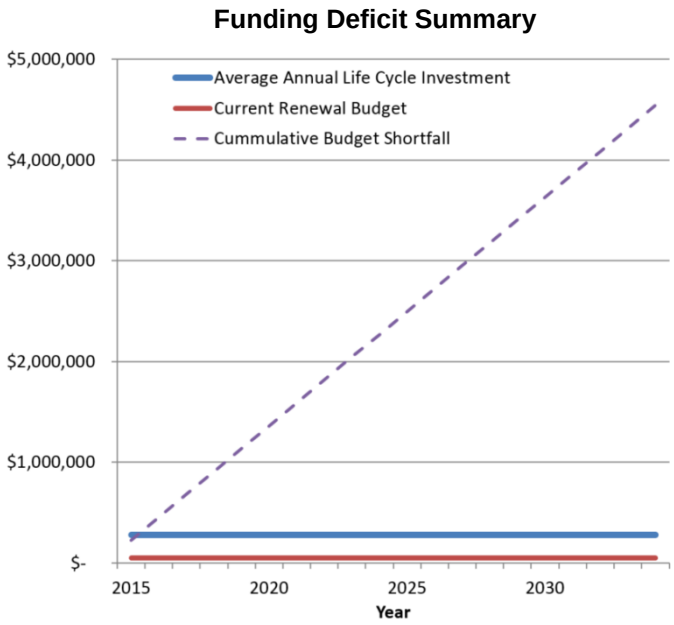


Understanding the Funding Deficit

In order to maintain levels of service of the existing water infrastructure in perpetuity the Village would theoretically need to annually invest approximately $280,000 in capital works.

If it is assumed that the Village would otherwise invest $50,000 in capital replacement, over the next 20 years the budget shortfall will be in the order of $4,500,000.

The funding deficit is significant and will grow substantially if not addressed. The 20 Year Capital Plan and related cash flow model includes investments in replacing infrastructure. A theoretical annual investment was not included in the Capital Plan but rather specific investments have been identified**.**

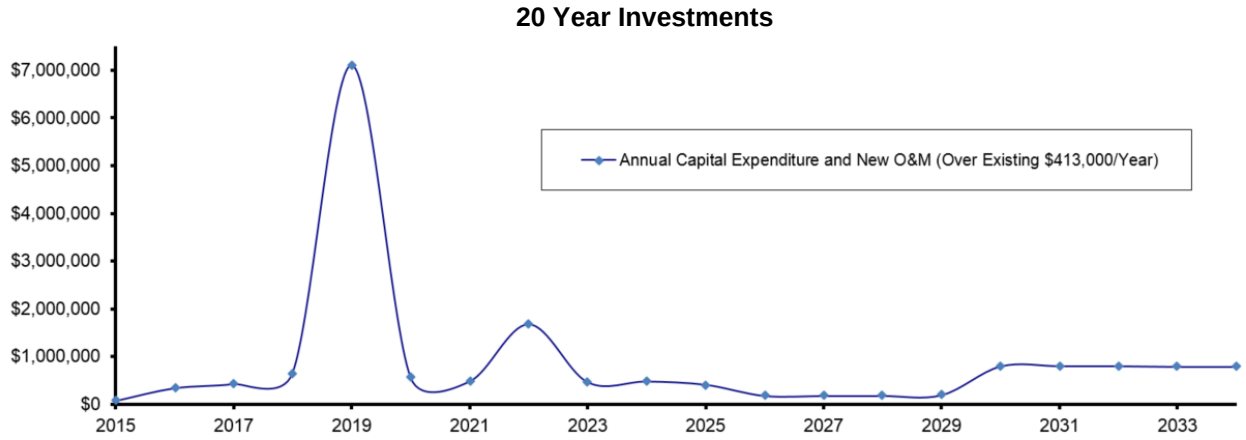


20 Year Capital Plan

The Village is moving towards sustainable financing of its water infrastructure, and has completed a financial analysis to guide investments (capital and operating) over the next 20 years and outlined an approach to achieving long term revenue stability. The timing of capital investments is based on balancing the risks associated with infrastructure failure over the next 20 years with the ability of the Village to raise rates to fund these investments.

Investments will not be limited to construction, repair or replacement of infrastructure. Additional operations and maintenance costs are also significant investments that the Village must consider when making plans. The new treatment plant will increase staffing, energy and chemical costs. There are also recommended actions that are currently not part of the Village's regular operations, such as completing cross connection control and investing in consistent water conservation efforts. The Capital Plan includes these additional items to help outline a more complete investment plan.

The following graph presents a summary of the 20 year investments, with more detail supplied in the Water Master Plan report and supporting appendices. The proposed water treatment plant, with the majority of that capital investment occurring in 2019, represents a significant expenditure in the near term. It is important to note that the timing of the treatment plant could adjust depending on timing of a possible senior government grant.



A detailed, interactive financial model was created to help understand the annual revenues and long term implications of the 20 Year Capital Plan on the long term financial sustainability of the Village's water infrastructure systems. Two primary cash flow scenarios were created for the treatment plant as it represents a significant capital investment. The first scenario assumes no senior government grant funding will be secured. The second scenario includes an assumed 2/3 grant funding for the water treatment plant construction.

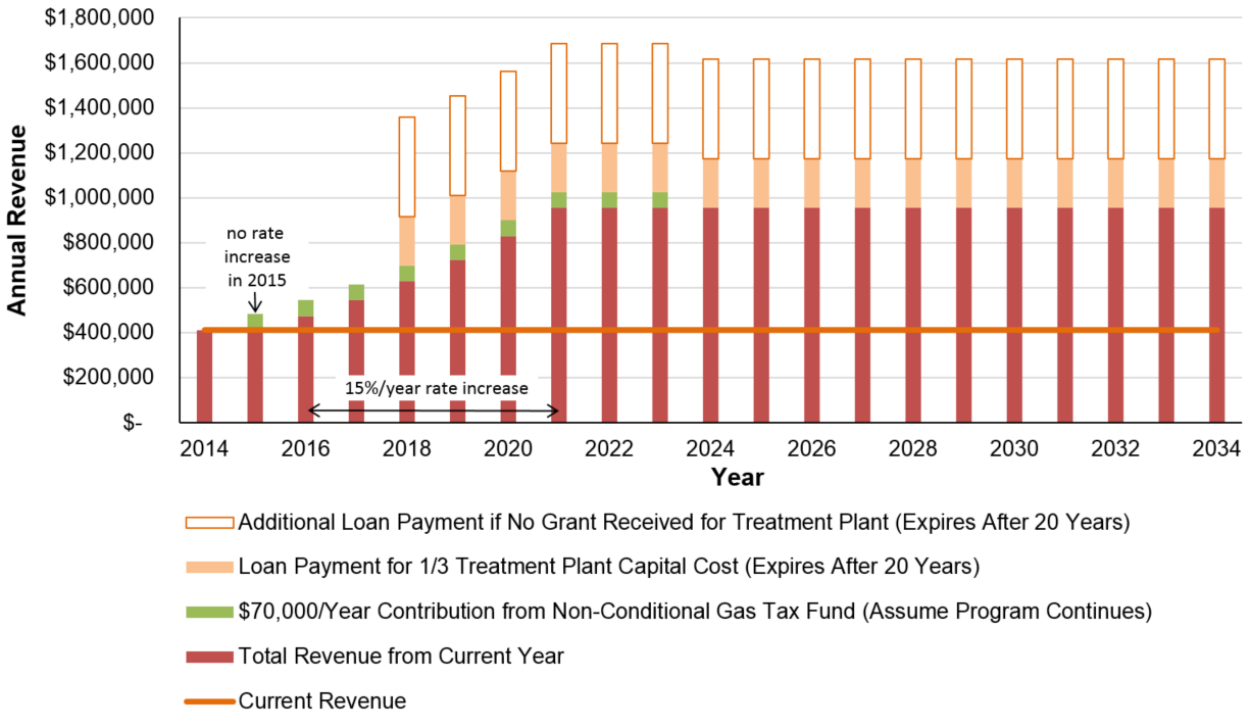
The following matrix that outlines the impact of receiving grant funding for the water treatment plant. The values relate to the increase in annual Water Utility Revenue compared to current rates.

Treatment Plant Capital Funding Scenario	Annual Increase in Revenues Associated with Cash Flow Scenarios *(In Addition to Existing $413,000 per Year)*
Without Grant for Treatment Plant	$1,200,000
With Grant for Treatment Plant	$760,000

For all cash flow scenarios it is assumed that the Utility Rate increase for all costs except for the treatment plant loan would be phased in between 2015 and 2020. The treatment plant loan payments would commence based on the timing of the water treatment plant investments.

The following figure outlines the revenue needed to fund the proposed 20 Year Capital Plan. The Total Revenue from Current Year category in the below graph includes capital replacement, distribution and storage upgrades as well as operations and maintenance costs for the treatment plant and other proposed activities, thus providing a holistic assessment of the Village's long-term Water Utility financial needs.

Summary of Annual Revenue Needs to Fund Proposed Capital Plan and Additional Operations Costs



Discussion and Recommended Next Steps

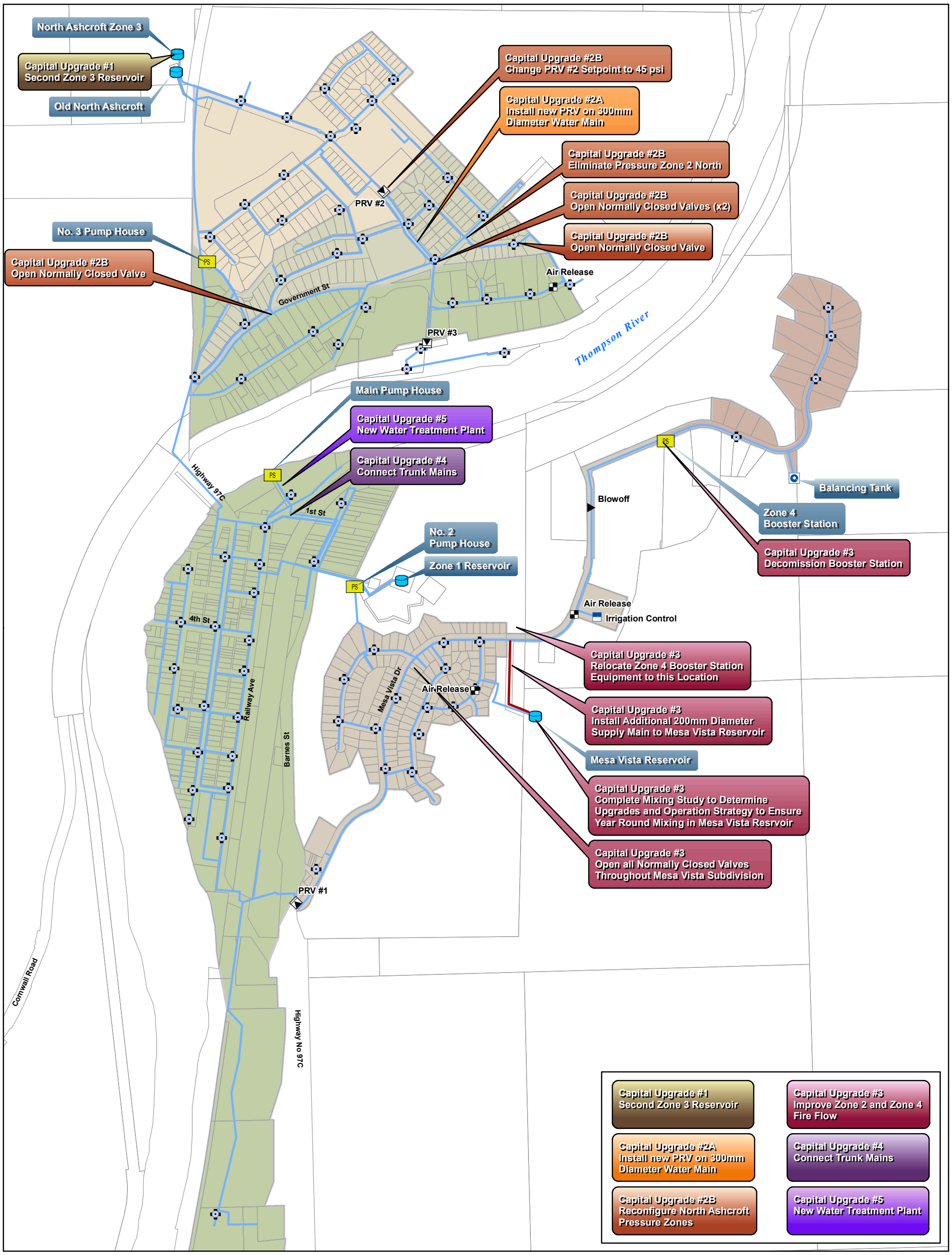
The Master Plan outlines an investment plan for the next 20 years that will allow the Village to provide water that is sustainable for the community. It will do so by achieving:

1. full compliance with existing Interior Health Authority policies;
2. adequate capacity to meet customer demands; and
3. a consistent level of service to all existing customers.

It is recommended that the Village undertake the following next steps to help realize these achievements:

- Complete subsequent water testing and design of the water treatment plant
- Engage the community in conserving water, including considerations for installing water meters
- Communicate with the community to help them understand the need to increase rates
- Engage with senior governments in making application for funding

The Water Master Plan represents a balanced approach, taking into consideration grants and affordable user rates. Without significant senior government grant funding, achieving sustainable financing of infrastructure renewal may not be affordable to Ashcroft residents or business.



Ashcroft
Wellness Awaits You

Water Master Plan

Proposed Major Capital Upgrades

Legend

- Reservoir
- Pump Station
- BalancingTank
- Water Control Facility
- Air Release
- Blowoff
- PRV
- Hydrant
- Pressure Zone 1
- Pressure Zone 2 North
- Pressure Zone 2 South
- Pressure Zone 3
- Pressure Zone 4

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

0.2 0.1 0 0.2
Kilometres

Coordinate System: NAD 1983 UTM Zone 10N

Scale: 1:10,000

Data Sources:
- All infrastructure data composed from original Village of Ashcroft AutoCAD base
-Cadastral received from ICIS

Project #:	1093.0038.01
Author:	JC/CR
Checked:	RC/HT
Status:	FINAL
Revision:	A
Date:	2014 / 11 / 12

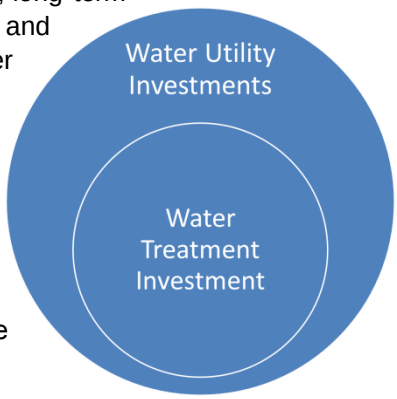
URBAN systems

FIGURE ES1

U:\Projects_KAM\1093\0038\01\D-Drafting-Design-Analysis\GIS\Projects\MXD\Current\Fig5.3-Proposed Major Capital Upgrades.mxd Last updated by jcrosman on November-12-14 at 4:13:23 PM

1.0 Introduction

The Village has worked with Interior Health Authority to identify that the Village's potable water system is worthy of improved water treatment. It is appreciated that the capital and ongoing operations costs associated with such an investment are significant for the Village. While it is essential to determine appropriate treatment considerations and recommended investments, it is important to adopt a holistic approach to considering these investments within the context of the overall, long-term management of the Water Utility. Affordability, long-term investment needs and appropriate asset management activities can be assessed and better accommodated with this additional context.



While previous reviews and studies had periodically been undertaken to address specific issues in parts of the Village's water system, an overall integrated review of all components of the water system had never been undertaken. Through discussions with Village staff and direction of Village Council, it was decided that this document should serve as a long-range planning document for the Village's water system and it would be appropriate to characterize it as a Water Master Plan.

1.1 Need for a Water Master Plan

The need for a long-term comprehensive plan arises out of a number of issues that Village operations staff must deal with on a daily basis. These issues include:

1. **The age of the infrastructure**. Some components of Ashcroft's water system were built more than 50 years ago and are approaching the end of their useful life.
2. **The adequacy of supply and treatment**. The Village experiences Boil Water Advisories that can last many months. These advisories occur annually.
3. **Village demographics**. The Village is not a growing population with anticipated additional major investment. The community is ageing and affordability must be considered as part of developing a realistic approach.
4. **Legislation and Public Health**. Legislation and public health protection protocols in the Province of British Columbia mandate the Village to review its water quality requirements.
5. **Level of Service**. Modern fire protection regulations, water conservation and demand management techniques compel the Village to meet new standards of performance.
6. **Risk Management**. The Village must contend with, and therefore plan for, a number of service loss scenarios, including financial concerns, loss of water supply, rupture of transmission mains, and low fire protection flows.

1.2 Scope

The Master Plan development process is not intended to examine operations and day-to-day repair and maintenance activities. It is recognized that Village staff must cope with very old infrastructure and they currently do their best to keep the system running as efficiently as possible.

The intent of the Plan is to identify those key elements which require updating and improvement in order to provide the required level of service.

1.3 Guiding Principles for the Water Master Plan

- Ensure **sufficient capacity of supply and system components** to accommodate the community.
- Comply with Drinking Water Protection Act by providing **safe drinking water**.
- Take a **long-term, big picture approach** to planning.
- Ensure short-term **improvements support long-term** plan.
- Promote **water conservation**.
- Be **strategic in financing** water system improvements over time.
- **Follow best practices** and principles for managing the water system.

1.4 Water System Overview

The Village's water supply system includes a river infiltration gallery and two surface intakes with submersible pumps that deliver water from the Thompson River to a wet well located at the Main Pump Station. The water is chlorinated using chlorine gas and then it is pumped to the Zone #1 Reservoir using two 200 HP vertical turbine pumps via a dedicated 400 mm diameter water main. Water is then delivered to a number of different pressure zones located throughout the Village. There are a number of valves that remain normally closed to separate the different pressure zones.

Figure 1.1 illustrates the major system components.

1.5 Approach and Methodology

The exercise examined the Village's historical water consumption patterns and develops projections for future demands. A comparison is made of per capita demand with other municipal water systems in the region. It then compares and assesses the Thompson River water source and supply infrastructure and its ability to meet those demands.

The study includes a review of the Village's main water system components, their performance and the need to reinvest or replace them. The review is based on a combination of information sources, including:

- Field observation and interview.
- Hydraulic modelling and analysis.
- Review of previous reports.
- Water sampling and testing.
- Available Provincial government data on river flows and groundwater.

Water quality and public health protection are reviewed in the context of the available options and costs of treatment. The distribution network and storage components are also examined in the context of maintaining the required levels of service.

The last chapters deal with risk management, demand management and discussion of phasing improvements in a fiscally responsible manner. A 20 Year Capital Plan is presented that balances risk, recommended investments and affordability. A cash flow analysis was prepared to help develop that balance.

1.6 Acknowledgements

Village staff were active participants in the process and we wish to thank the following for their timely assistance and advice:

- Michelle Allen, Chief Administrative Officer, for leading with the process and for informing and engaging Village Council before and during the Water Master Plan process
- Brian Bennewith, Foreman, for technical assistance and anecdotal information regarding system operations
- Linda Howika, Director of Finance, for providing financial information for the cash flow model
- Village Council, for engaging in two workshops and aiding with public communications
- Rob Fleming, CPHI (C), Specialist Environmental Health Officer – Large Water Systems Program from Interior Health Authority, for technical advice and guidance during the Master Plan process

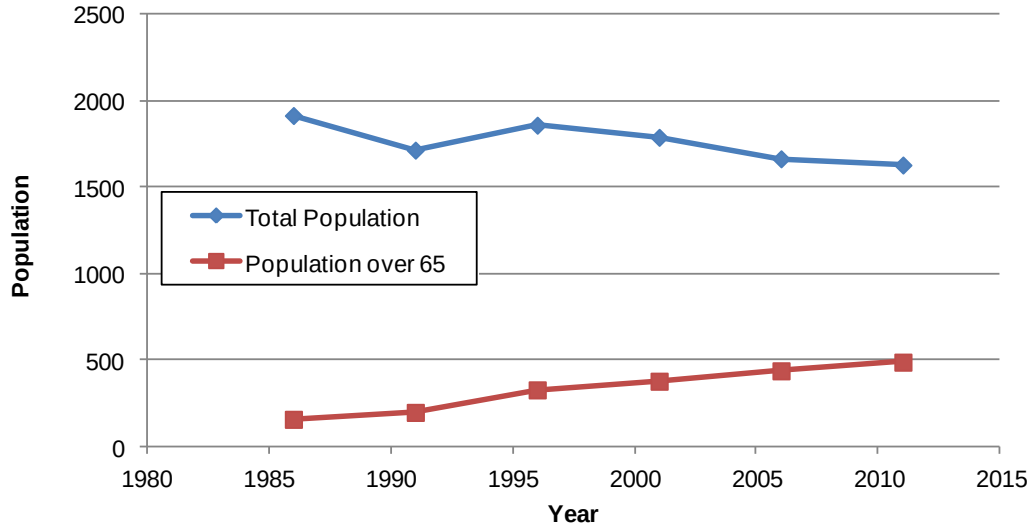
2.0 Water Consumption

2.1 Ashcroft's Population: An Overview

The projected population has a significant impact on the Water Master Plan as it is one of the parameters that dictate the Village's projected water demand. It was not the intent of this planning process to undertake a rigorous analysis of Ashcroft's population and growth patterns. Nevertheless, it is useful to provide as background an overview of historical populations in the Village.

Based on a review of Statistics Canada census data, the Village's current population is 1,628. The historical population data from census data (1981 to 2011) is displayed in Figure 2.1.

Figure 2.1: Village of Ashcroft Historical Population



Between 1981 and 2011 (30 years), the average annual growth rate is calculated to be -1.1%. Furthermore, the Village's population that is over 65 is increasing. Given the population trend over the past 30 years, it is recommended that a population growth factor of 1% be applied for population growth over the next 20 years.[1] With an ageing population affordability is an important consideration.

Assuming that the 2014 population is consistent with the 2011 census data (1,628 people), a growth rate of 1% results in a population of 1,986 by 2034, which represents an increase of 358 people.

For the purpose of water system planning, the populations which appear in the Census are not the only consumers of water. Visitors that occupy hotels/motels, businesses and water consuming industries must be considered, even if these do not appear in the resident census. The concept of an equivalent population can be adopted, and the per capita water consumption can be related to this equivalent population.

[1] An inland port facility in the Village has been proposed. In 2006, it was estimated that the facility could create up to 600 jobs in the Village. This population growth has not been included in the population projections summarized in this report.

2.2 Water Consumption Patterns

The two most commonly used parameters for describing water system demand are average day demand (ADD) and maximum day demand (MDD). ADD is used to represent the overall annual water use and will impact the system operation and maintenance costs. MDD is used to represent the highest daily demand on the water system. MDD has a more significant impact on water system capital costs, as treatment equipment, pump stations, and storage are all sized to meet MDD requirements. Distribution systems (water mains) are sized to deliver MDD and fire flow while maintaining required system pressure.

Design flow rates have been estimated based on historical water use and projected population.

Existing Water Use

Village staff provided flow meter readings from 2009 to 2013 for the water system. Table 3.2 provides a summary of the measured water consumption.

Table 2.1: Historical Water Use Data

Year	Total Annual Demand (m^3/yr)	ADD [2] (L/cap/d)	MDD (L/cap/d)
2002[1]	1,321,810	1,906	7,020
2009	1,208,310	2,033	4,935
2010	1,070,439	1,801	4,832
2011	1,023,803	1,723	4,276
2012	992,108	1,670	4,246
2013	1,049,793	1,767	4,407
Representative Value [3] (2011-2013)	**1,021,901**	**1,720**	**4,410**

***Notes:**

1. 2002 data taken from River Infiltration Gallery Improvements Report, December 2003.
2. Average per capita water use based on population of 1628 people (Canada 2011 Census)
3. A large leak was located and repaired in Desert Hills in the spring of 2011. For this reason, the average was calculated based on the 2011 to 2013 data.

The average ADD and MDD based on 2011 – 2013 water use records is less than that estimated in previous projects. The Village's Water Conservation Plan estimated the 2010 ADD and MDD to be approximately 1,800 L/cap/d and 4,800 L/cap/d, respectively. The reduction in ADD and MDD since 2002 may be due to leak detection and repair programs, as well as public education on water conservation. The further reduction in ADD and MDD observed since 2010 may be attributed to the repair of the leak in Desert Hills.

The Village of Ashcroft's ADD and MDD are considered high, even for a community located in an arid part of the province. The Village's water use could be reduced by water conservation actions and loss reduction programs.

Water Conservation

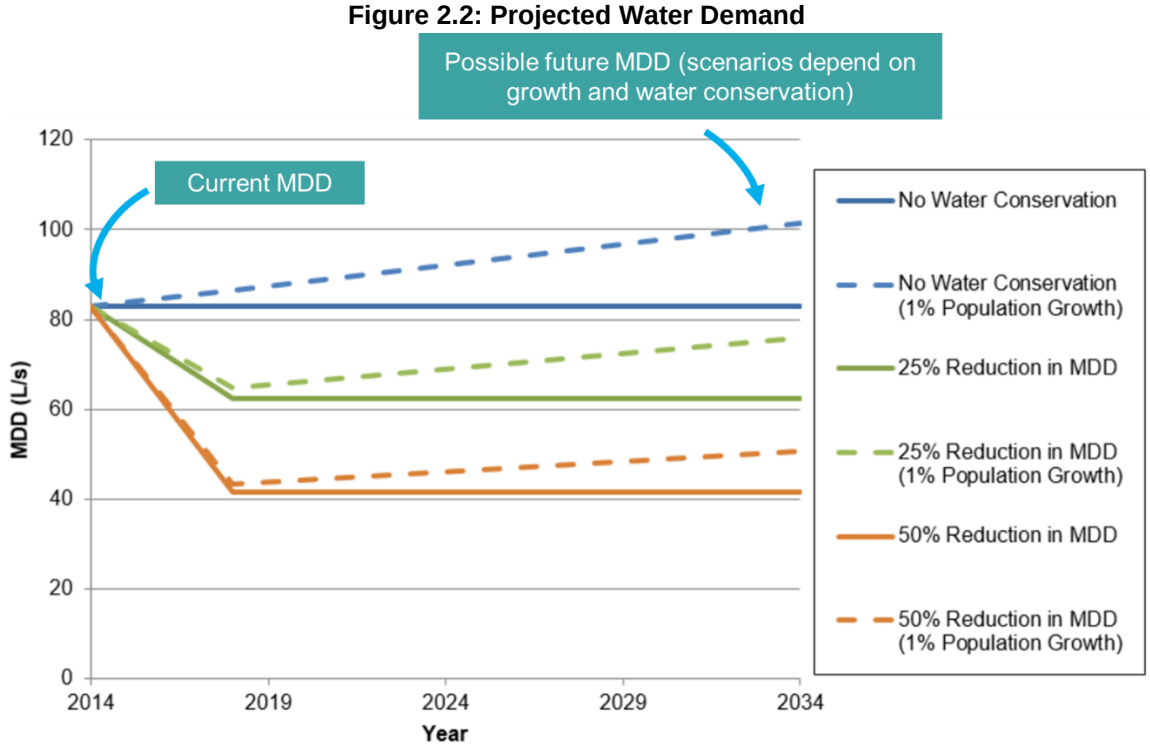
The Village's Water Conservation Plan, which was completed in February 2013, sets a water use reduction target of 10% by 2015 (this means an ADD of 1,620 L/cap/d).

The Water Conservation Plan recommends that the Village selects an ambitious yet achievable water conservation target for 2020 once they have a better understanding of the effectiveness of various water conservation tools. While this is still a desirable plan, a high-level understanding of a long-term water conservation factor is needed for the Water Master Plan.

2.3 Design Flow Rates

There is some uncertainty associated with future population growth and water use. Based on the per capita demands, projected population growth, and water conservation considerations outlined above, Figure 2.2 displays the range of projected future maximum day demands for the Village.

Figure 2.2: Projected Water Demand



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It is common practice to supply the MDD within 16 to 20 hours of pumping, Table 2.2 outlines the supply and pumping capacity that will be required for some MDD scenarios.

An MDD of 125 L/s is conservative and the least optimistic demand as it assumes that minimal water conservation will occur. An MDD of 100 L/s is a reasonable assumption at this stage as it relates to 25% reduction in MDD and also allows for community growth within reasonable pumping best practices. In the short-term, if minimal water conservation is actually realized, MDD can still be supplied with 20 hours of pumping at 100 L/s.

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3.0 Water Source and Intake Location

3.1 Primary Water Source

The Village of Ashcroft currently extracts water from the Thompson River. The Thompson River is the largest tributary of the Fraser River flowing through the south-central portion of British Columbia, Canada. The Thompson River has two main branches called the South Thompson and the North Thompson. The North Thompson originates at the toe of the Thompson Glacier in the Cariboo Mountains west of the community of Valemount and flows generally south to Kamloops and its confluence with the South Thompson. The South Thompson River includes Shuswap Lake and all of its tributaries. The combined Thompson River flows about 15 km to Kamloops Lake, then flows in a meandering course west until it joins the Fraser River in Lytton.

The Village's water intake is approximately 40 km downstream of the outlet of Kamloops Lake. Water quality is good and the yield has been consistent. The Thompson River is a major provincial watercourse that is anticipated to have sufficient capacity to supply drinking water to the Village.

It is important to consider the risk of intake damage and channel geometry associated with the Thompson River supply in order to help ensure that the most appropriate long-term supply and intake location is selected. Potential shortcomings of employing the Thompson River are:

- There is risk of landslides into the river upstream of the Village's intake.
- There is risk of forest fires in the watershed and possible application of fire retardants which may contaminate the water.
- The loss of mature timber may not be a concern for low season runoff. However, the loss can result in increases in peak runoff flows and heavier sediment load.
- Climate change may reduce the river's yield.

To help assess the risk of these shortcomings, M. Miles & Associates completed an assessment of the Thompson River in the vicinity of Ashcroft's existing intake and infiltration gallery as part of the Water Master Plan exercise. A number of items were assessed, including the suitability of the existing location in terms of quantity and quality of water, the reliability of the existing location, and whether there is a more suitable location for a future intake if required.

This section provides a brief summary of the report and its recommendation. The full report is included in Appendix A.

- Section 3 – provides background information regarding the Thompson River and watershed. A couple of salient points include:
  - "Kamloops Lake helps to regulate the flow in the Thompson River and will effectively trap the incoming sediment load from the upstream watershed. However there are extensive deposits of erodible fine-textured sediments downstream of the lake outlet".
  - "It is these fine-textured sediments that adversely affected the performance of the existing infiltration gallery."

- Section 3.2 provides historical streamflow information for the Thompson and Bonaparte Rivers.

- Sections 4.1 and 4.2 – describe the channel near the intake, and reference historical photos of the area. The information indicates that the channel is quite stable in the area of the Ashcroft intakes.
- Section 4.3 – provides calculations of sediment transport/loadings based on river flows, and provides a summary of some available turbidity data for the Thompson and Bonaparte Rivers. The Bonaparte River sediment loadings "are roughly twice as high as those observed on the Thompson River". "Previous experience in the Bonaparte watershed suggests that these comparatively elevated values reflect the lack of upstream lake regulation, the occurrence of fine-textured surficial materials and land use related impacts."
- Section 5 comments on how the assessment relates to the intake and infiltration gallery:

  <u>Intake</u>

  - There does not appear to be any justification for moving the intake to the other side of the Thompson River to attain lower suspended sediment concentrations.
  - There does not appear to be any justification for moving the intake to the Bonaparte River given that there are higher sediment concentrations in the Bonaparte River than the Thompson River.
  - It would be worthwhile investigating an intake location slightly further downstream in a deeper channel if there is concern regarding maintaining sufficient cover/depth over the intake during periods of low river flows. (Note: the existing primary intake is located below the 200 year low water level.)
  - In summary, the report concludes that the existing Village water intakes are in a suitable location on the Thompson River.

  <u>Infiltration Gallery</u>

  - There does not appear any information that would justify moving the infiltration gallery to attain lower suspended sediment concentrations.
  - Given the occurrence of fine-textured surficial materials in the watershed, an infiltration gallery is likely not the best approach for drawing water from the River, in that it will be prone to fouling during freshet and other high turbidity events.
  - However, if Village would like to pursue a new infiltration gallery or rehabilitation of the existing gallery, consideration could be given to moving it slightly downstream to improve system hydraulics/water cover over the gallery.

In summary, the report concludes that the existing Village primary water intake is in a suitable location on the Thompson River. The original emergency intake, located directly upstream of the primary intake, serves as a suitable backup should the primary intake be out of service due to damage or pump replacement. The infiltration gallery, while not overly productive, does not justify capital expense to repair it. It should, however, be allowed to operate until it no longer produces any water as it does not cost the Village anything to use. Investments in maintenance or rehabilitation are not viewed as worthwhile investments, especially since the two other intakes operate reliably.

3.2 Supplementary Source

The Village relies on the Thompson River as the sole water source for the community. It is common to consider the development of a supplementary source, if practical, in case the primary source is no longer usable. The potential of developing of a secondary water source is reviewed briefly here to help provide context and to help promote suitable investment in maintaining the primary river source.

The Village draws water in a good location from the Thompson River and employs two river intakes and continues to obtain a portion of the supply from the infiltration gallery. Therefore the Village has sufficient mechanical back-up, although stand-by power would be beneficial to reduce the risk of loss of supply in the event of an extended power failure.

A secondary source of water should have the ability to supply the demand in the case of catastrophic loss of the primary source (e.g. river contamination) or supplement the primary source should primary source capacity be reduced. The more common design is to provide water for inside use only, and issue a total sprinkling ban during the emergency event.

It should be noted that a secondary source should be capable of providing safe drinking water or else a water quality advisory or boil water advisory would need to be issued.

The M. Miles & Associates review discounted the Bonaparte River as a primary water source due to sediment loadings. It could perhaps be a secondary source if there is a spill in the Thompson River. A significant capital investment would be needed to convey water from the Bonaparte River to the Village water system. Ideally the secondary supply would be connected to the treatment plant. The cost of this supply is likely cost prohibitive, and was therefore not included in the Plan.

BC Water Resources records a total of 31 mapped well locations within 5 km of the Village Centre ranging from test wells to irrigation wells. Just over half of these have recorded well productivity, the highest being 100 USgmp (0.5 ML/d). Based on Urban Systems' understanding of the area, groundwater is highly mineralized, and generally has poor aesthetic quality and low yield.

Developing an emergency groundwater supply would require hydrogeological investigation and test well drilling to determine if a sufficient supply is available. It would require significant capital investment, and could result in a well field arrangement with multiple wells. In addition to capital investment, having a back-up groundwater supply would also incur operation and maintenance costs. Given the poor aesthetic water quality of groundwater wells in the area, it would be unlikely that the Village would want to use wells as a source without additional treatment.

In an emergency situation the Village could haul bulk water from the Village of Cache Creek and/or the District of Logan Lake.

The risk of not having a secondary water source can be minimized by having a robust primary supply with multiple intakes, maintaining infrastructure in good condition, supplying backup power where appropriate, and having suitable emergency response procedures. Applying the Village's limited funds to identify and develop a secondary water source and related system upgrades, which would likely come at the expense of treatment and reliability improvements of the Thompson River supply and treatment system, are not deemed to be as high a priority as other water system investments in the next 20 years.

4.0 Water Quality and Treatment

This section provides a summary of the water quality of the existing system, regulatory context in terms of treatment requirements, an overview of treatment options, and then a summary of a recommended treatment approach and associated costs.

4.1 Thompson River Water Quality

Thompson River water quality is generally very good and has relatively low turbidity, hardness, and alkalinity, and good aesthetic quality.

The Village of Ashcroft undertakes regular water quality monitoring specific to its intakes on the Thompson River, including manual and on-line turbidity analyses of the raw water. This historical information provides an indication of the extent of seasonal variation of the water quality, which is relatively low compared to other rivers but is still prone to some variability. This is attributable to having Kamloops Lake upstream of this location as it acts as a large settling basin.

A water quality sampling program was also developed for this study to further assess the Thompson River water quality and potential treatment requirements. The sampling results are summarized in Table 4.1 and compared to the Guidelines for Canadian Drinking Water Quality (GCDWQ). Please note that the table provides information from the 2014 spring/freshet/summer period, additional information should be collected for system design.

The water quality data shows that in terms of general water chemistry, the water meets the Guidelines for Canadian Drinking Water Quality:

- Aesthetic properties are good and below/within the aesthetic objectives (e.g. pH, hardness);
- Metals are either non-detect or below their respective maximum acceptable concentration (MAC) or aesthetic objective (AO); and
- Disinfection by-products (trihalomethanes and haloacetic acids) are below the maximum acceptable concentration.

More extensive testing would be needed to compare the water quality to the full list of parameters in the GCDWQ, but was not completed due to budget constraints. Parameters such as hydrocarbons, herbicides and pesticides, and endocrine disruptors are very costly to analyze, and may or may not be present at any given point in time. It is generally accepted to test for these parameters infrequently, unless there is a reason to specific that there is a specific source of contamination (e.g. spill or point source) that warrants further investigation.

There are two keys aspects where the Village raw water quality does not meet the Guidelines for Canadian Drinking Water Quality, part of which is currently addressed through chlorination:

1. Turbidity; and
2. Microbiological parameters including protozoa, bacteria and viruses.

Table 4-1 – Thompson River Water Quality

Parameter	*MAC[1]*	*AO[1]*	*Min*	*Max*	*Avg*
Raw Water Field Testing Results [2]					
pH		6.5 - 8.5	8.1	8.9	8.4
Turbidity, NTU	<0.1 [3]		0.59	8.50	1.88
Temperature			2.7	13.7	8.9
UV Transmittance @ 254 nm, cm^{-1}	-	-	69.9	88.3	81.3
Raw Water Lab Testing Results [4]					
Alkalinity, $mgCaCO_3/L$	-	-	35	43	39
Hardness, $mgCaCO_3/L$	-	-	37.4	49.2	43.3
Conductivity, µS/cm	-	-	88	120	104
Colour, TCU		15	6	13	10
Total Iron, mg/L		≤0.3	0.12	0.16	0.14
Total Manganese, mg/L		≤0.05	0.001	0.003	0.002
Total Organic Carbon, mg/L	-	-	3.6	4.5	4.1
Dissolved Organic Carbon, mg/L	-	-	0.25	0.25	0.25
UV Transmittance @ 254 nm, cm^{-1}	-	-	81.5	88.4	85.0
Total Coliforms, CFU/100 mL	nd [5]		6	38	24
E. Coli, CFU, 100 mL	nd [5]		0.5	6.0	2.4
Distribution System Lab Testing Results [6]					
Total Trihalomethanes, mg/L	0.1		0.045	0.057	0.051
Total Haloacetic Acids (HAA5), mg/L	0.08		0.045	0.053	0.049

***Notes:**

1. Maximum Acceptable Concentration (MAC) and aesthetic Objective (AO) from the Guidelines for Drinking Water Quality (GCDWQ)
2. Summary of approximately 80 daily readings, taken from March to June 2014.
3. The GCDWQ for turbidity varies depending on the source and treatment system. For systems with filtration, there are specific guidelines based on the principle that systems be designed and operated to reduce turbidity levels as low as reasonably possible, and strive for a turbidity of <0.1 NTU. It is also recommended that water entering the distribution system have a turbidity of <1.0 NTU.
4. Summary of lab testing taken March and June 2014. Total coliforms and E. Coli monitored weekly from February 26 to June 11, 2014.
5. Note that the sampling was completed on raw water, and the Guideline is none detectable per 100 mL at the outlet of the treatment plant (i.e. post treatment/chlorination). Raw water sampling was completed to provide an indication of bacteriological loading in the raw water. The Village undertakes separate testing of water quality in the distribution system for compliance with the Drinking Water Protection Regulation.
6. Summary of lab testing taken April and June 2014.

4.2 Regulatory Context

This Section provides an overview of legislation that pertains to surface water treatment requirements.

4.2.1 The Drinking Water Protection Act

The Drinking Water Protection Act covers all water systems other than ones that serve individual single-family dwellings and systems excluded through the Drinking Water Projection Regulation. It outlines requirements for water suppliers for ensuring that the water supplied to their users is safe and mandates that suppliers meet any additional requirements established by the Drinking Water Protection Regulation, or by the water supply system's operating permit, as set by the local Drinking Water Officer. In the case of the Village of Ashcroft, the requirements established by the Interior Health Authority Drinking Water Officer must be met.

The Drinking Water Protection Act sets out certain requirements for drinking water operators to ensure the provision of safe drinking water to their customers. In summary, the Act requires:

- The approval of water system construction proposals by Public Health Engineers.
- That water system operators operate their systems in compliance with the requirements of the Act through operating permits that may contain specific conditions and are set and approved by the health authority Drinking Water Officer.
- Water quality monitoring/testing, and specifies water quality standards in the Drinking Water Protection Regulation.
- Water suppliers to have microbiological samples analyzed by a laboratory that has been approved by the Provincial Health Officer.
- Public notification of water quality problems.
- That operators of water systems that serve more than 500 individuals become certified as operators through the Environmental Operators Certification Program.

4.2.2 Water Sustainability Act

The BC Water Sustainability Act replaces the old Water Act, and received royal assent in May 2014. The current Water Act will remain in force over the next year in order to maintain continuity of business. As the new Water Sustainability Act comes into force, the Water Act will be repealed. The earliest date for bringing the Water Sustainability Act into force is expected to be spring 2015, once the regulations supporting the new Act are completed. With the size and complexity of the new Act and the number of proposed regulations, government will implement a phased approach, starting with the priority regulations related to groundwater and water fees and rentals. It is understood that groundwater will be included in the licensing system, and the government’s ability to protect fish and aquatic environments will be expanded. However, as regulations have not been developed or released, it is unclear at this time what approvals will be required under this new Act.

4.2.3 Guidelines for Canadian Drinking Water Quality

Health Canada plays a leadership role in science and research, and protecting public health through the development of the Guidelines for Canadian Drinking Water Quality (GCDWQ). These guidelines are used by every jurisdiction in Canada as the basis for establishing drinking water quality requirements. In some provinces the GCDWQ are directly legislated/mandated. In others, such as BC, the legislation is less prescriptive.

Overall water quality objectives for a water system should be structured to address the following water quality issues:

1. **Microbiological parameters**: target the removal/reduction of protozoa (Giardia and Cryptosporidium), as well as the inactivation of bacteria and viruses.
2. **Chemical parameters**: The GCDWQ sets maximum acceptable concentrations for a variety of chemical, physical and radiological parameters.
3. **Organics and disinfection by-products (DBPs)**: The minimization of disinfection by-products needs to be considered in the selection of a treatment process. DBPs of chlorination include trihalomethanes (THMs) and haloacetic acids (HAAs). The current GCDWQ recommends that total THMs be less than 100 µg/L, and that total HAAs be less than 80 µg/L.
4. **Physical parameters**: The treatment process should produce water with acceptable physical characteristics (turbidity, pH, temperature, colour, taste and odour), so that it does not interfere with disinfection processes, is palatable to consumers, and is stable in the distribution system. The GCDWQ also includes aesthetic objectives.

A multi-barrier treatment approach is considered a best practice, because it is a safer and more reliable way to provide a treatment system than relying on a single process. It is an integrated system of procedures, processes and tools that collectively prevent or reduce the contamination of drinking water in order to reduce risks to public health.

4.2.4 Surface Water Treatment Objectives

In November 2012, the BC Ministry of Health (MOH) issued the “Drinking Water Treatment Objectives (Microbiological) for Surface Water Supplies in British Columbia” to articulate the approach that health authorities have taken over the past few years. These objectives are intended to provide a minimum performance target for water suppliers. Depending on the specific situation and risks identified, a higher level of treatment may be required. The general objectives are summarized as followed and described further in the MOH document (Appendix B):

- 4 Log (99.99%) reduction or inactivation of viruses;
- 3 Log (99.9%) reduction or inactivation of Giardia and Cryptosporidium;
- Two treatment processes for surface water;
- ≤1 NTU turbidity; and
- No detectable total and fecal coliforms and E. Coli.

Section 4.3 (Two Methods of Treatment) of the MOH document provides a summary of the requirements for deferring filtration, and meeting the above treatment objectives using two forms of disinfection (for example, UV disinfection and chlorination). Figure 4.1 presents the results of turbidity sampling of the Village's water supply at the Main Pump Station over the period January 1, 2011 to October 31, 2013. Turbidity is shown to be greater than 1 NTU every year for multiple months. The Thompson River water quality does not meet the turbidity requirements stated in the MOH document. Also, it would be extremely difficult to maintain a watershed control program to minimize fecal contamination in this source.

Figure 4.1: Historical Thompson River Turbidity

(Figure from M. Miles & Associates Report)

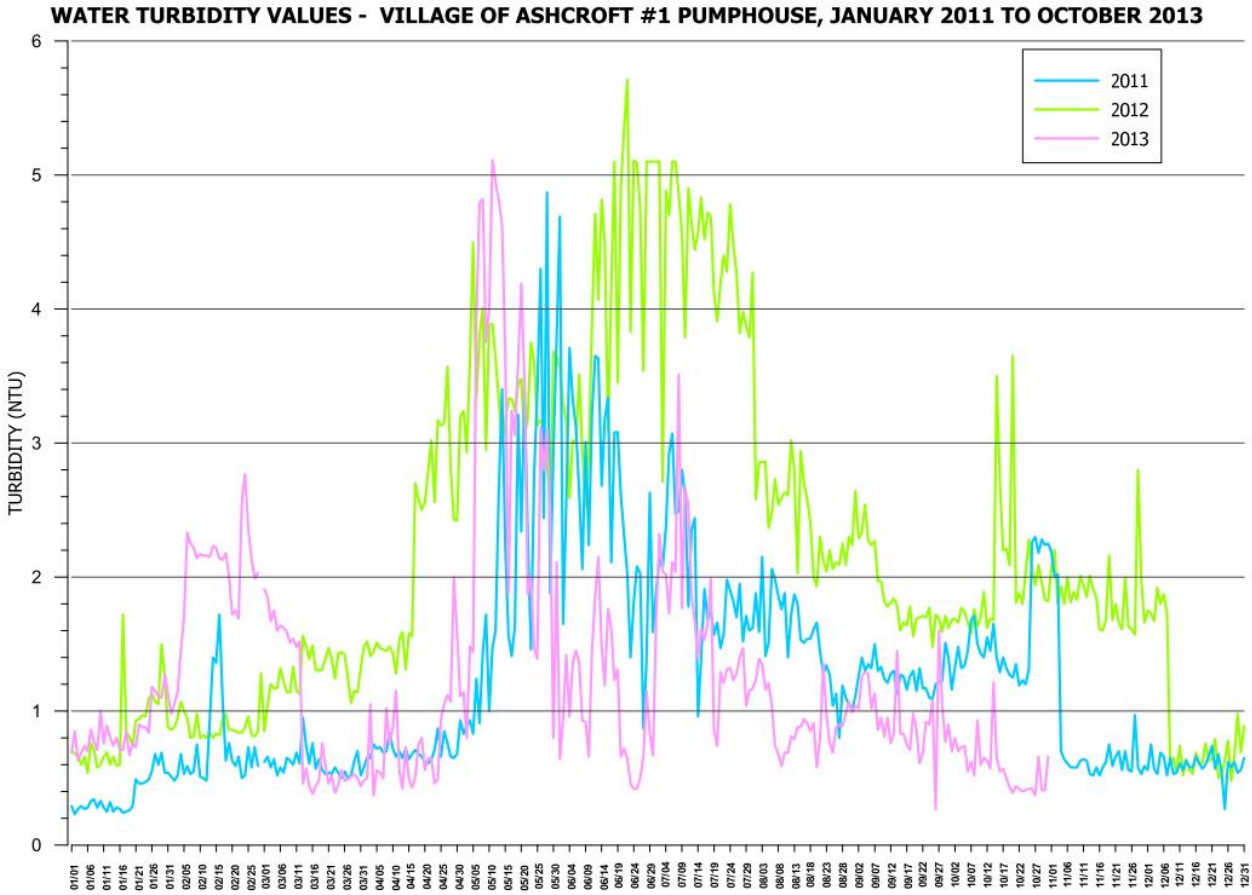

Figure 4.3.2: Water Turbidity Values - Village of Ashcroft #1 Pumphouse, January 1, 2011 to October 31, 2013.

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Therefore treating surface water often involves the following the process steps:

1. Filtration - To reduce turbidity and to remove microorganisms and suspended particles from the raw water. This also increases efficacy of UV disinfection and chlorination.
2. UV disinfection - for inactivation of protozoa (*Giardia* and *Cryptosporidium*). This is a second barrier to filtration, and is needed depending on source water quality (i.e. log reduction targets) and the filtration technology used.
3. Chlorination – for both primary disinfection of viruses, and secondary (residual disinfection) to protect the distribution system from microorganisms present or introduced into it.

Significant water quality monitoring of any new source is required to determine actual treatment requirements.

4.3 Filtration and Disinfection Options

The selection of a treatment train should consider a number of factors, including its suitability for the source water quality, the log credits of the process train compared to legislative requirements, life cycle costs, and other factors. This section provides first a summary of what log credits can be achieved by various processes, then a review of treatment and disinfection options that are most commonly used and likely to be suitable for this system. It is not an exhaustive review of all treatment and disinfection options.

4.3.1 Log Credits of Various Options

The term disinfection refers to the inactivation of micro-organisms by means of adding an oxidant such as chlorine or ozone. Disinfection can also be achieved by ultraviolet light which destroys the microorganism's ability to reproduce. Disinfection does not remove particles, but it can affect other properties of the water (for example, chlorine can be used to oxidize iron and manganese prior to filtration).

Filtration achieves particle removal, and, since many micro-organisms are small particles, filtration plays a role in disinfection. The effectiveness of microorganism reduction is expressed in terms of log credits.

Log credits refer to the effectiveness of removal or reduction of specific microorganisms by each treatment process (i.e. what percent of microorganism is removed or inactivated by each process) Log credits refer to the following percentages:

1-log:	90%
2-log:	99.0%
3-log:	99.9%
4-log:	99.99%

Table 4.2 provides a summary of the Log credits that can generally be achieved for a variety of filtration and disinfection processes (from the USEPA).

Table 4.2: Log Removals of Filtration and Disinfection

	Viruses	Giardia	Cryptosporidium
Filtration			
• Conventional	2.0	3.0	3.0
• Direct	1.0	2.5	2.5
• Slow Sand	2.0	2.5 to 3.0	2.5 to 3.0
• Membrane	no credit	Note 1	Note 1
Disinfection			
• Chlorination	4.0	Note 2	no credit
• Ultra-Violet	no credit to <1.0	Note 3	Note 3

***Notes:**

1. Log credit varies - removal efficiency demonstrated through challenge testing and direct integrity testing.
2. Up to 3 log credit may be granted but requires significant CT.
3. Log credit varies – depends on UV dose and other factors. A log credit of 3.0 can typically be achieved.

4.4 Filtration Techniques

4.4.1 Conventional and Direct Filtration

The most widely used form of filtration for municipal water suppliers is conventional rapid sand filtration. Conventional filtration includes several steps:

a) Application and mixing of a coagulant (usually an Iron or Aluminum - based salt).

b) Coagulation and Flocculation – formations of an easily filterable floc.

c) Clarification – removal of larger flocs.

d) Filtration – most often carried out with dual media beds of coal and sand.

A variation on conventional media filtration leaves out step (c) clarification when source water turbidity is consistently below about 20-50 NTU. This is referred to as Direct Filtration. Both capital and operating costs can be reduced by the use of Direct Filtration, and piloting can be used to confirm that Direct Filtration will achieve the desired quality and appropriate filter loading rates for the water source.

4.4.2 Slow Sand Filtration

Slow Sand Filtration has an even longer history of usage than Conventional Filtration. Slow Sand Filtration does not utilize coagulants; the process involves filtration through sand at a very low loading rate, which encourages the development of a biologically layer (Schmutzdecke) that removes micro-organisms.

This means that filter surface areas must be large and requires a large superstructure, which can make this process cost prohibitive for larger systems. Generally this higher building cost is not

compensated for by the lower operation and maintenance requirements of the system over its lifecycle, given the competitive costs of other packaged filtration systems that have a much smaller footprint

Slow Sand Filtration is therefore not considered further.

4.4.3 Membrane Filtration

Membrane Filtration consists of filtering raw water through a manufactured membrane with extremely small pores (usually less than 0.1 micron). All particles, including microorganisms, larger than the membrane pores, are trapped on the membrane.

Small pore size results in trapping of very fine silt and clay particles and the membranes must be backwashed at very frequent intervals to avoid plugging. Membrane filtration requires a relatively small footprint, but inlet water pressure requirements are relatively high. An alternative form of membrane filtration utilizes a reverse flow pattern with vacuum pumps drawing from the water through the fibers.

Some types of membranes have a pore size that is so small (e.g. Reverse Osmosis) that dissolved parameters can also be removed from the water. Log credits granted for membrane systems are specific to each manufacturer's technology and performance.

Membrane filtration is relatively sophisticated given the need for pretreatment, cleaning of the membranes, and sophisticated controls. However, they provide a firm barrier to microorganisms (if the membranes are intact) which makes membrane systems easier to operate than some systems that require operator input on an on-going basis to ensure good system performance. Also, membranes are relatively expensive and need to be replaced approximately every 8-10 years. This needs to be factored into life-cycle costs.

4.5 Disinfection Techniques

4.5.1 Disinfection using Chlorination

Disinfection by Chlorine does not inactivate *Cryptosporidium*. While it can be used for *Giardia* inactivation, it requires a high CT (i.e. very long contact time and high chlorine dose). Therefore chlorination is not a viable technique for *Giardia* and *Cryptosporidium*.

Chlorination is very effective at inactivation of other microorganisms such as bacteria and viruses. Another benefit of chlorination is that it can be used for secondary disinfection and/or protection of the distribution system. It is recommended that a chlorine residual of at least 0.2 mg/L be maintained throughout the distribution system to prevent the growth of microorganisms/biofilms, and oxidize chemicals or microorganisms if they are introduced into the distribution system (e.g. cross-connection, water main break, vandalism).

The Village has indicated that they would like to consider upgrading their current chlorine gas system to a sodium hypochlorite system. Sodium hypochlorite systems are safer for both the public and operators, and are simpler to operate than chlorine gas systems.

Sodium hypochlorite is typically supplied at concentrations of 6% to 12%. A 12% solution would likely be more suitable for this system in order to minimize the footprint required for chemical storage. Higher concentrations are also less expensive to supply/transport. The best product to use should be confirmed during the system design.

The Village has also expressed interest in on-site generation of sodium hypochlorite. On-site generation has several benefits, including reduced chemical storage space requirements, eliminated transportation risks, and improved operator safety. This type of system is more costly from a capital perspective, but could be evaluated further during predesign as it may be a good option for the Village.

4.5.2 Ultraviolet Disinfection

Irradiation with Ultraviolet (UV) disinfection has been proven to inactivate both *Giardia* and *Cryptosporidium* The log credit granted to UV for Giardia and Cryptosporidium depends on the UV dose achieved by the specific technology/manufacturer (through system validation).

Unlike chemical disinfectants, UV leaves no residual and is not known to create disinfection by-products.

The effectiveness of UV disinfection depends largely of the water's UV Transmittance (UVT). The samples in Thompson River indicate a variable UVT. However, UV would be implemented post-filtration; therefore a higher UVT would be expected. Further water quality review is needed during system predesign to confirm the UV system sizing. This can have significant impact on capital and operating costs and is critical to the system design.

Other water quality parameters such as iron and hardness can affect UV system performance as they can lead to lamp scaling, and should be considered during predesign.

It should be notes that UV disinfection may be used to reduce viruses in water, but the effectiveness of UV varies depend on the type of virus, and some are more resistant and require a high UV dose. Therefore it is generally accepted practice to use chlorination for virus inactivation rather than UV.

4.6 Best Apparent Options

The initial overview reveals the following best apparent options for filtration and disinfection that merit more detailed evaluation:

1. Direct Filtration with UV disinfection and chlorination.
2. Membrane Filtration UV disinfection and chlorination.

Option 2 may not need UV disinfection to provide the log credits required for *Giardia* and *Cryptosporidium*; however it is a relatively inexpensive second barrier for protozoa inactivation.

Both of these filtration options will produce process residual water (i.e. backwash and other residual water) that will require disposal. It is assumed that this water will require some pretreatment within the water treatment plant, and that the water will then be discharged to the River. This will require environmental review and approvals.

For both options, chlorination is required at the end of the process to provide primary disinfection of viruses, and ensure a minimum chlorine residual is maintained in the distribution network.

4.7 Source and Treatment Comparisons of Selected Shortlist Options

Water supply and treatment capital costs are dependent on many criteria including:

- Complexity of system and site conditions;
- Environmental approval requirements;
- Land ownership/acquisition costs; and
- Constructability costs.

Some options may be more favorable than others in terms of initial capital costs.

A detailed cost estimate has been prepared (Section 4.11) and includes site development, building and other required mechanical equipment. However, capital costs are just one aspect of decision-making. Other aspects that warrant consideration include operation and maintenance costs, risk of not providing safe drinking water, potential for environmental impacts, reliability, and other considerations. Table 4.3 is an option comparison matrix that has been developed for the Village's consideration and review, to aid in the selection of the preferred option.

The table was completed by comparing the two filtration options, where a positive number reflects an advantage compared to the other option, and a negative number is worse than the other option.

Based on the comparison matrix, a direct filtration system, with UV disinfection and chlorination is the best long-term option. However, both options are similar, and further analysis could also be completed to compare options; and confirm the treatment approach once additional investigation has been completed. The purpose of the current work has been to provide a reasonable cost estimate for system improvements that can be used for long-term master planning.

Table 4.3: Option Comparison Matrix

Criteria	Filtration Options	
Filtration:	Direct Media	Membrane (Micro or Ultra)
Social Cost/Benefit		
a. Known, proven treatment process	0	0
b. Improvement to Public Health	0	0
c. More operator input required for optimal treatment performance	-0.5	0
d. Better aesthetic water quality	0	0
e. Infrastructure reliability	0	0
f. Requires pilot testing	0.5	0
g. Faster timeframe for Implementation	0	0
Subtotal:	*0*	*0*
Environmental Cost/Benefit		
a. Lower potential impacts on water resources	0	0
b. Lower potential impacts on sensitive habitat	0	0
c. Lower potential impacts from process residuals	0	0
d. Lower chemical use	0	0
e. Lower energy use	0.5	0
Subtotal:	*0.5*	*0*
Financial Cost/Benefit		
a. Estimated Project Cost		
Land acquisition	0	0
Environmental Cost/Benefit	0	0
Filtration	1	0
UV/Chlorination	0	0.5
b. Estimated Annual O & M Costs	0	0
c. Media/Membrane Replacement	2	0
d. Ability to phase upgrades	0	0
Subtotal:	3	0.5
Overall Totals:	**3.5**	**0.5**

4.8 Facility Siting Options

Since the Village owns sufficient land near the Main Pump Station for construction of a filtration facility, this site will be adopted as the longer-term preferred site for treatment. However, the overall layout of the facility should be considered in order to minimize the impact on the adjacent hotel parking and access as well as the municipal campground.

Figure 4.2 depicts the site and a potential allocation for a filtration facility. This location is above the 200 year flood level, and the river morphology review by M. Miles & Associates identified that the pump station area is situated along a stable river channel area.

Figure 4.2: Overview of Pump Station and Treatment Plant Area

The water system includes a dedicated water main between the Main Pump Station and the Zone 1 Reservoir. Therefore, the filtration plant could theoretically be installed along that trunk main route. While an in-depth analysis of topography, property acquisition or potential layout options were not undertaken at this master planning stage, it is reasonable to assume that properly organizing the site beside the Main Pump Station to accommodate the treatment plant and surrounding activities is a more practical solution than developing a new site along the trunk main route. However, further review of technical issues and public consultation should be undertaken during predesign to confirm this approach and the final layout.

4.9 Staffing

The Village elected to include the assumption that operation of the new treatment plant will require an additional staff member. It is beyond the scope of this Master Plan to assess if staff role adjustments could alleviate the need to hire an additional certified operator. Cost estimates include an estimate of salary and benefits associated with employing the new operator.

4.10 Recommended Improvements

For the purposes of preparing a cost estimate for the recommended improvements, it has been assumed that the water treatment plant will include direct filtration with UV disinfection and chlorination.

A number of items need to be assessed during system predesign. For the purposes of the cost estimate, the following has been assumed that:

- The treatment system will be located at the River site;

- The process train will involve pumping from the River intake pumps to a new water treatment plant where the water would then flow by gravity through the water treatment plant (flocculation tanks, filters, UV units, and chlorine dosing point) to the clearwell;
- The existing River intake pumps would need to be replaced, but not the intake structures;
- The existing Main Pump Station would be retrofitted, and used as the clearwell and high lift to the distribution system (existing pumps retained);
- It may be beneficial to also retrofit the existing Main Pump Station to include the UV disinfection and chlorination system at this location;
- A packaged gravity filtration system with stainless steel tanks will be used rather than a custom-designed system;
- Process residuals (e.g. backwash water) can be discharged to the River after pretreatment for solids reduction;
- The chlorine gas system will be converted to sodium hypochlorite;
- CT for virus inactivation using chlorine will occur in the dedicated main to Zone 1 Reservoir;
- Stand-by power will be included; and
- Electrical and control systems will be updated, and simple SCADA will be included.

These assumptions need to be verified, particularly relating to the use and retrofit of the Main Pump Station.

4.11 Cost Estimates and Assumptions

Because this is a high-level analysis, site-specific costs are difficult to determine and therefore, subsequent study (e.g. geotechnical investigation) is required to estimate costs more precisely. As the Village will be drawing from the existing river intakes, it is anticipated that environmental approvals will be limited to building site reviews and approvals for the discharge of process residual water to the river.

Land acquisition costs have not been included in the cost estimates as it has been assumed that work would either be completed on Village property or on existing easements/right-of-ways.

Table 4.4 summarizes the capital cost for constructing direct filtration, UV disinfection and chlorination treatment system with both a 125 L/s and a 100 L/s capacity. For comparison purposes, it is expected that employing membrane filtration will result in a higher capital cost, and higher life-cycle cost due to the cost and frequency membrane replacement, as described in Section 4.7.

The cost estimate is in 2015 Canadian dollars, and does not include HST, interim financing, Village Administration, inflation or special architecture. Appendix C includes a detailed capital cost estimate.

Table 4.5 outlines the increased operations and maintenance costs, compared to the Village's existing costs, for direct filtration, UV disinfection and chlorination. This table summarizes new/increased costs (e.g. does not include intake and high lift pumping costs). These costs are in 2015 Canadian dollars and have been estimated for an average day demand of 40 L/s (i.e. for an MDD of 100 L/s with a 2.5 MDD:ADD peaking factor). Appendix C includes a more detailed estimate.

Table 4.4: Capital Cost Estimates for Treatment Plant Options

Description	Plant Capacity:	125 L/s *Costs*	100 L/s *Costs*
1) Conceptual Design		$50,000	$50,000
2) Pilot Testing & Predesign		$260,000	$260,000
3) Detailed Design & Tendering		$600,000	$600,000
4) Construction			
0.1	General Requirements	$200,000	$185,000
0.2	General Site Work	$500,000	$460,000
0.3	Site Piping	$250,000	$250,000
0.4	River Intake Pumps	$50,000	$40,000
0.5	Building	$1,250,000	$1,090,000
0.6	Rapid Sand Filtration	$1,400,000	$1,220,000
0.7	Water Quality Monitoring Equipment	$50,000	$50,000
0.8	Process Piping & Valving	$300,000	$300,000
0.9	Chlorination System	$50,000	$50,000
0.10	UV Disinfection	$250,000	$220,000
0.11	Electrical & Controls, SCADA	$900,000	$860,000
0.12	Standby Power	$160,000	$160,000
0.13	Retrofit Existing Main Pump Station	$200,000	$200,000
0.14	Solids Handling	$200,000	$170,000
0.15	Uni-directional Flushing of Distribution System	$60,000	$60,000
0.16	Engineering - Construction & Post Construction	$350,000	$325,000
	Subtotal Construction:	$6,170,000	$5,640,000
	Construction Contingency (20%):	$1,234,000	$1,128,000
	PST (5%)	$308,500	$282,000
	TOTAL (rounded)	**$8,620,000**	**$7,960,000**

Table 4.5: Annual Operations and Maintenance Estimate (Increase Over Current Costs)

Description	Estimate ($/year)
1) Chemical Systems	$37,600
2) Main Pumps	per current
3) UV Disinfection	$18,000
4) Filtration	$1,500
5) Water Quality Testing	per current
6) General Maintenance Labour	$90,500
7) Miscellaneous	$5,700
Total	**$157,800**

4.12 Anticipated Permits and Approvals

The following list provides a general review of permits and approvals that may be required for the water treatment plant approvals. This list should be reviewed as the project progresses to assess whether legislative changes have impacted project requirements.

BC Water Act (or Water Sustainability Act) / BC Environmental Management Act	Permit may be required for discharge of process residual water from the water treatment plant to the Thompson River. This process may include referral to the Department of Fisheries and Oceans Canada, and/or Environment Canada.
BC Drinking Water Protection Act	Construction Permit, and amendment of Operating Permit will be required.
BC Heritage Conservation Act	Archaeological Overview Assessment should be completed by professional archaeologist for projects involving excavation or land-altering activities. If this reveals that archaeological sites may be present, then an Archaeological Impact Assessment may be recommended. Activities within the boundary of a recorded archaeological site require a Section 12 permit. May require First Nation consultation.
BC Land Act	Crown Tenure for works below the high water mark of the Thompson River may be required.
Federal Navigation Protection Act	Works must meet legal requirements in the Minor Works Order.
Federal Migratory Birds Convention Act	No official permit required, but any construction activities (land clearing) taking place during nesting season of migratory birds should be assessed by an environmental professional.
Federal Fisheries Act	Request for Review application may be required for work near the Thompson River.
Other Agency permits	Predesign required to determine whether approval to other agencies required for utilities (e.g. Telus, BC Hydro, Terasen).

5.0 Distribution System and Storage Upgrades

While Section 6 of this Master Plan outlines capital investments associated with replacing infrastructure, the Village recognizes that investments are also necessary to improve fire flow, system reliability and storage capacity.

5.1 Performance Standards for Normal Operations

Normal operations are defined as all times when emergency operations, such as fire flow events, are not occurring. The following performance standards for normal operations are based on recommendations provided in the Master Municipal Construction Document (MMCD) Design Guideline Manual (2005).

Maximum System Pressure

A maximum water pressure of 850 kPa (123 psi) is recommended. However, where the maximum pressure exceeds 515 kPa (75 psi), service connections must be individually protected by pressure reducing valves located in the buildings being served.

It is worth noting that water pressures in excess of 515 kPa increase stress on plumbing fixtures and fittings, and that any leaks that may arise are exasperated by higher pressures.

Minimum System Pressure

Under the Peak Hour demand scenario, the minimum residual pressure which should be maintained at street level is 300 kPa (44 psi).

Maximum Velocity

The maximum velocity of water in mains during regular operation should not exceed approximately 3 m/s during regular operations in order to minimize energy losses.

Dead End Water Mains

When considering general operations and maintenance, it is good to minimize the number of dead end mains in a water distribution system. Dead end mains, specifically those that experience low water demands, can result in stagnant water, low chlorine residuals, and bacteriological/water quality issues. A looped water system can help to keep water circulating.

5.2 Existing Performance During Normal Operations

A hydraulic water model has been developed using Bentley WaterCAD V8i to help assess the existing performance of the Village's water system. The water model is a representation of actual infrastructure and attempts to simulate the actual operation and flow demands in the Village. The model has been used to identify concerns related to the operation of the system under normal operating conditions.

The following is a summary of the main concerns:

Maximum Pressure – Static Demand
As noted in Figure 5.1, static pressure exceeds 515 kPa at locations throughout the Village. Pressures between 515 kPa and 580 kPa typically do not have a negative impact on building fixtures. If the buildings in the areas experiencing pressure exceeding 580 kPa are not already individually protected by pressure reducing valves, the Village may wish to encourage their installation.

Minimum Pressure - Peak Hour Demand
Figure 5.1 identifies locations where pressures during peak hour demand do not meet the minimum requirement of 300 kPa. Specifically, the following locations in the Mesa Vista area that do not receive sufficient pressure during peak hour demand:

- Cemetery on Mesa Vista Drive – 145 kPa (21 psi)
- South End of Heustis Drive – 228 kPa (33 psi)
- Cul-de-sac of Vista Heights – 248 kPa (36 psi)

There are also significant concerns in these areas from a fire flow perspective, which is explored in detail in Section 5.4.

System Connectivity
There exist some dead end mains in the system. Some present looping opportunities that should be considered for construction when overlying road rehabilitation occurs.

There does exist two significant issues regarding system connectivity, as follows:

- North Ashcroft is serviced from South Ashcroft by a single main that is attached to the bridge. If this water main broke the Village would not be capable of supplying water to North Ashcroft. The only reservoir in North Ashcroft is in the upper zone (i.e. Zone 3) and there is currently no means of backfeeding water from this zone into the North Ashcroft part of Zone 1. The risk of the river crossing failing could be reduced if the reservoir in North Ashcroft could supply all of North Ashcroft.
- The dedicated water main from the Main Pump Station to the Zone 1 Reservoir is the only supply pipe. If this main were out of service no additional flow would be provided.

5.3 Performance Standards During Fire Flow

Fire flow requirements are to meet guidelines outlined in Water Supply for Public Fire Protection (1999), prepared by Fire Underwriters Survey (FUS). FUS fire flows are dictated by building size, construction material, contents, if automatic sprinklers are installed, distance to other buildings and other similar characteristics. FUS also outlines the duration that a given fire flow is required. Many communities specify minimum fire flows for planning and development purposes based on class of buildings. The MMCD Design Guideline (2005) is commonly used by municipalities in BC to help establish appropriate flows. The following table presents the minimum flow and duration requirements for buildings without sprinklers outlined in those Design Guidelines.

Table 5.1: MMCD / FUS Minimum Fire Flow Requirements

Development	Fire Flow (L/s) During MDD*	FUS Duration (hours)
Single Family	60	1.5
Multi-family, Townhouses	90	2.0
Commercial	150	2.0
Institutional	150	2.0
Industrial	225	3.0
Industrial Flow Adopted for Hollis Rd. Area	180	2.5

***Note**:

1. minimum residual pressure at street level of 150 kPa (22 psi) during a power outage

It is worth noting that the Village has deemed it reasonable to establish a fire flow requirement of 180 L/s for the existing industrial area on Hollis Rd. Current development in this area is of a nature and building size that likely requires less fire flow. The Village also does not envision a large industrial development in this area in the coming years.

The above flow rates have been adopted as reasonable estimates for planning purposes for this analysis. It is recommended that detailed fire flow requirements be conducted for larger buildings in the Village, such as schools, the hospital and buildings in the industrial zoned areas.

5.4 Existing Performance During Fire Flow

The existing system was analyzed under MDD plus fire scenario water model that was developed using Bentley WaterCAD V8i software. Calculations are based on nominal pipe diameters identified in the Village's water system composite drawing and through discussions with Village staff. The model has not been calibrated using field flow tests but is reasonable to help identify areas of concern for long term capital planning.

The results from the analysis, as summarized in Figure 5.2 indicate that a significant portion of the Village cannot be delivered fire flows in compliance with the FUS guidelines. The following table summarizes some key scenarios.

It is worth noting that the fire flow needs for some larger buildings could be less than the recommended flow rates noted in the table above. Detailed building inspections and flow estimates for each of these buildings should be confirmed as part of subsequent design phases.

Table 5.2: Highlighted Fire Flows

Fire Location	Calculated Fire Flow (L/s)	Recommended Fire Flow (L/s)
Government Street and Ash Street (FH 106)	**50**	**60**
Cariboo Road and Rail (FH-501 to FH-503)	**34 to 112**	**150**
Mesa Vista Residential Area (FH-202 to FH 218)	**20 to 52**	**60**
Cornwall Place Multi-Family (FH-201)	**13**	**90**
Upper Mesa Vista Residential (FH-507 to FH-510)	**7**	**60**
Sporting Facility and Public Works Yard (FH-514)	**9**	**150**

5.5 Reservoir Performance Standards

The MMCD Design Guidelines provides the following reservoir water storage recommendation:

Volume = A + B + C
A = Fire Storage (Fire Underwriters Survey Fire Flow Storage)
B = Equalization Storage (25% of Maximum Day Demand)
C = Emergency Storage (25% of A + B)

This standard has been adopted by many communities in BC, including the City of Kamloops. It is recommended that the Village also adopt this storage volume standard.

Emergency Storage may be reduced or eliminated based on consideration of key factors such as the presence of more than one supply source, dependability of water source(s), presence of other reservoir(s) in the system and availability of standby power.

5.6 Existing Performance of Reservoirs

Existing reservoir storage volumes are compared to the MMCD Design Guidelines in the following table. The recommended storage requirement for Zone 1 has been calculated under two fire scenarios because they represent the highest demand/duration in Zone 1 in different areas of that zone, and are described as follows:

- Scenario 1 - Zone 1 Reservoir: 150 L/s for 2.0 hours at arena
- Scenario 2 - Zone 1 Reservoir: 180 L/s for 2.5 hours at Hollis Road (current available fire flow)

Table 5.3: Recommended Storage Requirements

Reservoir	A = Fire Storage (m^3)	B = Equalization Storage (m^3)	C = Emergency Storage (m^3)	Total Required Storage (m^3)	Actual Volume (m^3)	Storage Deficiency (m^3)
Zone 1 Scenario 1	1080	502	395	1976	1620	356
Zone 1 Scenario 2	1620	502	530	2652	1620	1032
Zone 2	324	558	221	1103	1365	Sufficiently Sized *
Zone 3	1080	695	444	2219	1140	1079 **

***Notes:**

1. The Zone 2 Reservoir storage requirement calculation is based on residential land used (60 L/s for 1.5 hours).
2. The Zone 3 Reservoir storage requirement calculation is based on institutional land used (150 L/s for 2.0 hours).

The Zone 2 Reservoir is sufficiently sized.

The Zone 3 Reservoir does not meet the recommended storage capacity, however, provision was made for a second cell when the Zone 3 Reservoir was originally constructed. If an additional cell was constructed the Zone 3 Reservoir would be sufficiently sized to provide fire flow for Zone 3.

In addition to providing storage within a pressure zone, additional credits for fire protection storage can be acquired by cascading water down from storage in a higher pressure zone. For the purposes of evaluating available water storage it is assumed that the amount of storage that can be utilized from an upper zone is equal to the actual reservoir capacity minus the equalization storage (B).

Table 5.4: Available Fire Storage

Reservoir	Actual Capacity (m^3)	B = Equalization Storage (m^3)	Available Fire Storage to Help Fight Fire in Zone 1 (m^3)
Zone 2 Reservoir	1365	558	806
Zone 3 Reservoir	2470 *	695	1775

***Notes:**

1. Based on addition of second 1330m3 cell

The water model was utilized to determine the theoretical demand from each reservoir during the two existing Zone 1 fire scenarios. The following table summarizes the results.

Table 5.5: Storage Surplus/Deficiency Summary

Scenario	Total Available Storage (m^3)	Total Required Storage (m^3)	Storage Deficiency (m^3)
Scenario 1: 150 L/s for 2.0 hours at arena	1478	1976	498
Scenario 2: 180 L/s for 2.5 hours at Hollis Road	1434	2652	1218

Zone 1 storage capacity does not meet the recommended storage capacity under either scenario.

5.7 Proposed Distribution System and Storage Capital Upgrades

Figure 5.3 provides a graphical summary of the proposed capital upgrades.

It is important to note that either Capital Upgrade #2A or #2B would actually be completed. Both options address similar issues but just in slightly different manners.

5.7.1 Capital Upgrade #1 – Second Zone 3 Reservoir

Constructing this new reservoir cell is recommended to meet fire flow storage requirements. This additional storage, in conjunction with the below PRV recommendations, will also provide the benefit of increasing storage for North Ashcroft if the single water main crossing of the river is out of service.

5.7.2 Capital Upgrade #2A - Proposed North Ashcroft PRV

This improvement option includes the installation of a new PRV on the 300 mm diameter water main along Government Street in North Ashcroft, which will allow water to flow from Zone 3 to Zone 1 when the pressure on the downstream side of the valve drops below the set point during a fire in the lower zone. The proposed PRV setting is 415 kPa (65 psi).

This upgrade will improve fire flows in the Hollis Road Industrial area and allow the Zone 3 Reservoir capacity (including the proposed second Zone 3 Reservoir cell) to supplement the Zone 1 Reservoir capacity. This option has been modeled to determine the resulting impact on system pressure and available fire flow. A summary of these calculations are provided in the following table.

Table 5.6: Storage Surplus/Deficiency Summary

Scenario	Total Available Storage (m^3)	Total Required Storage (m^3)	Storage Result (m^3)
Scenario 1	2032	1976	56 surplus
Scenario 2	2468	2651	183 deficiency

5.7.3 Capital Upgrade #2B - Reconfigure North Ashcroft Pressure Zones

North Ashcroft is currently divided into a three pressure zones by a PRV and a number of normally closed valves (refer to Figure 1.1 for existing system configuration). The intermediate pressure zone (Zone 2 North) can be eliminated by changing the PRV set point to approximately 310 kPa (45 psi) and opening a number of the normally closed valves. Figure 5.3 illustrates which valves would be opened and the resulting pressure zone configuration.

There are a number of areas in North Ashcroft that experience pressures over the recommended maximum of 515 kPa (refer to Figure 5.1). Figure 5.4 illustrates the anticipated system pressures with this capital upgrade.

The system pressure near the top of Western Avenue during peak hour currently approaches the minimum requirement of 300 kPa. It is recommended that pressure tests be completed at this location to determine the impacts of reconfiguring the pressure zones. If some of these properties do not receive sufficient pressure during peak hour demand, there may be an opportunity to include them in Zone 3 by moving the normally closed valve further southeast on Western Avenue.

As with Capital Upgrade #2A, this upgrade results in an increase in the available fire flow at the Hollis Road Industrial Area and increases the reservoir storage used to fight a fire in Zone 1. An analysis has been completed, with storage results summarized in Table 5.7, to determine the total available Zone 1 storage capacity. This analysis assumes that an additional 1,330 m^3 cell is added to the Zone 3 Reservoir as per Capital Upgrade #1.

Table 5.7: Storage Surplus/Deficiency Summary

Scenario	Total Available Storage (m^3)	Total Required Storage (m^3)	Storage Result (m^3)
Scenario 1	2248	1976	271 surplus
Scenario 2	2657	2651	6 surplus

5.7.4 Capital Upgrade #3 – Improve Zone 2 and Zone 4 Fire Flow

Pressure Zone 4 is serviced by a booster station that pumps water from Zone 2 to a balancing tank. Fire flow is provided to Zone 4 by bypassing the booster station using two fire hydrants (on either side of the booster pumps) and pumping water from Zone 2 using the Village's fire truck.

The current configuration of pressure zones 2 and 4 does not provide acceptable fire flows to either zone. This proposed capital upgrade includes the following improvements:

- Install an additional reservoir supply main
- Move the Zone 4 booster station closer to the lower Mesa Vista residential subdivision

System pressures and fire flows have been analyzed to determine if this reconfiguration will have any negative impacts. The following is a brief summary of this analysis.

Install Additional Reservoir Supply Main

Based on the available record drawings and discussion with Village staff, it appears that the existing Mesa Vista reservoir supply/distribution line is 300 mm in diameter. Urban Systems recommends that the existing main diameter be confirmed to ensure the proposed main is properly sized.

This upgrade includes installing an additional reservoir supply main from the Mesa Vista Reservoir along the public access on the eastern end of the Mesa Vista subdivision. This upgrade results in significant fire flow improvements in the lower Mesa Vista subdivision and Cornwall Place multi-family development. It will provide minimal improved fire flow to the Upper Mesa Vista residential area. Table 5.8 provides a summary of the flow rates.

Table 5.8: Fire Flow Calculations

Fire Location	Recommended Fire Flow (L/s)	Existing Calculated Fire Flow (L/s)	Calculated Fire Flow with Proposed Upgrade (L/s)	
			With Additional Main	With Additional Main and Relocated Booster Station
Cornwall Place Multi-Family	90	13	53	59
Mesa Vista Subdivision (FH-202 to FH-218)	60	20 to 52	52 to 158	52 to 179
Upper Mesa Vista (FH-507 to FH-510)	60	7	13	170 60*

***Notes:**

170 L/s is the calculated fire flow available for Upper Mesa Vista with this proposed upgrade is based on the flow available at the proposed booster station location.

60 L/s is the calculated fire flow permissible to keep velocities in the 150mm water main below 3.5 m/s (MMCD maximum allowable velocity during fire flow).

These calculations do not take into account the pumping equipment that the Village uses to bypass the booster station.

Relocate Zone 4 Booster Station Closer to Lower Mesa Vista

This upgrade includes relocating the Zone 4 Booster Station closer to the lower Mesa Vista subdivision (and also involves installing the new reservoir supply line). Relocating the Zone 4 Booster Station will change the boundary between Pressure Zone 2 South and Pressure Zone 4, resulting in approximately 900 m of water main being moved from Pressure Zone 2 South to Pressure Zone 4. There would be positive and negative impacts on system pressures.

Positive:

- The highpoint near the cemetery will be moved to the higher pressure zone
- The static pressure will be increased from 25 psi to 90 psi.

Negative:

- The portion of main located at the low point (north of the cemetery) will be moved to the higher pressure zone
- The static pressure will be increased from 340 kPa (49 psi) to 815 kPa (118 psi)
- This still falls under the maximum recommended pressure of 850 kPa

5.7.5 Capital Upgrade #4 – Connect Trunk Mains

The dedicated water main from the Main Pump Station to the Zone 1 Reservoir is the only supply pipe. If this main were out of service no water could be provided by the treatment plant. The dedicated main is needed to provide chlorine contact time.

While having a dedicated main is recommended, there is a risk that, if this single main is out of service, no additional supply can be provided to the community. It is recommended that a connection between this pipe and the trunk main along 1st Ave. occur where the two mains run in parallel.

Because chlorine contact time in the supply main should remain, this distribution system connection should only be employed in emergency situations, when the supply main is out of service. To ensure water quality is sustained the system connection should be made using a double block and bleed arrangement

5.8 Cost Estimates

Table 5.9 outlines cost estimates related to the main distribution system and reservoir capital works identified in this investigation. The refined priority and timing for these projects are outlined in the 20 Year Capital Plan.

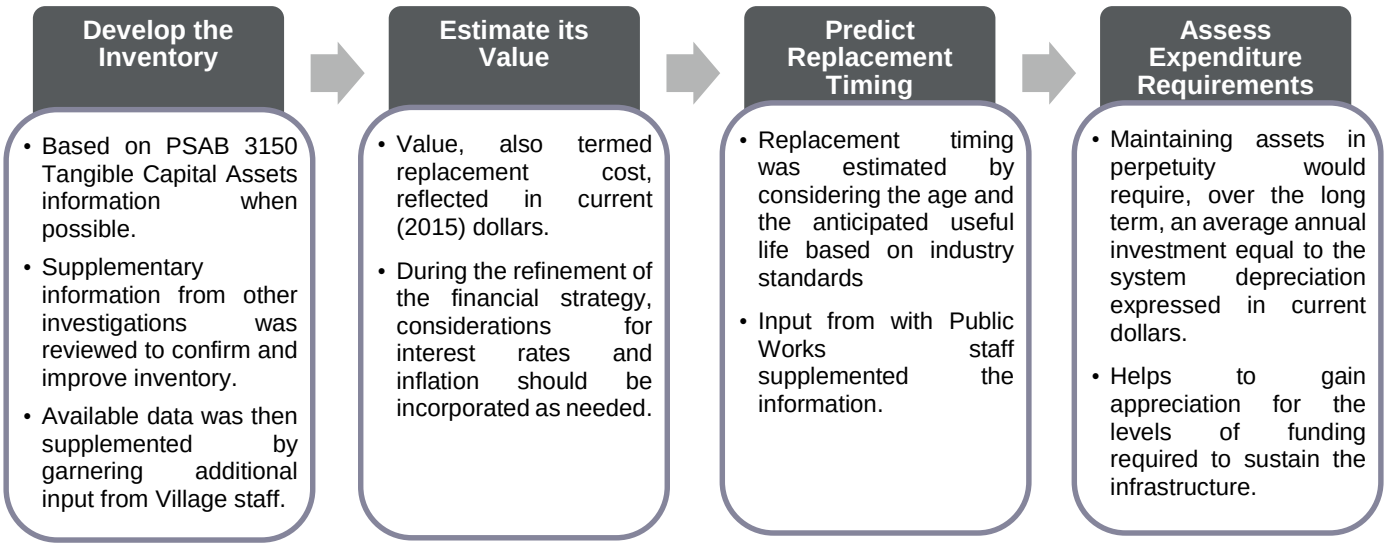
Table 5.9: Summary of Cost Estimates

Item	Cost Estimate
Capital Upgrade #1 – 2nd Cell of Zone 3 Reservoir (1,330 m3)	$1,200,000
Capital Upgrade #2A – Proposed North Ashcroft Supply and Install New Pressure Reducing Valve Station	$250,000
Capital Upgrade #2B – Reconfigure North Ashcroft Pressure Zones - Not capital cost – but field investigations would be required	
Capital Upgrade #3 – Improve Fire Flow in Zone 2 South and Zone 4 - Supply and Install 260m of 200 mm Reservoir Supply Main and Relocate Zone 4 Booster Station	$125,000
Capital Upgrade #4 – Connect Supply Main to Distribution Main Along 1st St. Using Double Block and Bleed Arrangement	$30,000

6.0 Replacing Ageing Infrastructure

Decisions regarding when and how to best maintain, repair and replace capital infrastructure has long term financial implications. Identifying necessary capital reinvestments and related expenditures must be completed in a clear and logical manner which prioritizes the need for capital works and balances this with the financial resources. With this approach and the resulting information the Village is well positioned in terms of advancing and improving their asset management practices. To gain an understanding of the capital investment needs the Village has taken stock of their existing water system infrastructure by following the four steps identified in Figure 6.1.

Figure 6.1: Steps in Taking Stock



6.1 Developing the Inventory

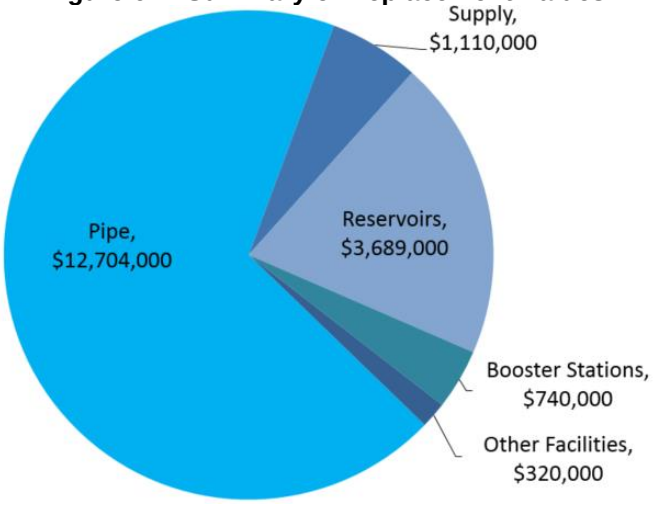
The focus of developing an inventory was on the Village's water infrastructure. It is appreciated that the Village is responsible for other classes of infrastructure. These other infrastructure classes should be merged into the overall analysis as the Capital Program evolves. The related spreadsheets that were created as part of this Water Master Plan are designed to allow for their subsequent use to summarize other classes of Village infrastructure (i.e. sewer, roads, buildings, etc.) should the Village wish to make that investment.

6.2 Estimating the Replacement Value

An overall replacement value in 2015 dollars was developed, as summarized in Figure 6.2, for the Village's Water Utility infrastructure. Additional detail is provided in Appendix C.

At a current replacement value of approximately $18.5 million, a substantial investment in infrastructure has been made. In order to ensure that this investment is maximized, it will be critical that proactive rehabilitation and replacement of assets be undertaken. This will require fiscal resources to be allocated towards maintaining existing levels of service.

Figure 6.2: Summary of Replacement Values



6.3 Predicting Replacement Timing

The inventory and valuation of water system assets provide a snapshot of what the Village owns and the replacement value. However the inventory and valuation alone do not provide information as to when and what financial resources will need to be allocated to maintain current levels of service. An approximation of the useful life of each asset is required to help determine when specific infrastructure components will need replacement. For this exercise replacement timings were developed based on industry averages.

With knowledge of anticipated replacement timing the Village can also consider the approximate remaining life of the inventory, which is considered a high level indicator of the condition of assets. As infrastructure nears the end of its useful life it is expected that the Village must also contend with:

- Increased unplanned maintenance (e.g. water main breaks); and
- Additional maintenance duties (e.g. more work on mechanical equipment to keep it operational).

<u>Reservoir Structural Assessment</u>

The following is a summary of when the Village's three existing concrete reservoirs came into operation:

1981 – Zone 1 Reservoir
1970 – Zone 2 Reservoir
1962 – Z3 Pump Chamber

Replacement of these structures can be expensive and would present a significant risk if a failure occurs. For these reasons the Village invested in a structural assessment of these reservoirs.

CWMM Consulting Engineers Ltd. (CWMM) therefore completed a condition assessment of the reservoirs as part of developing the Master Plan to determine the general condition of the structures and to provide recommendations if required for remedial works. The condition assessment was based on a field review which included primarily a visual examination of those components which could be observed directly, with some hammer soundings carried out on the components. CWMM also completed an analytical review of the concrete work in these reservoirs based on the limited number of structural drawings that were available.

The complete CWMM report is included in Appendix A. In summary, the structures are in very good condition with only minor improvements being recommended. This is viewed as positive news.

6.4 Theoretical Investment Needs

By starting with the theoretical investment needs the Village can define sustainable capital funding levels. Then, as the actual 20 Year Capital Plan is developed, these theoretical funding levels can be used as benchmarks for helping to assess sustainability.

It is important to note that these investment needs are theoretical as they represent a high level approximation of replacement timing and costs. The development of the actual Capital Plan includes a more refined review of replacement priorities.

Average Annual Investment Levels

In order to maintain levels of service of the existing water infrastructure in perpetuity the Village would theoretically need to annually invest approximately $280,000 in capital works. The average annual investment equals the annual depreciation, in current dollars, of that existing infrastructure.

It is worth noting that the annual investment amount developed from depreciation can be somewhat disjointed from what the Village could be expected to spend over the short or medium term (i.e. 5 to 10 years). For example, water mains installed in the last decade are not expected to require capital investment for several decades into the future.

It is also essential to note that the average annual investment reflects capital works to address reinvestment needs. It does not take into account capital expansion or other system improvements, such as construction of the new water treatment plant.

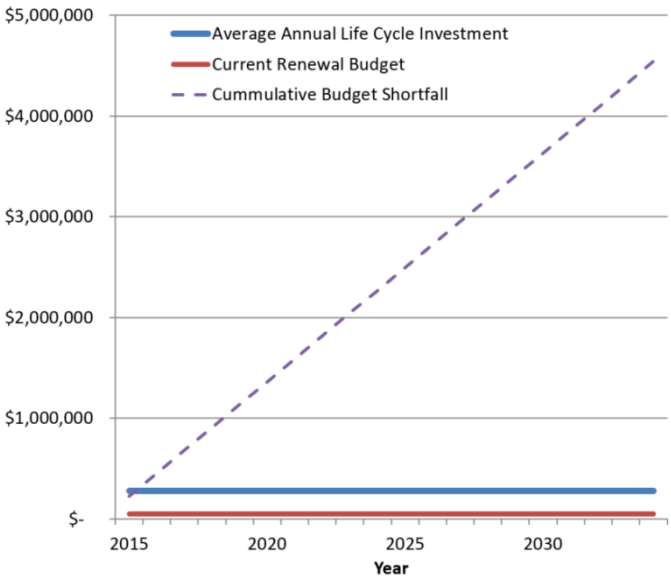
One objective of developing a sustainable capital program is to ensure that suitable funding is made available, in perpetuity, to replace and repair existing infrastructure. Therefore, it is recommended that the Village strive to, over the long term, adjust to a more consistent funding program as per what the annual depreciation indicates.

Deficit Growth

Figure 6.3, identifies the funding shortfall impact associated with a $280,000/year reinvestment in infrastructure. If it is assumed that the Village would otherwise invest $50,000 in capital replacement, over the next 20 years the budget shortfall will be in the order of $4,500,000.

The funding deficit is significant and will grow substantially if not addressed. The 20 Year Capital Plan and related cash flow model therefore includes investments in replacing infrastructure. A theoretical annual investment was not included in the Capital Plan but rather specific investments have been included.

Figure 6.3: Funding Summary



7.0 Financial Analysis and Capital Plan

A financial plan has been developed to support the implementation of this Water Master Plan. This financial plan considers capital upgrades presented in this report and infrastructure renewal requirements to address existing infrastructure condition.

The Village is moving towards sustainable financing of its water infrastructure, and has completed a financial analysis to guide investments (capital and operating) over the next 20 years and outlined an approach to achieving long term revenue stability. The timing of capital investments is based on balancing the risks associated with infrastructure failure over the next 20 years with the ability of the Village to raise rates to fund these investments.

7.1 20 Year Capital Plan

The development of the list of anticipated projects commenced as a data collection exercise. Previous investigations and Village input was sought out to help populate a list of works. Specific investigations were also undertaken, as outlined in this report. As the project list evolved, priorities were refined and reasons for the works were further outlined in a collaborative approach with the Village. The resulting projects were then presented to Village Council to help refine priorities.

Appendix C provides the Capital Plan table. This table identifies the following information:

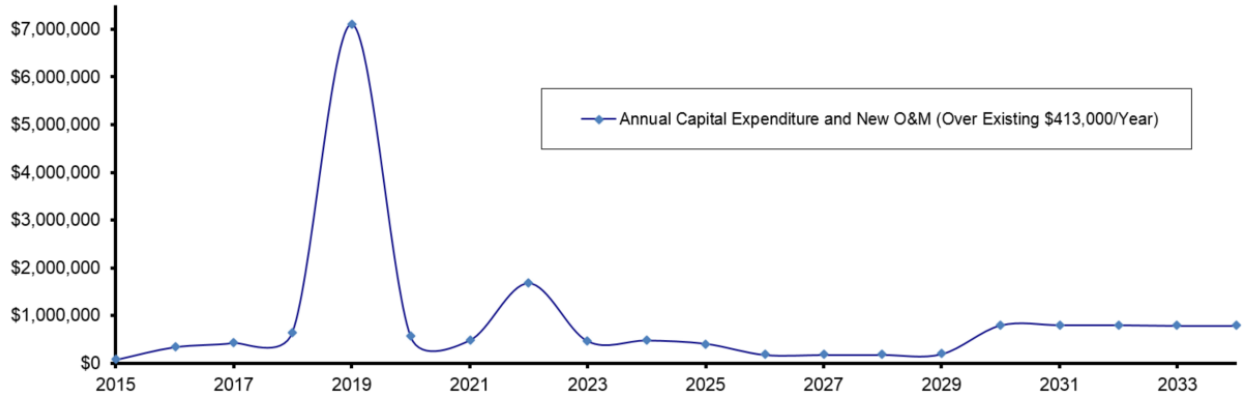
- Project names
- Recommended project timing and cost
- Division of each cost into the assumed amount of grant funding, amount that will be funded through proper developer finance methods and the remaining portion to be funded by Water Utility charges. As limited grow this expected the Plan does not include reliance on development to fund any of the investments.
- ~ $17 million in investment is expected during the next 20 years for:
    - Proposed water treatment plant and related upgrades to the pump station
    - New reservoir cell in North Ashcroft
    - Pressure zone changes in North Ashcroft to improve fire flows and system redundancy
    - Improve fire flows to Mesa Vista Heights
    - Replacement of aging water mains and major system components
    - Investment in water meters

The Capital Plan table is presented based on the assumption that the Village will be successful in securing 2/3 capital funding for the project through senior government funding. However, the cash flow analysis presented below identifies the financial impact based on a grant being secured and if no grant is secured. For the purposes of the analysis it is assumed that the treatment plant will be operational in 2019, however timing is dependent on the Building Canada Fund – Small Communities Component schedule.

Investments will not be limited to construction, repair or replacement of infrastructure. Additional operations and maintenance costs are also significant investments that the Village must consider when making plans. The new treatment plant will increase staffing, energy and chemical costs. There are also recommended actions that are currently not part of the Village's regular operations, such as completing cross connection control and investing in consistent water conservation efforts. The Capital Plan table includes these additional items to help outline a more complete investment plan.

Figure 7.1 presents a summary of the 20 year investments. The proposed water treatment plant represents a significant expenditure in the near term.

Figure 7.1: 20 Year Investments



7.2 Cash Flow Analysis

A detailed, interactive financial model was created to help understand the annual revenues and long term implications of the 20 Year Capital Plan on the long term financial sustainability of the Village's infrastructure systems. The financial model uses a constant dollar analysis (in 2015 dollars).

It is important to note that this model is intended to help staff, Council, and the public develop a better understanding of the financial implications associated with their infrastructure. It is not a plan intended for detailed budgeting purposes.

Financial model details are provided in Appendix C, and is built using the following baseline information:

- Current Annual Water Utility expenditures are $413,000
- Residents currently pay $274/yr per house
- No debt currently carried by the Utility
- Borrowing Term: 20 years
- Reserves: No transferring of reserves between utility or general revenue funds.
- Interest: On invested reserve funds = 3% On debt: 5%

- ~ $600,000 in Water Utility reserve at end of 2013 (Village Council has directed that this reserve not be drawn from for financing the Capital Plan. Instead, this reserve is for protecting against emergency repairs and operations.)
- $117,000/yr in non-conditional Gas Tax transfer (Community Works Fund)
    - $351,000 total potential funds in Village reserve by end of 2014
    - Assume $70,000 year for next few years can be directed towards water system investments

Two cash flow scenarios have been created for the treatment plant as it represents a significant capital investment. The first scenario is based on securing no grant funding. The second scenario includes an assumed 2/3 grant funding for the water treatment plant construction. The Village is preparing based on the Village's portion of the treatment plant capital costs being secured through a 20 year loan.

Another factor is the predicted replacement of ageing water mains. These replacements also represent significant investment but limited information is available to assess the actual remaining life of mains. In addition, some mains may be able to be replaced using trenchless relining. Potential cost savings using trenchless relining is dependent on which mains are replaced (number of service connections and continued reduction in relining cost due to advancement of that technique are significant cost factors). A 1/3 reduction in the base scenario water wain replacement costs was applied to help understand the impact of less capital investment in water mains due to pipes having longer actual remaining lives and potential cost savings of apply trenchless relining,

Figure 7.2 presents a matrix that outlines the impact of receiving grant funding and the sensitivity analysis of reducing the pipe replacement costs. The values relate to the increase in annual Water Utility Revenue compared to current rates.

Figure 7.2: Annual Increase in Revenues Associated with Cash Flow Scenarios
(In Addition to Existing $413,000 per Year)

Without Grant for Treatment Plant	$1,080,000	$1,200,000
With Grant for Treatment Plant	$640,000	$760,000
	Reduced Pipe Replacement by Cost by 1/3	**Pipe Replacement as per Original**

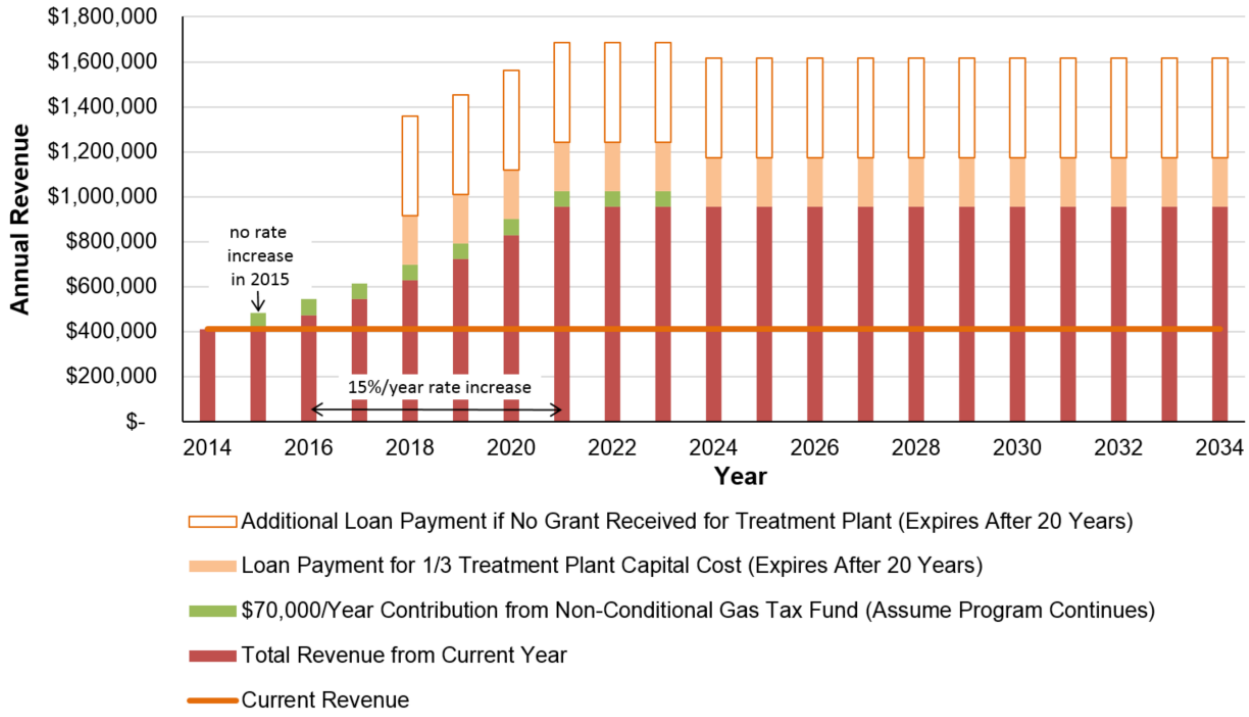
For all scenarios it is assumed that the utility rate increase for all costs except for the treatment plant loan would be phased in between 2015 and 2020. The treatment plant loan payments would commence based on the timing of the water treatment plant investments.

The above matrix helps identify that the impact of receiving senior government funding will have a significant impact on revenue requirements. In contrast, the impact of adjusting pipe replacement costs has less of an impact. The focus now should therefore be on securing senior government

funding. Investing in assessing pipe condition, while important, can be delayed until other higher priorities are addressed.

The following figure outlines the revenue needed to fund the proposed 20 Year Capital Plan, based on the assumption that pipe replacement will occur as per the base scenario. The Total Revenue from Current Year category in that graph includes capital replacement, distribution and storage upgrades as well as operations and maintenance costs for the treatment plant and other proposed activities.

Figure 7.3: Summary of Annual Revenue Needs to Fund Proposed Capital Plan and Additional Operations Costs



The resulting financial plan will represent a balanced approach, taking into consideration grants and affordable user rates. Without significant senior government grant funding, achieving sustainable financing of infrastructure renewal may not be affordable to Ashcroft residents or businesses.

8.0 Discussion and Next Steps

The Master Plan outlines an investment plan for the next 20 years that will allow the Village to provide water that is sustainable for the community. It will do so by achieving:

1. full compliance with existing Interior Health Authority policies;
2. adequate capacity to meet customer demands; and
3. a consistent level of service to all existing customers.

This plan takes a proactive risk management approach to address the major sources of risk exposure. The Village has assessed the following risks and will monitor them with the intention of adjusting the plan when or if it becomes necessary.

Table 8.1: Potential Risks to Providing Clean and Sustainable Water Service

Risk	Response
Climate Change	River channel assessed for stability across a number of years Water conservation measures to reduce water use requirements Identified that river intake could be lowered if installed downstream of existing intake – but confirmed that existing intake is below 200 year low river level
Infrastructure Failure	Identified primary, ageing transmission and pumping system investments Reservoir storage increase is recommended Distribution system redundancy increased with new PRVs and mains Universal metering will allow quicker response time to system leaks
Changes to Population and Development	Works have been scheduled to allow flexibility Revisit population decline, and related reduction in customer base, as part of the annual utility rate adjusting to ensure appropriates revenue is obtained Continue with investments in tourism and economic development to help sustain or grow the customer base
Resistance to increasing Water Utility rates	Public outreach program
Inadequate funds from government grants	Village now understands the impact of not receiving a grant for the Water Treatment Plant Engage with the Province and the Union of British Columbia Municipalities to help ensure they understand the priority of this project to the community Seek support from Interior Health Authority as part of making application
Major Changes with the Watershed	Collaboration with other jurisdictions and stakeholders should be undertaken Filtration plant will provide buffer to water quality changes

8.1 Subsequent Testing and Design

As stated in previous sections, additional investigation and design work is needed to advance the proposed water treatment improvements. This section provides a brief list of the anticipated items:

- Apply for funding and/or confirm that the Village would like to proceed with treatment plant improvements;
- Develop proposed implementation plan and schedule;
- Prepare conceptual design and identify information gaps;
- Continue water quality testing;
- Confirm whether to undertake pilot testing, confirm scope, timing and budget;
- Complete predesign, including field investigations and environmental review;
- Complete detailed design and tendering;
- Apply for required permits and approvals;
- Construct and commission system, including uni-directional flushing; and
- Operations and monitoring.

The Village should consider engaging various agencies such as Interior Health at each stage. The Village should also consider hiring a new water treatment operator at some point during system design so that they can provide input into the design, assist with project Administration on behalf of the Village, and be involved in system construction and commissioning.

8.2 Water Conservation and Metering

The Village acknowledges that reducing water use will provide the following benefits:

- Reduce energy & chemical costs;
- Reductions in treatment plant capital costs; and
- Promote stewardship of a valuable resource.

The Village has noted that the introduction of water meters will likely be necessary to reduce water consumption by the targeted 25%. Taking a phased approach will let the Village judge the impacts of actions before investing in subsequent actions. It will also allow the Village to spread the costs of water conservation initiatives over a longer time period to help make it more affordable.

The following figure presents the proposed staged approach. Investments related to water meters have also been included in the 20 Year Capital Plan.

Figure 8.1: Summary of Proposed Water Metering Activities

Phase 1 – Engagement and Watering Bylaw (2015)

- Public education – outline value of saving water and provide water saving tips
- Enact water restrictions bylaw

Phase 2 – Leak Detection (2015)

- Complete acoustic monitoring of leaks
- Repair leaks on Village system
- Inform residents of leaks on private property for them to repair

Phase 3 – Industrial, Commercial and Institutional Water Metering (2017)

- Study water use and impact of conservation efforts from first two phases
- Complete benefit/cost review of metering option
- Engage with stakeholders
- Install meters and adopt a "user pay" water rate

Phase 4 – Study Residential Water Metering (2018)

- Residential water meters can be expensive to install and manage but they are proven to help reduce water use
- Before "jumping in" the Village will study if there is a case for the meters (What are the impacts of actions thus far? Will benefits outweigh costs?)
- Engage with the public

Phase 5 – Residential Water Metering (2020)

- If residential meters are found to be a good step – identify best process
- Install meters and adopt a "user pay" water rate

8.3 Communications

Changes to utility rates should be founded upon practical accounting however these decisions require careful consideration to community/stakeholder preferences. In other words, this study is intended to provide options for rate increases (including one selected scenario) however rate changes should be implemented as part of a public consultation process.

It is essential that the Village clearly articulate the benefit to the community, and the need to adjust revenues reasonably yet responsibly in order to fund capital improvements and infrastructure renewal. By embarking on a proactive communications plan and clearly outlining that the funds will be directed to water quality and fire flow improvements the Village will be able to judge the degree to which rate increases are acceptable.

The Village has already taken steps to communicate with the public aimed at achieving community support on the plan as proposed. The following key activities are were undertaken:

- Media coverage (newspapers, website)
- 2 public open houses
- Engagement with community members by Council

To ensure the long-term sustainability of the program, the Village will need to continue to communicate the objectives of the program. A communications plan framework should be developed and followed to assist with this ongoing effort to ensure the appropriate messages are being delivered and received by both the public and senior government.

8.4 Consider Proposed Revenue Increases

This Master Plan has outlined capital and operations investments associated with providing customers safe, reliable drinking water. It also identifies that recommended investments cannot be afforded based on the current Water Utility revenue, even if a senior government grant is secured for the proposed water treatment plant.

The cash flow analysis indicates that revenue increases can be phased in to help minimize the impact on customers. However, manipulating the cash flow numbers indicates that delaying the revenue increases will result in delays in recommended capital investments and additional operations tasks.

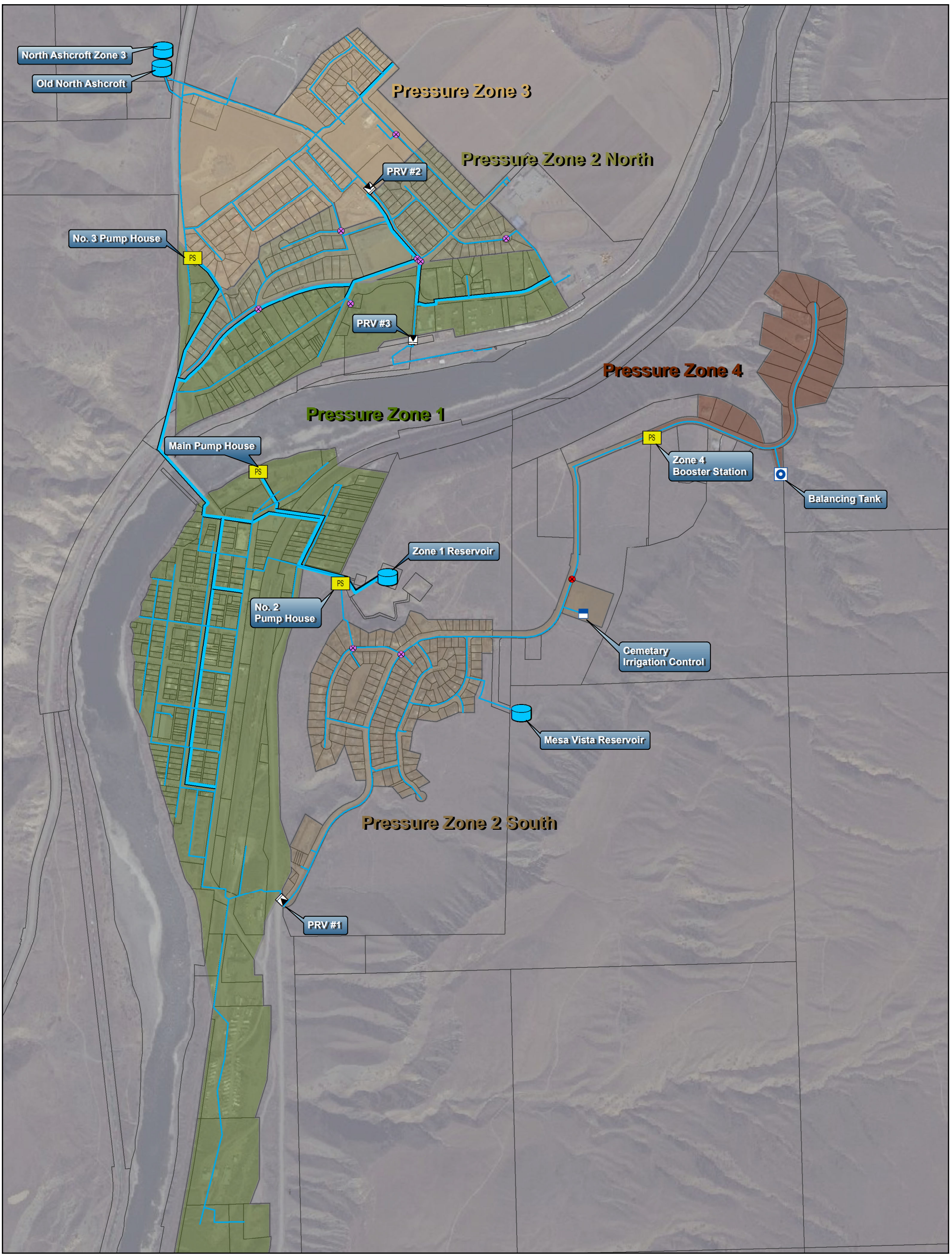
It is recommended that the Village adopt the proposed revenue increases presented in the Master Plan.

8.5 Engage with Senior Governments

Securing a senior government grant to help fund the water treatment plant would have a dramatic impact on the affordability of the overall Water Master Plan. It is recommended that the Village be proactive in making application for funding (such as the current Building Canada Fund – Small Communities Component that could potentially fund up to 1/3 of the treatment plant capital cost) and engage with senior government officials to help promote the Village's commitment to sustainably operating and funding the Water Utility and the impact and the risks if funding is not secured.

Figures

Distribution System Analysis and Recommended Upgrades



Ashcroft
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Water Master Plan

Village of Ashcroft Water System

Legend

- Reservoir
- Pump Station
- BalancingTank
- Irrigation Control
- PRV
- Zone Valve
- Closed Valve
- Trunk Main 250mm+
- Main <250mm

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

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Kilometres

Coordinate System: NAD 1983 UTM Zone 10N

Scale: 1:10,000

Data Sources:

- All infrastructure data composed from original Village of Ashcroft AutoCAD base
-Cadastral received from ICIS

Project #:	1093.0038.01
Author:	JC/CR
Checked:	RC/HT
Status:	FINAL
Revision:	A
Date:	2014 / 11 / 12

URBAN systems

FIGURE 1.1

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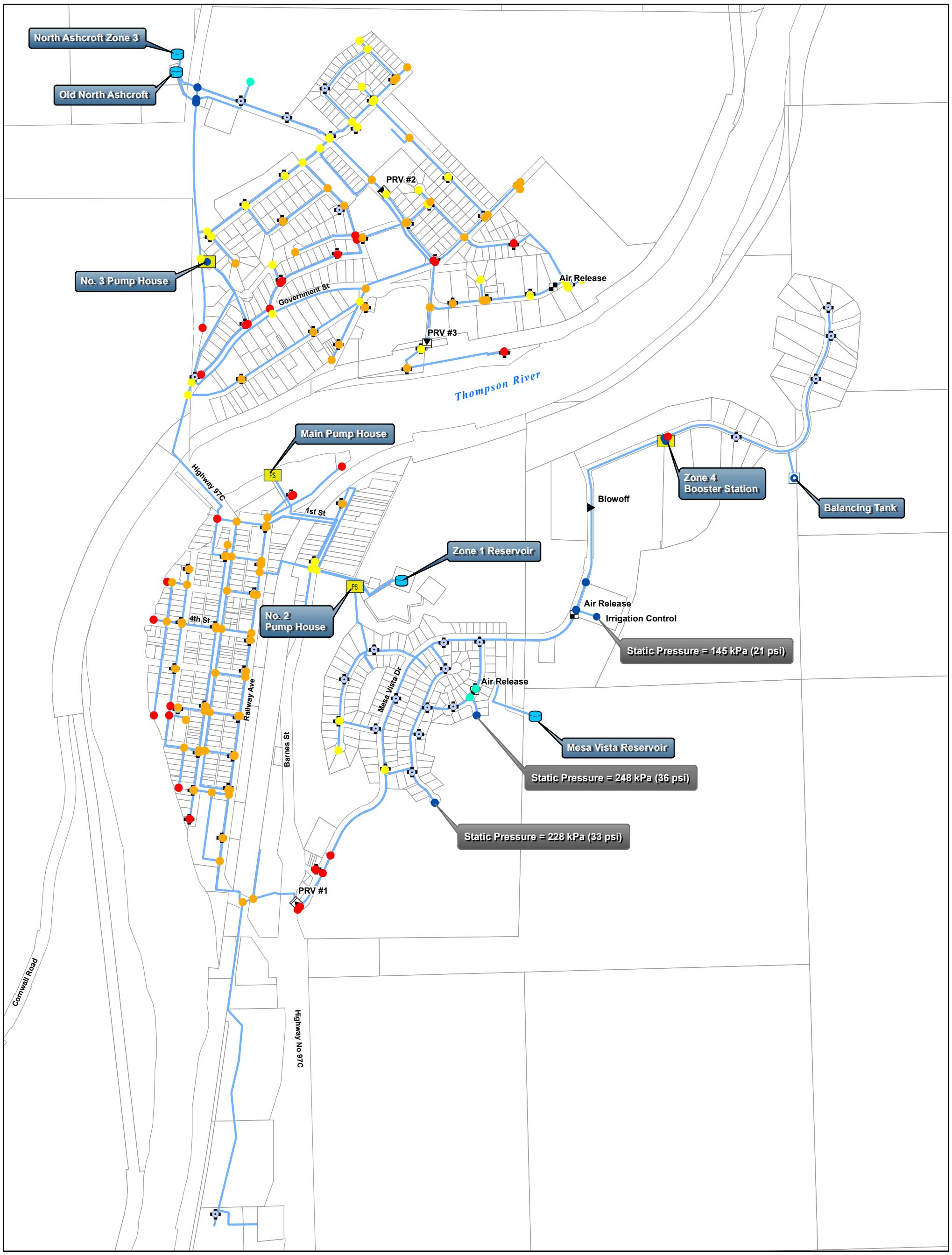

Ashcroft
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Water Master Plan

System Pressure Analysis - Existing Conditions

Legend

- Reservoir
- Pump Station
- BalancingTank
- Air Release
- Blowoff
- PRV
- Hydrant

Maximum Pressure at Static for Higher Pressure Areas

- 515 to 585 kPa (75 to 85 psi)
- 586 to 655 kPa (85 to 95 psi)
- 656 to 850 kPa (95 to 123 psi)

Minimum Pressure at Peak Hour for Lower Pressure Areas

- Less than 275 kPa (< 40 psi)
- 276 to 300 kPa (40 to 44 psi)

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Kilometres

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Date:	2014 / 11 / 12

URBAN systems

FIGURE 5.1

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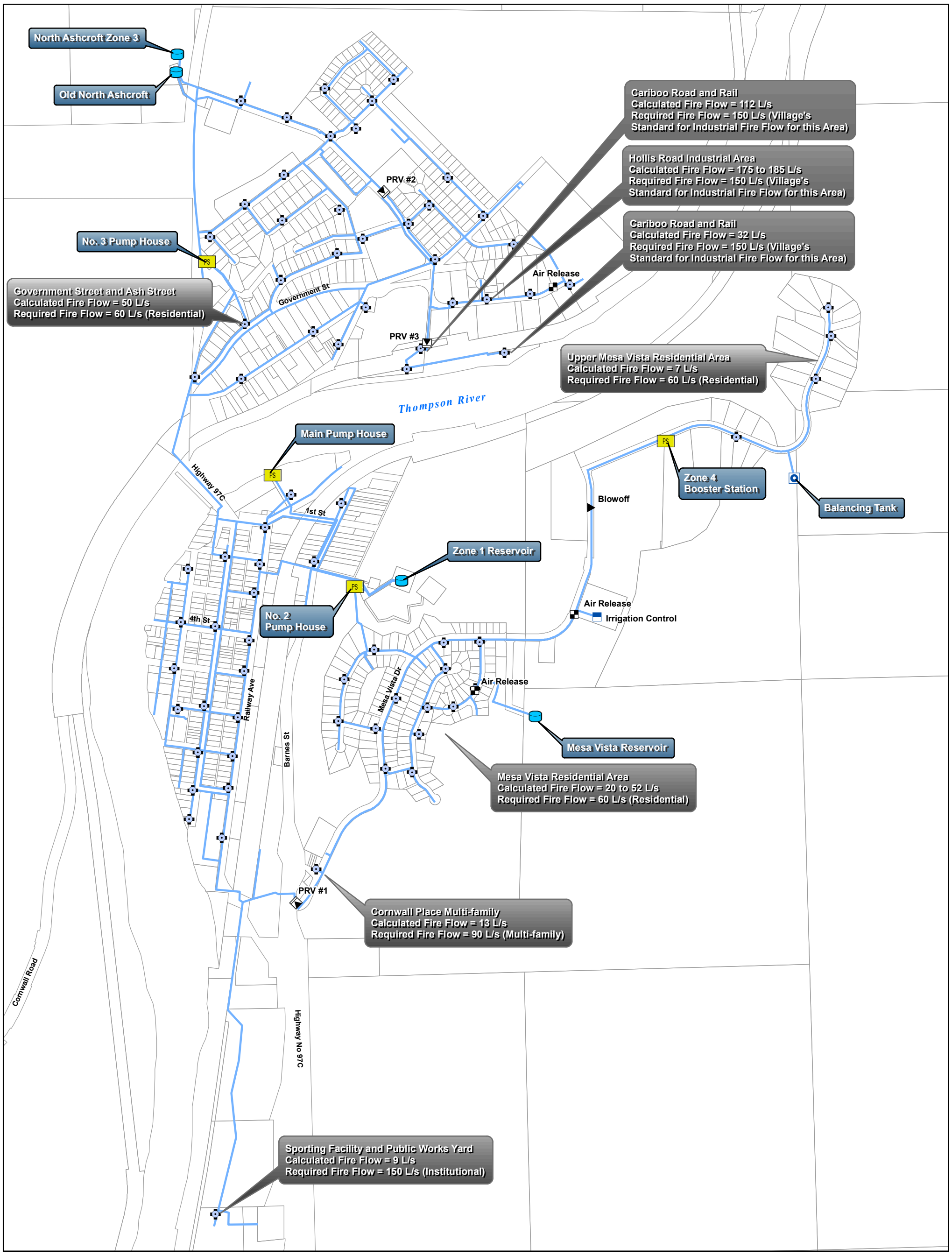

Ashcroft
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Water Master Plan

Fire Flow Analysis
Existing Conditions

Legend

- Reservoir
- Pump Station
- BalancingTank
- Water Control Facility
- Air Release
- Blowoff
- PRV
- Hydrant

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Kilometres

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Coordinate System:
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Data Sources:
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URBAN systems

FIGURE 5.2

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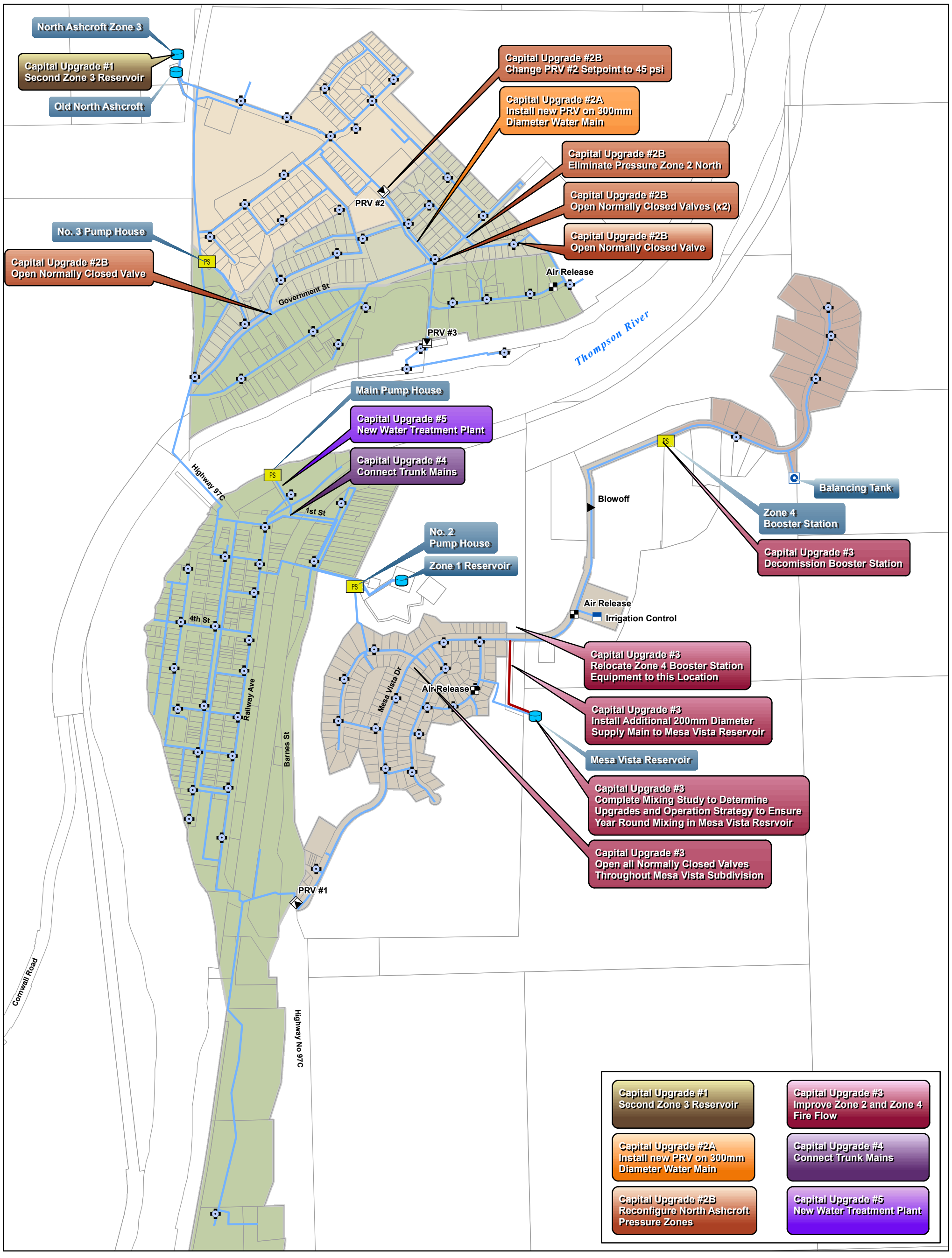

Ashcroft
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Water Master Plan

Proposed Major Capital Upgrades

Legend

- Reservoir
- Pump Station
- BalancingTank
- Water Control Facility
- Air Release
- Blowoff
- PRV
- Hydrant
- Pressure Zone 1
- Pressure Zone 2 North
- Pressure Zone 2 South
- Pressure Zone 3
- Pressure Zone 4

The accuracy & completeness of information shown on this drawing is not guaranteed. It will be the responsibility of the user of the information shown on this drawing to locate & establish the precise location of all existing information whether shown or not.

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URBAN systems

FIGURE 5.3

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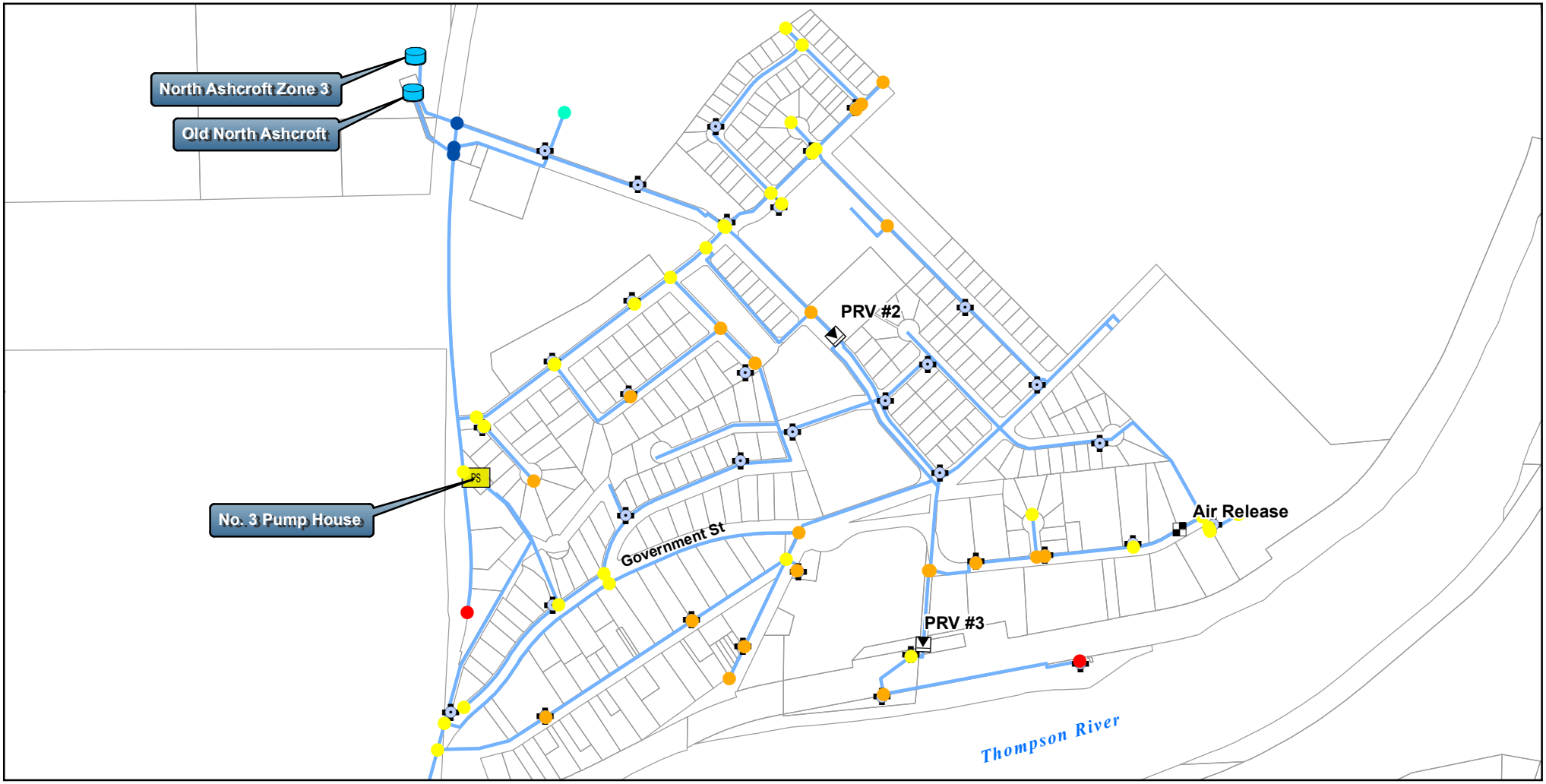

Ashcroft
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Water Master Plan

North Ashcroft System Pressures - Capital Upgrade #2

Legend

- Reservoir
- Pump Station
- BalancingTank
- Air Release
- Blowoff
- PRV
- Hydrant

Maximum Pressure at Static for Higher Pressure Areas

- 515 to 585 kPa (75 to 85 psi)
- 586 to 655 kPa (85 to 95 psi)
- 656 to 850 kPa (95 to 123 psi)

Minimum Pressure at Peak Hour for Lower Pressure Areas

- Less than 275 kPa (< 40 psi)
- 276 to 300 kPa (40 to 44 psi)

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Kilometres

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Coordinate System: NAD 1983 UTM Zone 10N

Scale: 1:8,000

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-Cadastral received from ICIS

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Date:	2014 / 11 / 12

URBAN systems

FIGURE 5.4

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Appendix A

Additional Investigations

THOMPSON RIVER AT ASHCROFT CHANNEL STABILITY ASSESSMENT WITH RESPECT TO UPGRADING THE WATER SUPPLY INTAKE

Prepared for:

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Chief Administrative Officer

VILLAGE OF ASHCROFT

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Draft: March 18, 2014
Final:

Frontispiece:

Looking upstream to the 'emergency' water supply intake on Thompson River at Ashcroft.

The buried, non-functional, infiltration gallery is located immediately upstream.

Photo by Brian Bennewith of the Village of Ashcroft Feb 25, 2014

Discharge at WSC Station "Thompson River near Spences Bridge" was 164.5 m^3/s (prelim.)

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Hazen Todd	Urban Systems Ltd., Kamloops
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Gabriele Matscho	BC Ministry of Environment, Kamloops
Jennifer Puhallo	BC Ministry of Environment, Kamloops
Bob Grace	BC Ministry of Environment, Kamloops
Lynne Campo	Water Survey of Canada, Vancouver, BC
Lauren Wicke	Water Survey of Canada, Vancouver, BC

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STATEMENT OF LIMITATIONS OF REPORT

This document has been prepared by M. Miles and Associates Ltd. [MMA] for the exclusive use and benefit of the Village of Ashcroft and Urban Systems Ltd., with respect to upgrading the water supply intake. No other party is entitled to rely on any of the conclusions, data, opinions, or any other information contained in this document.

This document represents MMA's best professional judgement based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by scientists and engineers currently practicing under similar conditions. No warranty, expressed or implied, is made.

THOMPSON RIVER AT ASHCROFT
CHANNEL STABILITY ASSESSMENT WITH RESPECT TO UPGRADING THE WATER SUPPLY INTAKE

1: INTRODUCTION AND OBJECTIVES

The Village of Ashcroft [VOA] has retained Urban Systems Limited [USL] to prepare a Water System Master Plan. M. Miles and Associates Ltd. [MMA], consulting fluvial geomorphologists, has been engaged to provide advice related to the long-term sustainability of the current water supply intake within Thompson River. More specifically, the existing infiltration gallery is presently operating at a decreased capacity. An additional inlet pipe has therefore been placed in the river on the left bank [*1] downstream of the infiltration gallery to provide a more reliable water source. USL wishes to determine if repair activities should include moving the infiltration gallery or the replacement intake to a more favourable location. The present report compiles relevant information on hydrological and channel conditions as a basis for answering the initial question on the intake location. Subsequent phases of this project could include assisting with the design of intake repairs or upgrading these structures.

2: STUDY PLAN

The initial steps in this project were to review relevant background material and to assemble a chronological series of air photos to document how channel conditions on Thompson River at Ashcroft have changed over time. A field inspection was undertaken on November 21, 2013 to discuss the intake operational history with the VOA staff, review channel conditions, inspect the existing intake structure and to evaluate upstream sediment sources. Turbidity data collected by the Provincial Government and the VOA were also obtained and reviewed. The present report is an augmented version of an initial draft which was prepared in March 2014.

3: PHYSICAL SETTING

3.1 PHYSIOGRAPHY AND GEOLOGY

Ashcroft is located on Thompson River 35 km downstream of Kamloops Lake. Sizeable tributaries to this section of channel include Deadman River, Battle Creek and Bonaparte River *(Figure 3.1.1)*. The VOA spans both sides of Thompson River in the vicinity of the Highway 97C bridge *(Figure 3.1.2)*. The water supply intake is located near the left bank approximately 170 m upstream of the Highway Bridge and 2.3 km downstream of the Bonaparte River confluence.

Kamloops Lake helps to regulate the flow in Thompson River at Ashcroft and will effectively trap the incoming sediment load from the upstream watershed. However, there are extensive deposits of erodible fine-textured sediments downstream of the lake outlet. The surficial geology and post-glacial geological history in the vicinity of Ashcroft has been extensively studied. Particularly useful references include:

Fulton, 1967, 1969 & 1975A & B;
Ryder, 1972, 1976;
Fulton and Smith, 1978;
Luttmerding and Sprout, 1978;

---

1 While looking downstream.

Ryder, Fulton and Clague, 1991;
Fyles and Clague, 1991; and
Young, Fenger and Luttmerding, 1992

As summarized in *Johnsen (2004)* and *Johnsen and Brennard (2004)*, a complex series of lakes formed during deglaciation in the Thompson River valley at Ashcroft *(Figure 3.1.3)*. Surficial geology mapping (*Figures 3.1.4 & 3.1.5)* illustrate the resulting distribution of deep unconsolidated and readily erodible sediment deposits which occur along the river valley downstream of Kamloops Lake. *Figure 3.1.6* illustrates a typical section across the Kamloops River Valley at Ashcroft. Upland surfaces are typically covered with either till or colluvium. Surficial materials along the lower valley walls are commonly 100 to 150 m thick and typically include comparatively recent alluvial fan material overlying glacio-fluvial outwash deposits, various aged tills and glacio-lacustrine silt and sand *(see Figure 3.1.6)*. Materials within present day Thompson River include a surface deposit of fluvial sediments overlying glacio-lacustrine silts and sands overlying till, which may overlie older glacio-lacustrine sediments.

The materials forming the terraces along Thompson River are susceptible to gullying and landslides *(see Hall, Porter, Quinn and Savigny, 2012)* which provide an ongoing source of sediment to Thompson River. The presence of fine textured sediments at shallow depth under the present channel bed can also result in unusually deep scour holes if the surface 'armour' is eroded *(see Neill and Morris, 1980)*. These fine textured underlying materials can also have comparatively poor permeability which could adversely affect the performance of a buried infiltration gallery.

3.2 HYDROLOGY

The nearest Water Survey of Canada [WSC] stream gauging station on Thompson River is located 34 km downstream of Ashcroft at Spences Bridge. The station has been in continuous operation since 1951. The upstream basin area is 54,900 km² which is 2% larger than that at Ashcroft (53,900 km² - *see northwest hydraulic consultants ltd. [nhc], 1992)*. Much of the difference in basin area is a result of the comparatively dry Nicola River watershed. For the present purposes, discharges measured at Spences Bridge are a suitable basis for describing the discharge regime at Ashcroft.

The seasonal variation in discharge on *"Thompson River near Spences Bridge"* is illustrated on *Figure 3.2.1*. This analysis indicates that streamflow starts to increase between late-March and late-April. The peak flow can occur between early-May and late-July and streamflows then gradually decrease to winter minimum flow values which typically occur between January and March. Fall rain or rain-on-snow events can result in elevated flows until November or December.

The historical variation in annual maximum daily and instantaneous discharges observed on *Thompson River near Spences Bridge* is illustrated on *Figure 3.2.2*. A flood frequency analysis has been undertaken using the BC Government computer program 'FREQAN' *[1] and the results are compiled on *Tables 3.2.1 & 3.2.2*. This analysis indicates that the largest recorded flood was in 1972 and has an average return period of approximately 75 years. More recent sizeable events occurred in 1997, 1999 and 2012 and the average return periods are ~15, 25 and 15 years, respectively. The historical variation in annual minimum flows is illustrated on *Figure 3.2.3* and the results of a low flow frequency analysis are compiled on *Table 3.2.3*. These analyses indicate the average annual minimum flow is ~180 m³/s with predicted extreme values at Spences Bridge being as low as ~120 m³/s. The associated flows at Ashcroft could be marginally lower.

1 The adopted flood values are based on the Log Pearson Type III distribution, fitted by the Method of Moments

Similar analyses have also been undertaken on streamflow data collected at the WSC station *"Bonaparte River below Cache Creek"*. The gauge has operated between 1912 and 1921 and post-1972. This site, which has a basin area of 5,020 km², is located 11 km upstream of the Thompson River confluence. The seasonal variation in flow is illustrated on *Figure 3.2.4*. Flows again begin to rise in late-March or April, the snowmelt freshet typically reaches a maximum around early-June, but intense rain-caused runoff can occur into late-July followed by declining discharges through the winter period. Winter ice effects can result in locally increased water levels.

The historical variation in annual maximum daily and instantaneous discharges on Bonaparte River are illustrated on *Figure 3.2.5*. The results of flood frequency analyses are presented on *Tables 3.2.4 & 3.2.5*. This analysis indicates the largest recorded flood occurred in 1980 and has an average return period of 75 to 100 years. Recent sizeable events also occurred in 1999, 2002 and 2011 with average return periods being ~20 years. The historical variation in annual minimum flows on Bonaparte River is illustrated on *Figure 3.2.6* and the results of a low flow frequency analysis are presented on *Table 3.2.6*. This analysis indicates that the average annual minimum flow is 1.2 m³/s and extreme values can be ~0.1 m³/s.

4: CHANNEL PROCESSES

4.1 CHANNEL DESCRIPTION

Thompson River in the vicinity of the existing water supply intake has an unvegetated width of approximately 150 m. A previous study by *nhc (1992)* includes river surveys and indicates that the channel slope is approximately 0.0017 m/m. The VOA is located on a low terrace which *nhc (2009)* indicates is above the predicted 200-year return period flood level. [*1] Their analyses indicate that the difference in elevation between the 200-year return period maximum and minimum water levels is approximately 6.7 m.

A poorly developed point bar which transitions into a diagonal bar extends from the left bank upstream of the water supply intake towards the downstream right bank. *Plates 4.1.1 to 4.1.4* illustrate this bar at streamflows of 283 and 164.5 m³/s (as measured at the WSC station *Thompson River near Spences Bridge)*. *Plate 4.1.5* is a 2003 photo of the same area illustrating conditions at a flow of ≥183 m³/s near Spences Bridge. This bar becomes better defined as the river discharge decreases. The location and orientation of the bar preferentially reduces the water level over the infiltration gallery. In contrast, the downstream end of the bar directs the low flow discharge towards the left bank and results in deeper water depths at the recently constructed water supply intake pipe and depths locally increase downstream of this point.

The bed material texture was measured along the edge of the left channel bank on November 21, 2013 using the 'tape-grid' procedure described in *Yuzyk (1986)*. The sampling location is illustrated on *Plates 4.1.6 & 4.1.7*, and the results are compiled on *Figure 4.1.1*. These data indicate that the B-axis D_{50} [*2] value is 250 mm. The D_{10} and D_{90} values are 120 and 460 mm, respectively. This material is very coarse in comparison to most rivers where the bed material is regularly mobilized. These surface bed materials therefore represent a lag deposit of residual rocks which likely eroded from formerly overlying

1 MMA has not verified this conclusion as part of the present study.

2 or grain size which is finer than 50% of the sampled rocks as measured on the intermediate axis of the rock.

sediments. The persistence of individual large boulders on the mid-channel bar surface over the 60-year period covered by the historical air photograph analysis attest to the stability of these materials.

4.2 HISTORICAL ANALYSIS

A series of historical air photographs has been compiled to illustrate changes in channel conditions over the period since 1948. Imagery covering an ~7 km long section of Thompson River is presented in Appendix 1. Changes in the extent of the active point bar located 1.6 km upstream of the Bonaparte River confluence indicate that the section of Thompson River below Kamloops Lake is periodically transporting sizeable quantities of gravel as bedload. Similarly, a periodically well-defined sediment plume from Bonaparte River and changes in the size of this fan indicate that this stream is contributing both fine and coarser textured material to Thompson River. Gully erosion or mass-wasting in both glacial till and glacio-fluvial or glacio-lacustrine materials adjacent to the river are also delivering sediment to the channel.

The Thompson River channel downstream of Bonaparte River shows little morphometric change since 1948 which indicates that the incoming sediment load is being transported through the reach. However, excavation holes are evident on the historical air photos analyses at the sizeable left bank point bar located 1.6 km upstream of the Bonaparte River confluence. This indicates that gravel mining has reduced the downstream coarse sediment supply after approximately 1959. The present status of this operation is unknown.

A series of larger scale historical air photos centred on the water supply intake is presented as *Figures 4.2.1A & B*. These photos further illustrate the stability of this section of channel. The persistence of individual large boulders on the channel bed and remnants of former upstream bridge piers suggest that the bed elevation has remained quite stable over the last sixty years. However, high water levels generally cover the mid-channel bar at the water supply intake site and prevent detailed comparisons of river bed characteristics. The cessation or reduction of gravel removal in the upstream channel has the potential to increase the bed elevation on the mid-channel bar located at the existing infiltration gallery. The stability of this site over the last 60 years suggests that this effect will likely be small as the incoming sediment load appears to be carried through this reach. Improved riparian and upslope land use practices on Deadman River or Battle Creek could also be reducing rates of coarse textured sediment production and might mitigate the effects of any reduction in future volumes of upstream gravel extraction.

4.3 SEDIMENT TRANSPORT

The field inspection undertaken on November 21, 2013, indicates that sediment loadings to Thompson River upstream of Ashcroft include:

i) Erosion or mass wasting of unconsolidated materials from the glacio-fluvial or glacio-lacustrine terraces bordering the upstream channel *(e.g. Plate 4.3.1);*

ii) Erosion from gullies or small streams *(e.g. Plates 4.3.2 & 4.3.3);*

iii) Material delivered to the channel by larger streams such as Bonaparte River *(Plate 4.3.4);* and

iv) Limited erosion of material from the channel bed and banks *(Plate 4.3.5).*

As discussed in Section 4.1, the channel bed and banks are frequently armoured with cobbles or boulders which are unlikely to be mobilized by normal flows. The diameter of rock which can be entrained by varying flow conditions can be estimated on the basis of the following equation:

$$dc = 13.7 R\, So$$

where: dc is the critical sediment diameter which can just be eroded by the flow (m)
R is the hydraulic radius (wetted area ÷ wetted perimeter which is approximately equal to the mean depth) (m); and
So is the water surface slope (m/m).

On the basis of the previous work by *nhc (1992)*, [summarized in Section 3.1], a maximum water depth of ~7 m and a river slope of 0.0017 m/m results in a mobile rock diameter of ~0.16 m. As indicated on *Figure 4.1.1*, approximately 75% of the material near the channel bank is coarser than this value. This implies that most of the material on the bed surface will be immobile under normal flows, although smaller sediments could be transported through this reach as either bed load or suspended load.

No measurements are available to document rates of bedload transport or to quantify suspended sediment concentrations or loadings. However, the BC Ministry of Environment is conducting a water quality sampling program in cooperation of the Village of Ashcroft [Jennifer Puhallo, R.P.Bio, pers. comm.] Seasonal data has been collected on Thompson River at Ashcroft and on lower Bonaparte River. All data available in March 2014 have been compiled on *Tables 4.3.1 & 4.3.2*, and this information includes 'turbidity' measurements. Information is only available for the winter low flow months when turbidity values will be at their seasonal minimum values. The collected data, which are plotted on *Figure 4.3.1*, suggest that fall and winter turbidity values on Bonaparte River are roughly twice as high as those observed on Thompson River. Measured values on Thompson River are typically less than 2 NTU and reached a maximum of 13 NTU. In contrast, turbidity observations on Bonaparte River are generally less than 4 NTU, occasionally reach values of 16 NTU and the maximum observed winter value is 42 NTU. Previous experience in the Bonaparte watershed suggest that these comparatively elevated values reflect the lack of upstream lake regulation, the occurrence of fine-textured surficial materials and land use related impacts. Turbidity values in the spring and summer *(see Figures 3.2.1 & 3.2.4)* are expected to be much higher than those observed in the winter low flow period.

The VOA routinely measures turbidity values along with other parameters in the raw water supplied to the #1 Pumphouse. These data have been extracted for the period between January, 2011 and October 2013 and the results are summarized on *Figure 4.3.2*. The seasonal variation in turbidity values generally follows the previously discussed seasonal variations in flow. Maximum observed values of ~5 NTU have occurred in May, June and July. To put these values in perspective, the maximum recommended drinking water turbidity values for treated water are summarized below, based on guidelines prepared by *Health Canada (2012)*.

TURBIDITY GUIDELINE		NOTES
Treated water <0.1 NTU at all times		Where possible filtration systems should be designed and operated to reduce turbidity levels as low as possible, with a treated water turbidity target of less than 0.1 NTU at all times.
Where not achievable:		
	≤ 0.3 NTU	Chemically assisted filtration: ≤0.3NTU in at least 95% of a) measurements made or b) the time each calendar month; never to exceed 1.0 NTU.
	≤ 1.0 NTU	Slow sand or diatomaceous earth filtration: ≤1.0 NTU in at least 95% of a) measurements made or b) the time each calendar month; never to exceed 3.0 NTU.
	≤ 0.1 NTU	Membrane filtration: ≤1.0 NTU in at least 99% of a) measurements made or b) the time each calendar month; never to exceed 0.3 NTU.

The GCDWQ Guideline Technical Document for Turbidity recommends turbidity levels for systems that use conventional, direct, slow sand, diatomaceous earth or membrane technologies. It states that:

"To ensure effectiveness of disinfection and for good operation of the distribution system, it is recommended that water entering the distribution system have turbidity levels of 1.0 NTU or less. For systems that are not required to filter by the appropriate authority, a higher turbidity level may be considered acceptable, provided that it does not hinder disinfection."

The 'raw water' data shown on *Figure 4.3.2* periodically exceed the recommended 1.0 NTU treated water value.

5: PROJECT IMPLICATIONS

5.1 INTAKE LOCATION WITH RESPECT TO SEDIMENT LOADINGS

There are no field data available to document spatial variations in suspended sediment concentrations or rates of bedload transport in the vicinity of the existing water supply intake. It is possible that elevated sediment loadings from Bonaparte River (or possibly other sizeable right bank tributaries) could not be fully mixed across the river channel. If this were the case, then higher values might occur on the right bank side of the channel. However, two smaller tributaries, which are active sediment sources, occur on the left bank downstream of the Bonaparte River confluence and these could possibly result in elevated loadings on the left bank. Field data would be required under a range of flow conditions to determine if this was an issue of concern. Given the fine texture of the material which is back-flushed from the infiltration gallery plumbing *(Plate 5.1.1)*, suspended sediment concentrations are likely to be well mixed and my impression is that spatial variation in sediment loadings are not sufficient to justify moving the water supply intake or pumphouse location (e.g. to the other side of the river).

5.2 INFILTRATION GALLERY OR INTAKE PIPE

The extensive fine textured sediment sources located upstream of Ashcroft will occasionally result in periods of high suspended sediment concentrations in Thompson River. These fine materials are likely to adversely affect the performance of a buried infiltration gallery. Any such structure will therefore require an effective and reliable long term method of back-flushing. Previous experience suggests that this may

be difficult to achieve. There may therefore be benefit in maintaining something similar to the present screened pipe intake as long as other criteria related to water quality or treatment can be met successfully.

5.3 INTAKE LOCATION WITH RESPECT TO CHANNEL MORPHOMETRY

As discussed in Section 4.2, the size and orientation of the Thompson River channel in the vicinity of the water supply intake has been very stable over the 60-year period covered by the historical air photograph analysis. However, high water levels do not allow small changes in the size and location of the mid-channel bar located adjacent to the existing infiltration gallery to be reliably evaluated. The bar surface appears to be quite stable, but this area could preferentially be subject to future sedimentation if sizeable quantities of coarse material were introduced to the upstream channel, or if reduced upstream gravel extraction resulted in elevated coarse sediment loading. At present, water depths over the existing infiltration gallery are very shallow during periods of minimum flow in mid-winter *(see Plates 4.1.1. to 4.1.5)*. This limited depth could restrict inflows, particularly if a heavy ice cover occurred at the same time, or if additional sediment accumulation were to occur. This potential issue was recognized by *nhc (1992)* and they suggest that, if required, a small channel could be excavated through the bar to the upstream section of the mainstem of the river to increase the winter water supply to the infiltration gallery.

Any proposed instream channel excavation would require provincial approval for 'construction in and about a stream', fisheries approval and appropriate sediment control measures. This work could be comparatively easily undertaken during the late-winter low flow period when much of the bar would be exposed (but this would likely conflict with the preferred fisheries 'instream construction window'). The longevity of this excavation would depend on the magnitude of future flood flows and the quantity of incoming coarse textured sediment loadings. Additional analyses would therefore be needed to quantify these risks and develop an optimum configuration.

As discussed in *Section 3.2*, extreme minimum discharges could be approximately 25% smaller than those illustrated on *Plates 4.1.2 & 4.1.4* (i.e. 120 m^3/s vs. 165 m^3/s as measured at Spences Bridge). There is therefore a possibility that extreme low water levels or future sediment deposition could adversely affect the performance of the present pipe intake. The design drawings for this structure prepared by USL are reproduced as *Figures 5.3.1 & 5.3.2*. This information suggests that the 200-year return period low flow water elevation is 285.4 m and that the bottom of the 250 mm HDPE pipe is to be installed at an elevation of 284.3 m. This implies that there would be an ~1 m water depth during the 200-year average return period low flow. However, there is some unavoidable uncertainty in the predicted water level elevations and winter ice accumulation or sediment deposition could adversely affect the flow of water to the intake. There is also a risk that water levels lower than the one in 200-year event will occur over the life of the project *(see Table 5.3.1)*. It would therefore be prudent to monitor the performance of the intake during low water conditions. If warranted, the intake could be shifted downstream to the area where deeper water depths are located adjacent to the left bank.

5.4 REHABILITATION OF THE INFILTRATION GALLERY

Pumping out and back-flushing experiments (illustrated on *Plate 5.1.1)* indicate that the infiltration gallery is plugged with fine sediments. It is likely that these sediments are preferentially lodged in the geotextile filter cloth that was placed within the backfill *(see nhc, 1992)*. There may also be integrity issues with the piping, given the limited number of locations where 'back flow' was visible.

The sediment accumulations reflect the fine-textured suspended material which is regularly transported by Thompson River. Any future repairs or gallery re-design must be able to either successfully 'treat' this

incoming load in the raw water supply or have the capability to periodically and reliably flush it out of the system. It could be desirable to further explore both alternatives if this has not already been undertaken.

5.5 SUITABILITY OF SUB-SURFACE MATERIAL WITH RESPECT TO ADDITIONAL INSTREAM EXCAVATION

As indicated on *Figure 3.1.6*, there is the potential for fine-textured sediment to occur at shallow depth under the alluvial materials forming the bed of Thompson River. Records and photos from previous construction work should be consulted and, if necessary, additional test pitting conducted to ensure that the performance of any re-configured infiltration gallery will not be adversely affected by these materials.

6: FUTURE WORK

Additional studies which would improve the present analysis or provide a basis for identifying future changes in channel conditions which could adversely affect the VOA's water supply intake include:

i) Conducting a topographic survey of the river bed in the vicinity of the infiltration gallery and emergency intake, such that future changes in elevation can be detected. This would assist in identifying any long term trend in the size and location of the mid-channel bar. Regularly obtaining low water photographs of this area from the identical locations would also assist in this analysis, as would locating any older photographs illustrating former low water channel conditions;

ii) Installing a water level recorder on the present intake structure to document seasonal and annual water depths. This record should be compared to discharge values observed by the WSC at Spences Bridge, as it may be possible to detect changes in the relationship between water depth or elevation and discharge over time;

iii) Requesting the VOA (and in particular Brian Bennewith) to record the dates and take photographs to document the conditions associated with any issues related to the water supply intake;

iv) Obtaining additional data to document the seasonal or annual variability in suspended sediment concentrations on Thompson River. This could include obtaining updated results from the on-going Ministry of Environment study and collecting samples which would allow the determination of seasonal relationships between suspended sediment concentration and turbidity in both river water and within the water treatment facility;

v) Better defining the volumes of gravel which have been mined from the point bar located 1.6 km upstream of the Bonaparte confluence and determining the planned future operations at this site; and

vi) obtaining historic information and/or conducting a test pit to better determine near surface stratigraphy, grain size and permeability if the infiltration gallery is to be repaired or moved.

7: CERTIFICATION

This report was prepared by:

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8: SOURCES OF INFORMATION

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8.2 PERSONAL COMMUNICATIONS

Jennifer Puhallo, R.P.Bio BC Ministry of Environment, Kamloops

FIGURES

Figure 3.1.1: Google Earth (2004 & 2005) imagery of the regional study area.

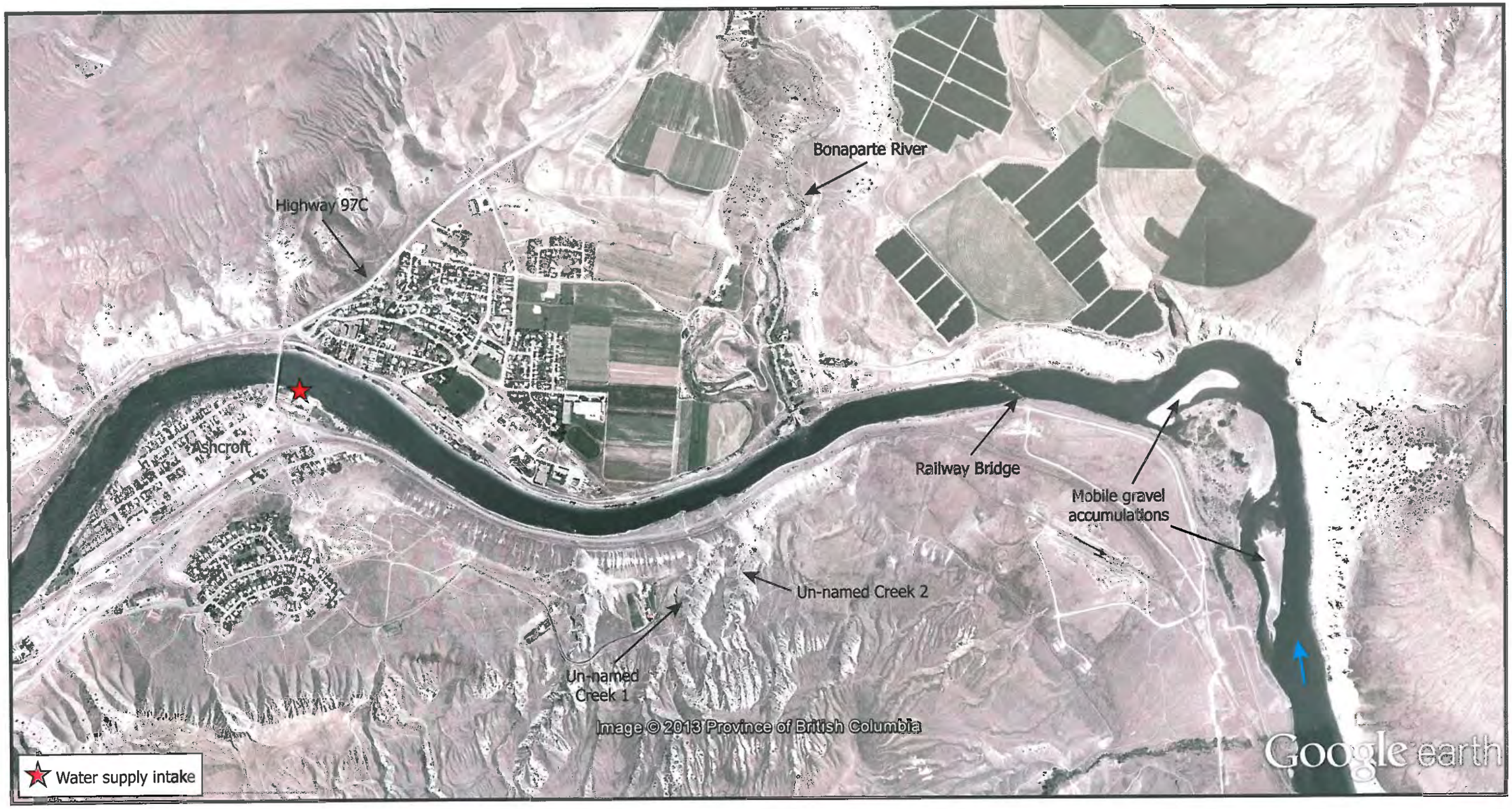


Figure 3.1.2: Google Earth image of Thompson River in the vicinity of Ashcroft.

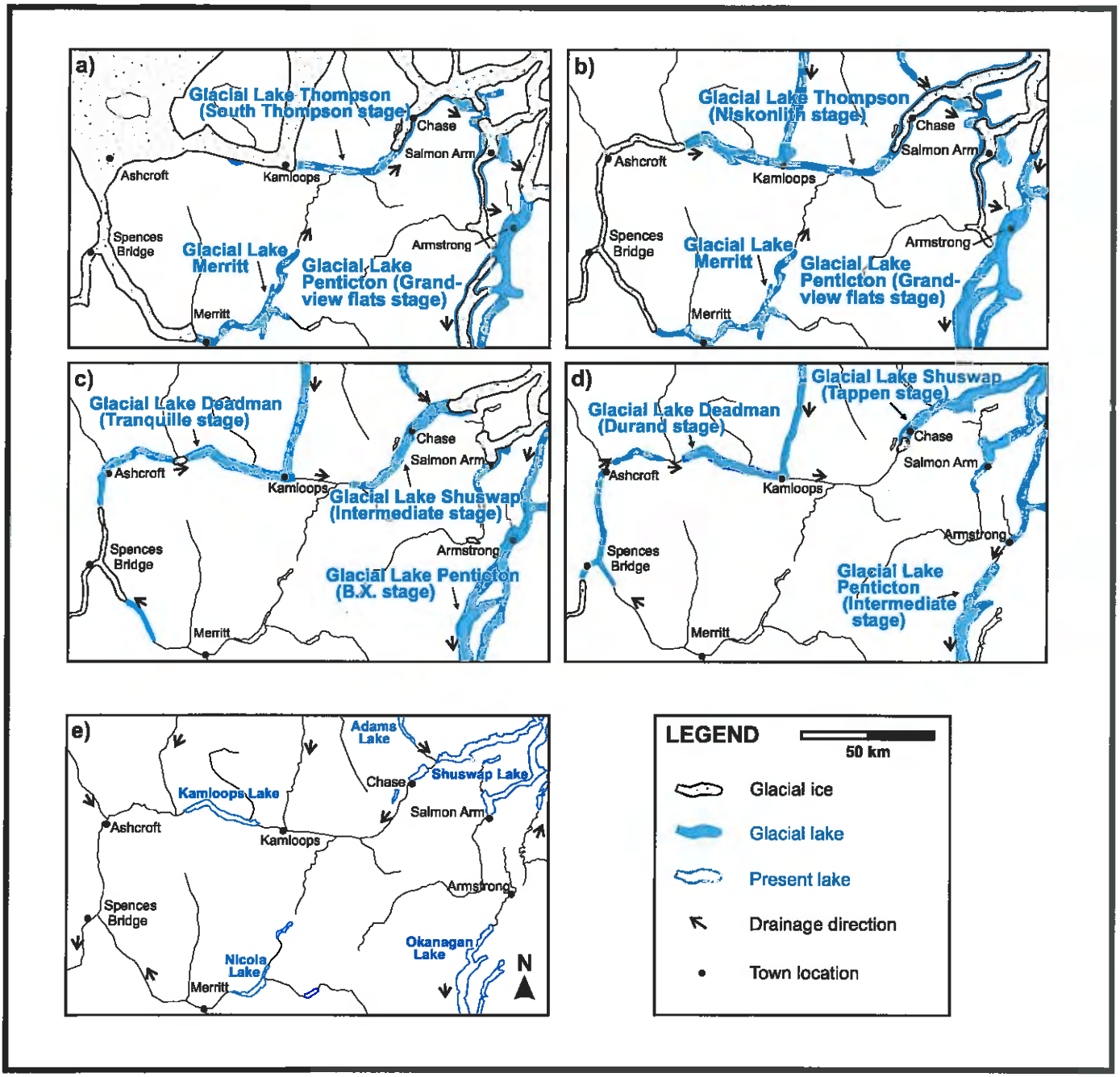


Figure 3.1.3: Evolution of late glacial lakes in the southern interior of British Columbia as proposed by Fulton (1969) and Ryder (1981). (From Johnsen, 2004).

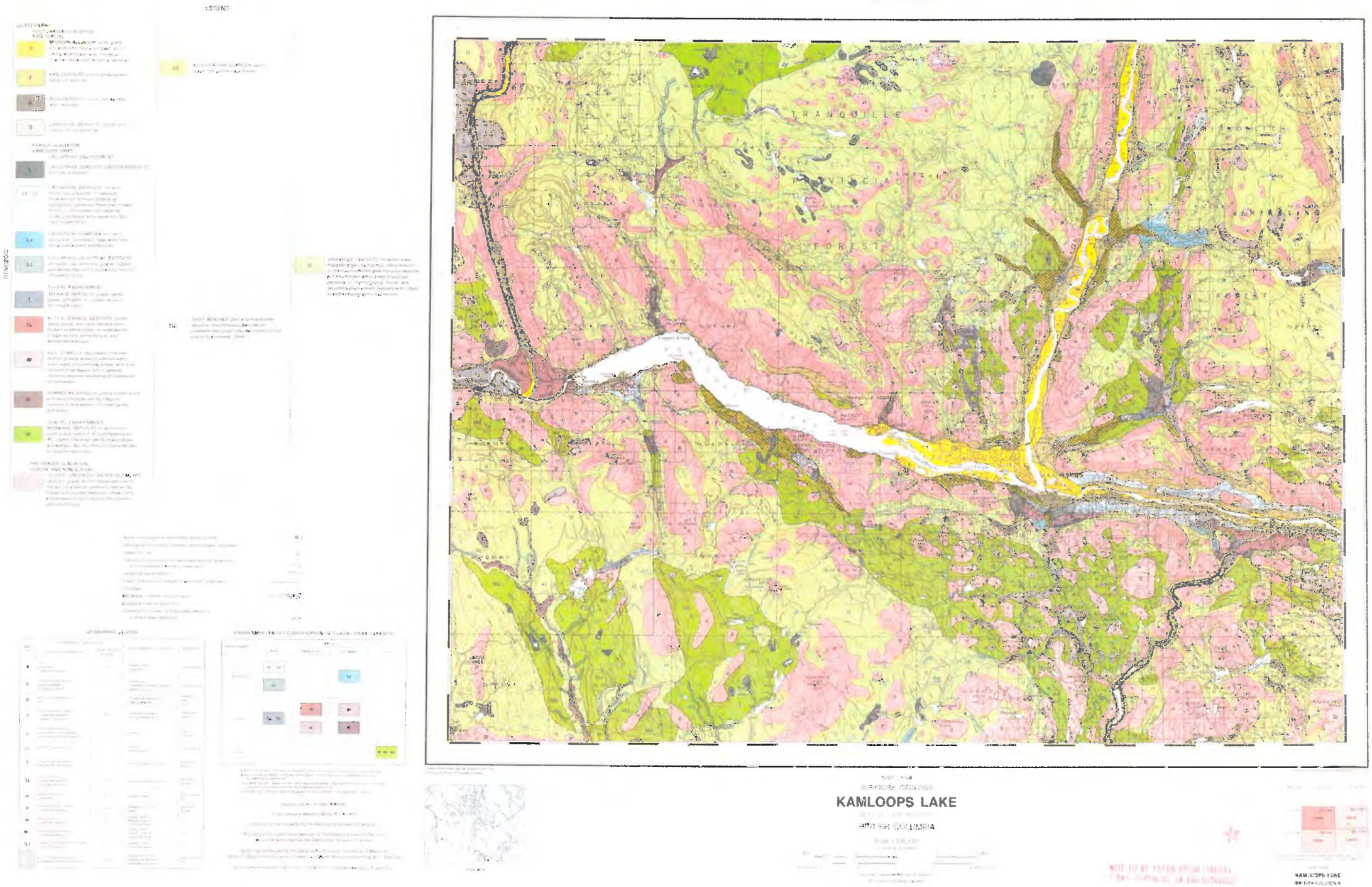


Figure 3.1.4: Surficial geology of Kamloops.

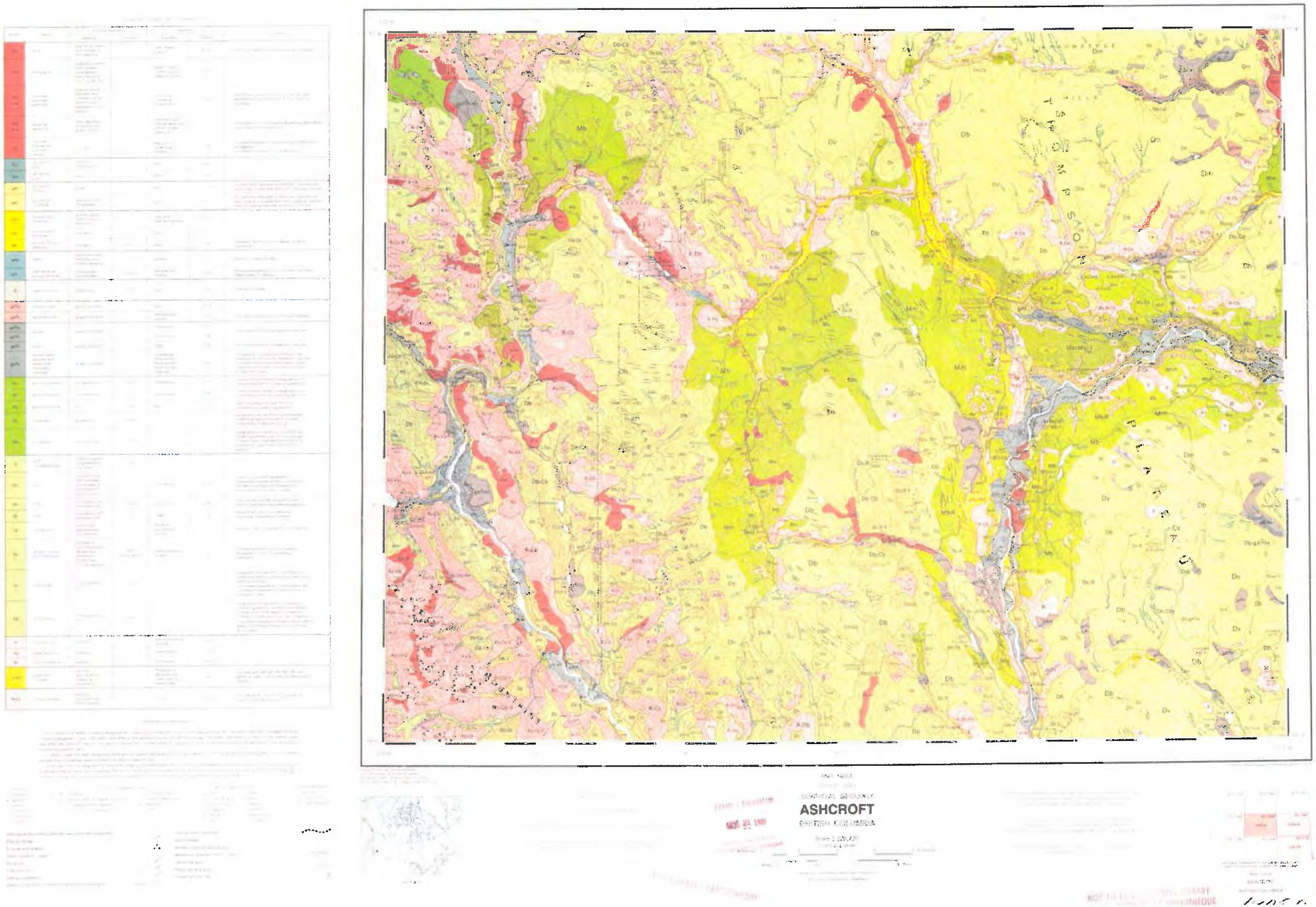


Figure 3.1.5: Surficial geology of Ashcroft.

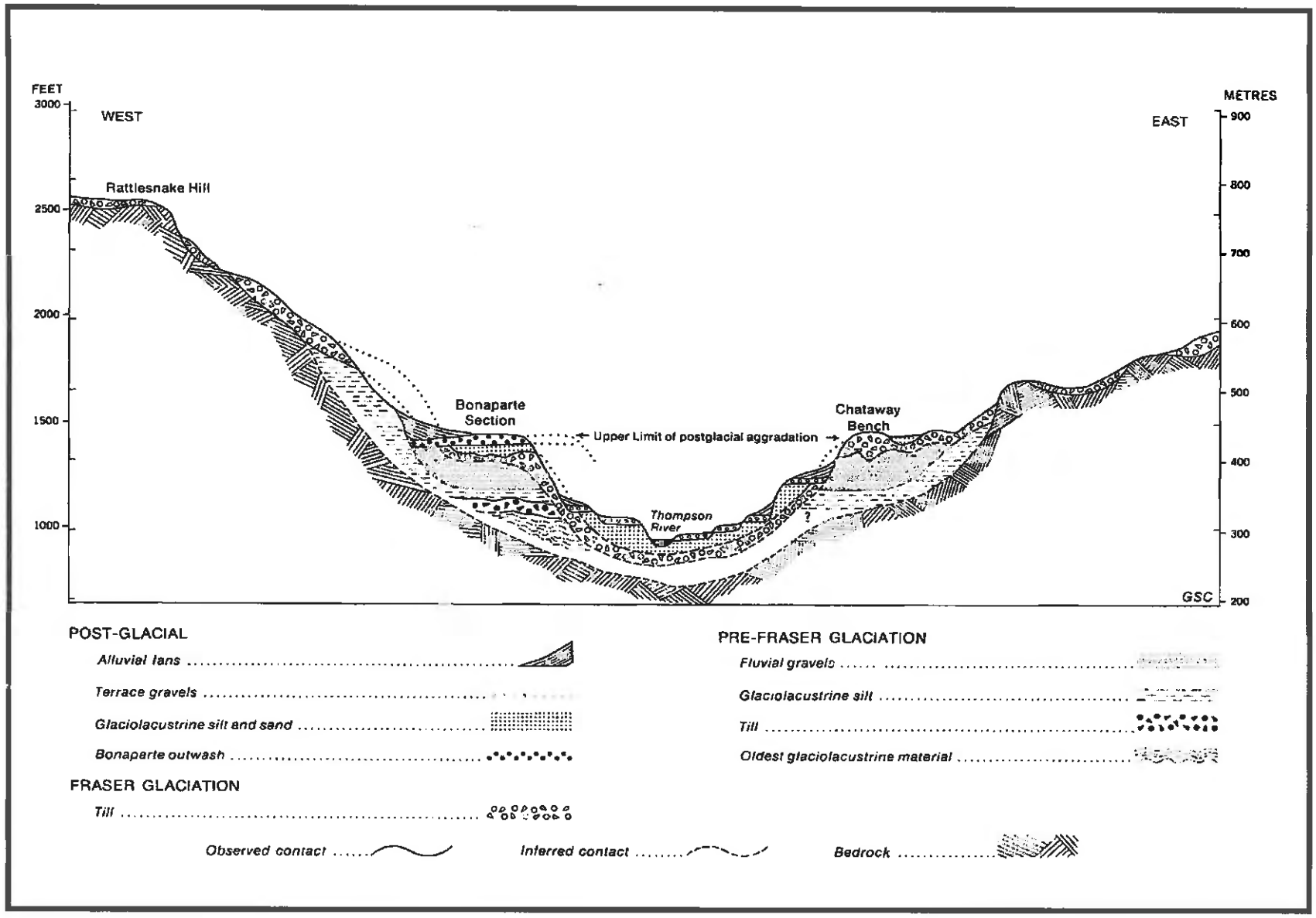


Figure 3.1.6: Schematic diagram illustrating inferred interrelationship of Quarternary sediments in the Thompson Valley near Ashcroft (after Ryder, 1976).

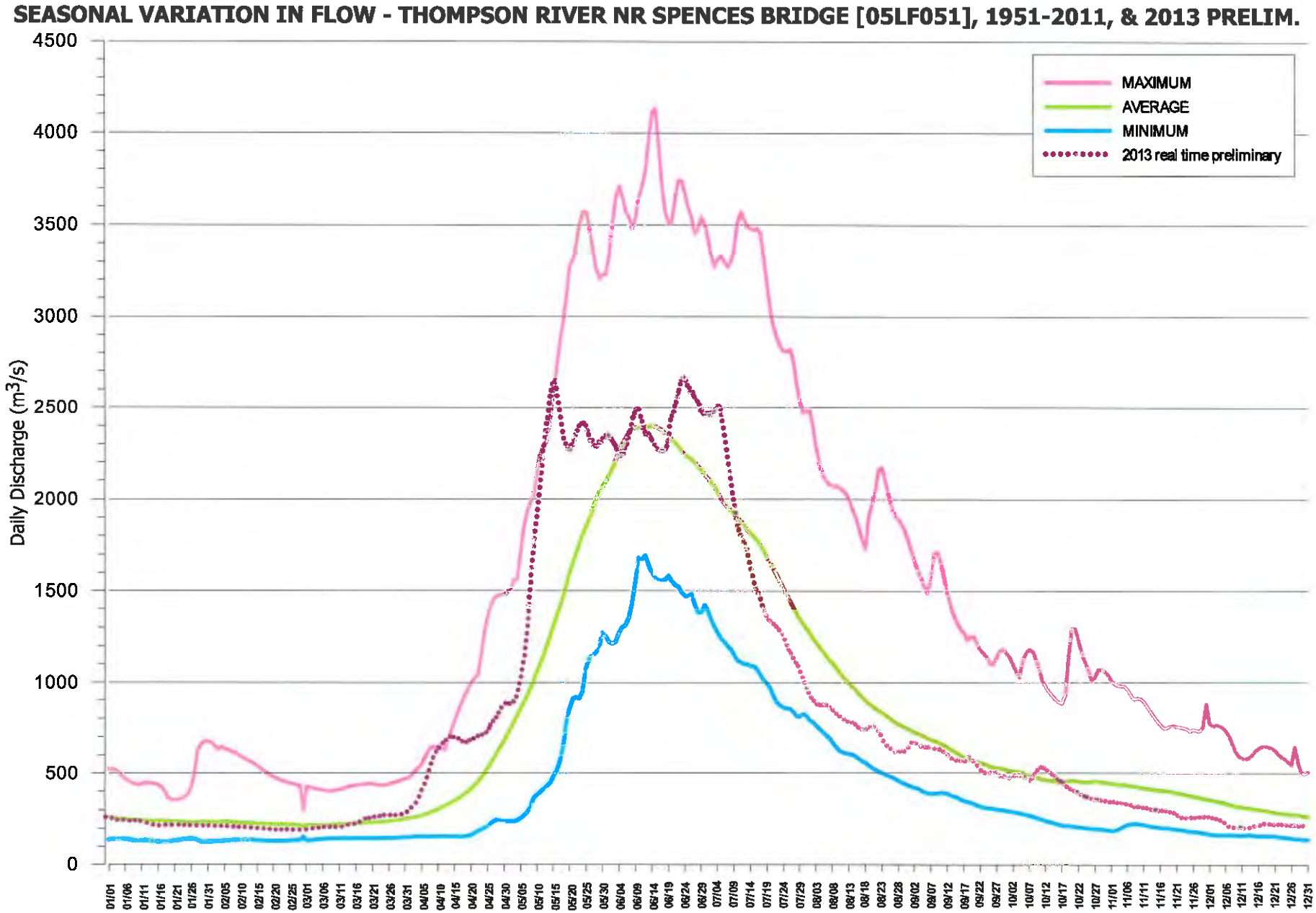
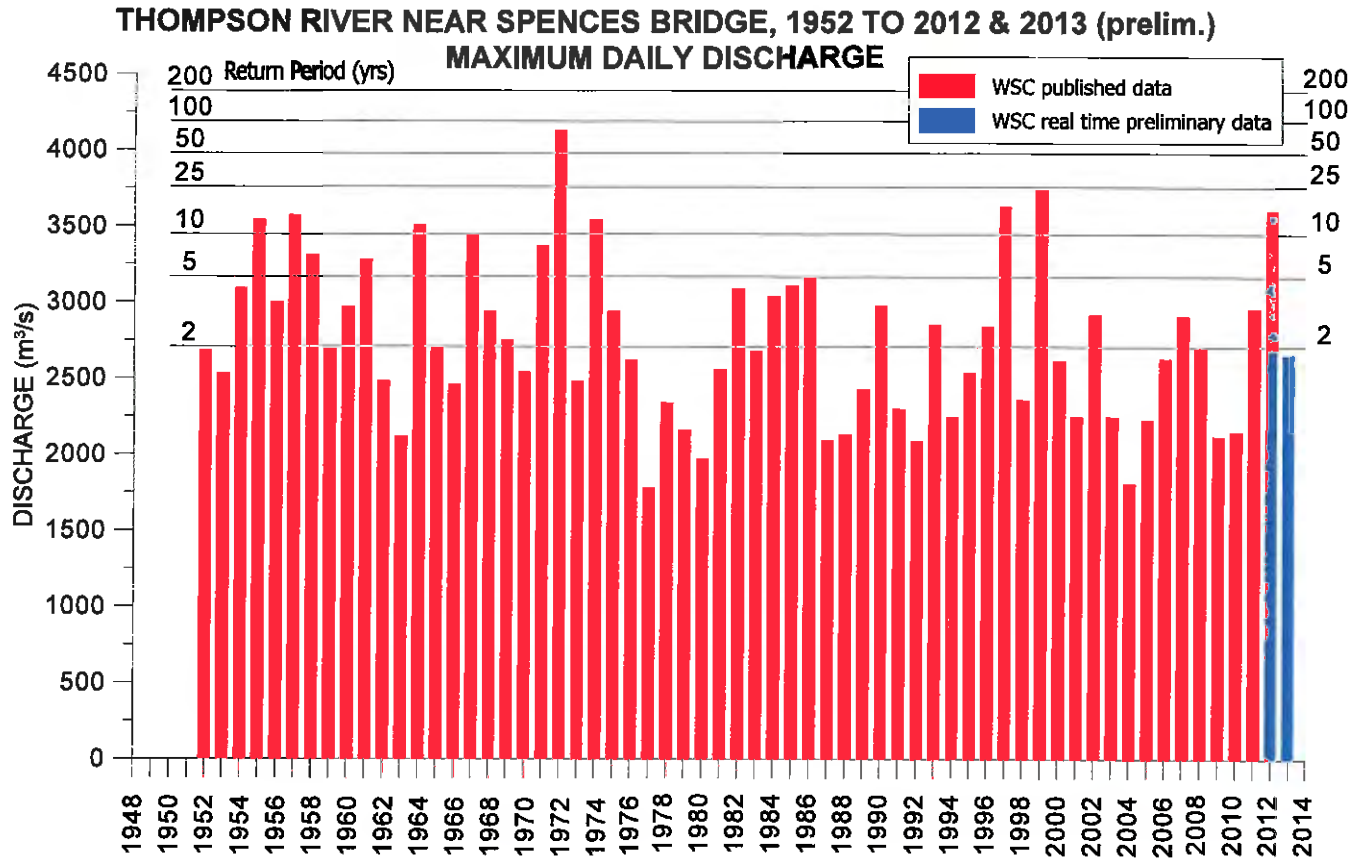


Figure 3.2.1: Seasonal Variation in Flow, Thompson River near Spences Bridge, 1951 to 2011, plus 2013 preliminary.



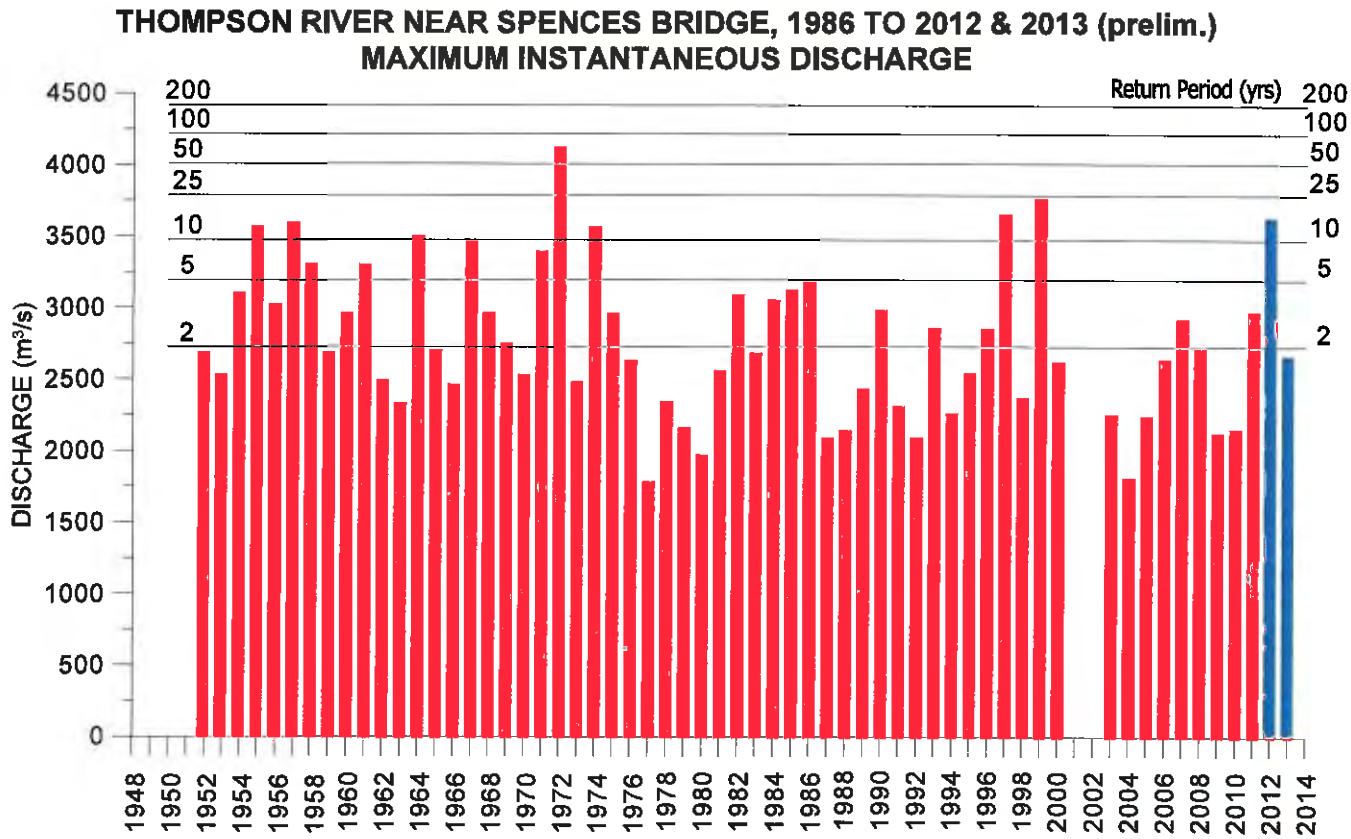


Figure 3.2.2: Historical variation in annual maximum daily and instantaneous discharge, Thompson River near Spences Bridge, 1952 to 2012 & 2013 (prelim.).

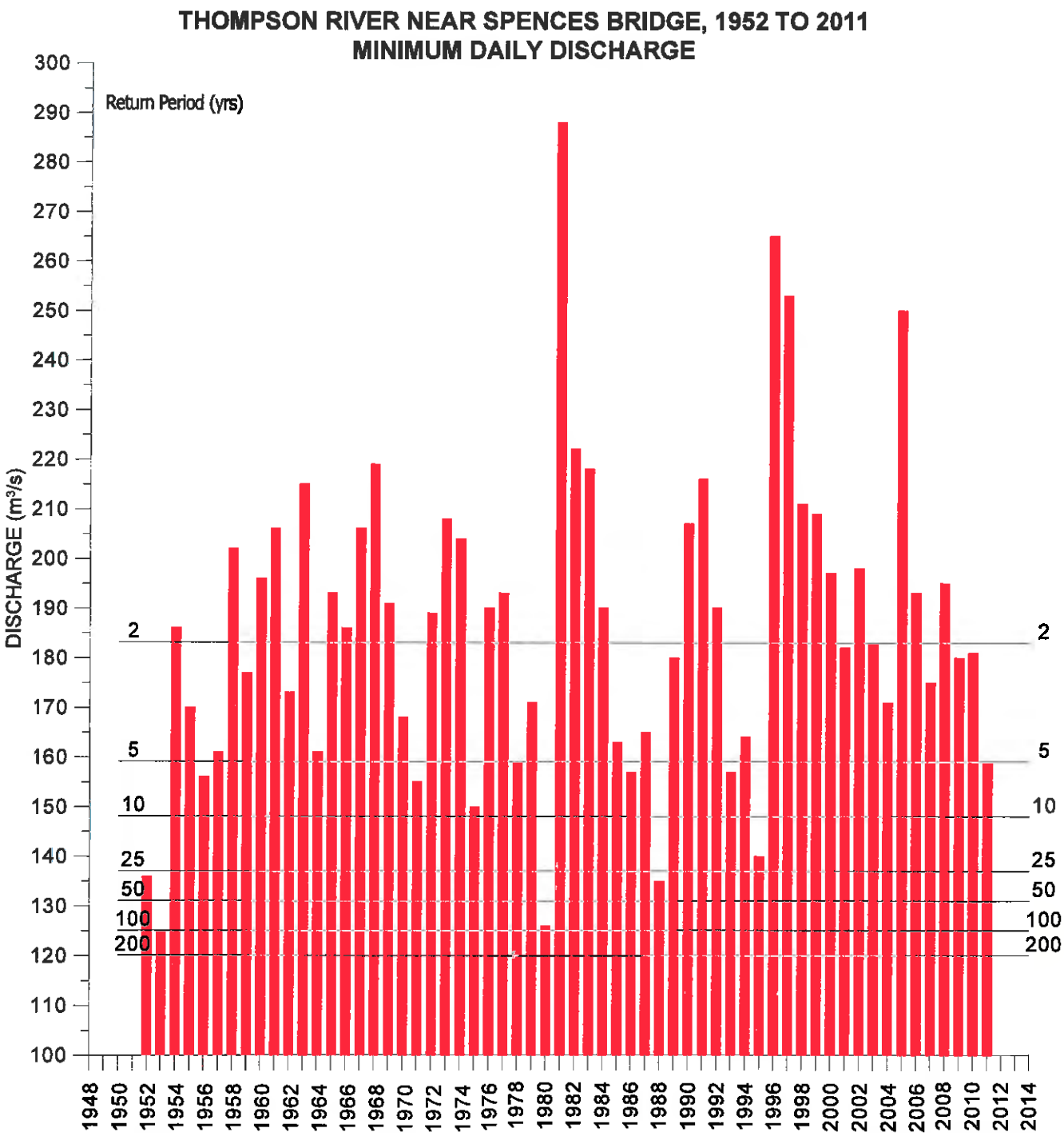


Figure 3.2.3: Historical variation in annual minimum daily discharge, Thompson River near Spences Bridge, 1952 to 2011

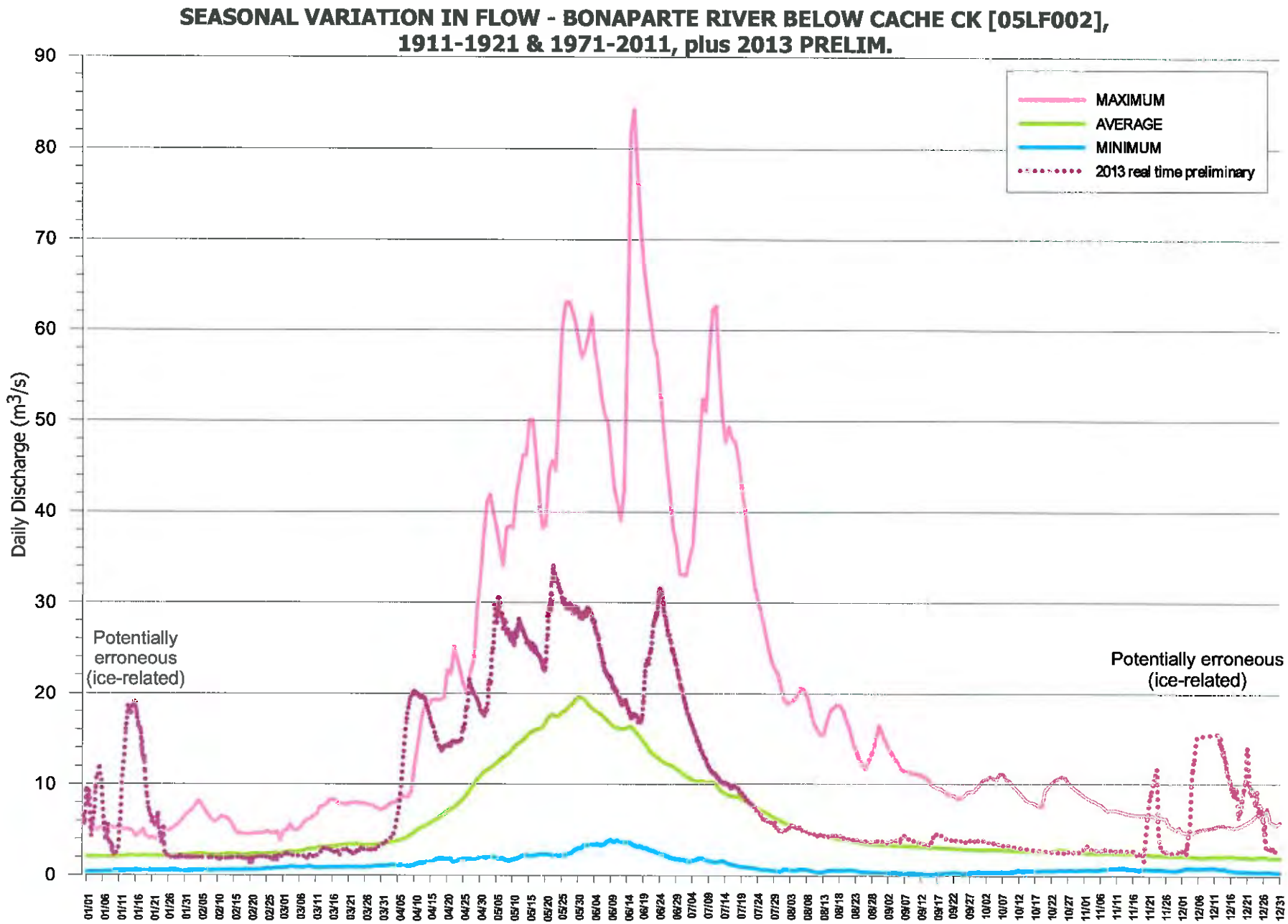
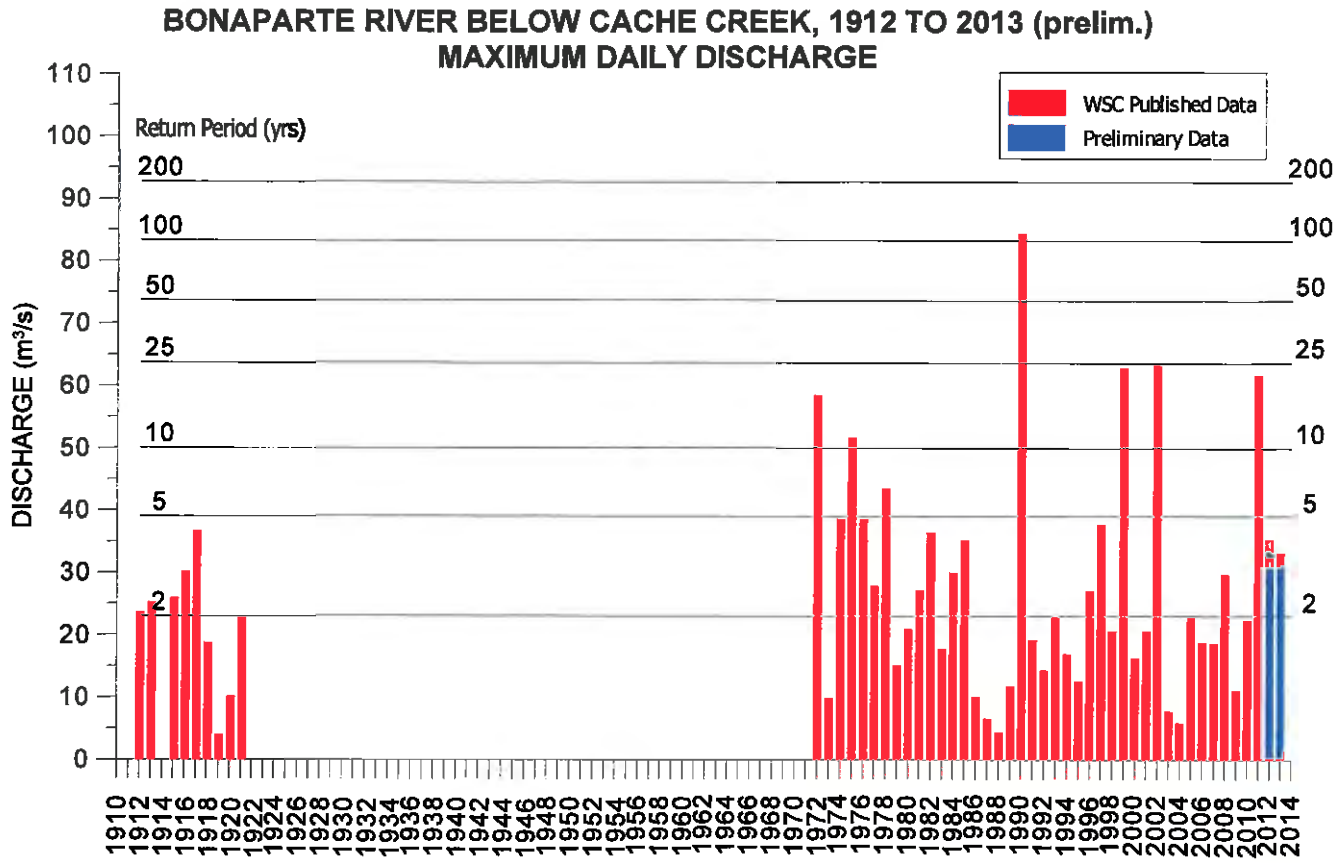


Figure 3.2.4: Seasonal Variation in Flow, Bonaparte River Below Cache Creek, 1911-1921 and 1972 to 2011, plus 2013 preliminary.



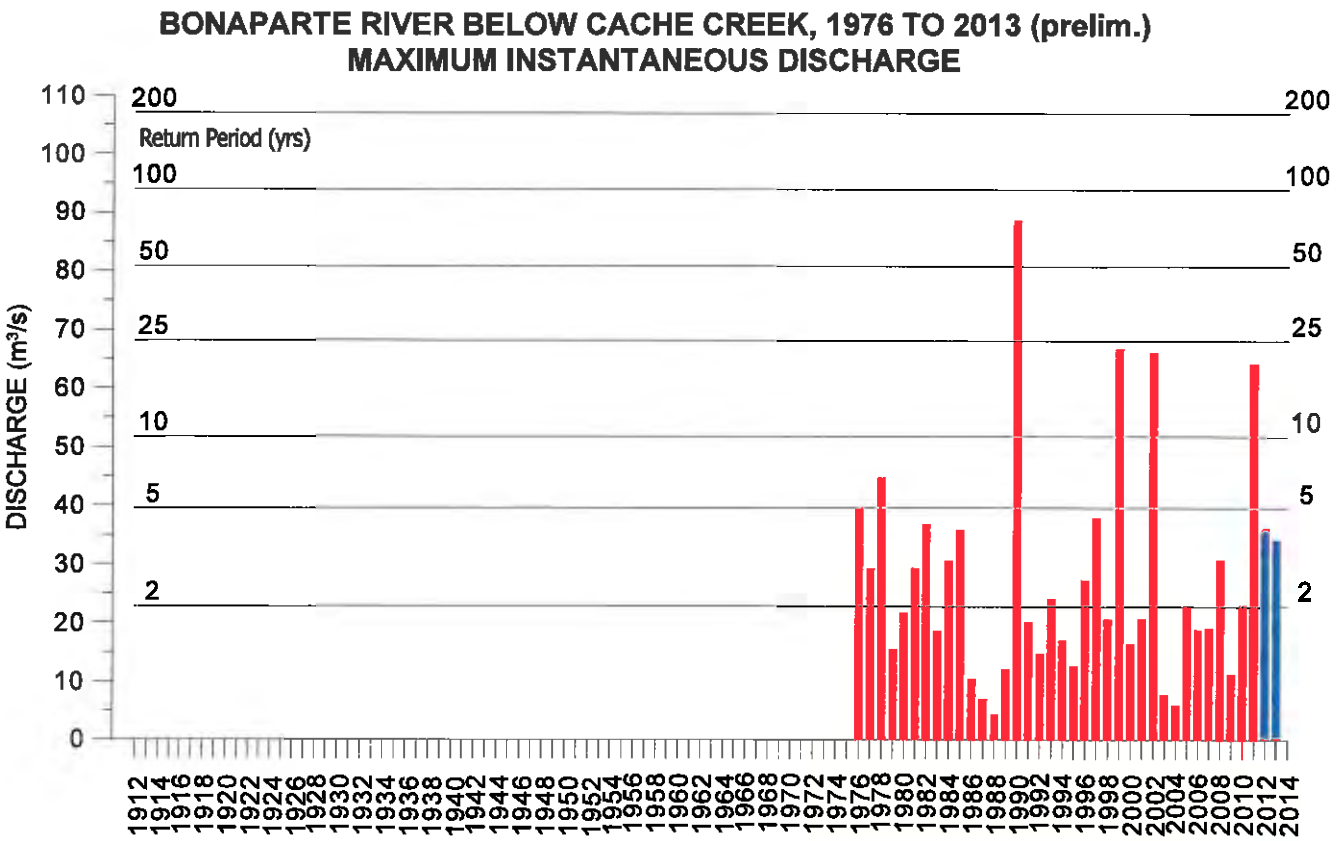


Figure 3.2.5: Historical variation in annual maximum daily and instantaneous discharge, Bonaparte River below Cache Creek, 1912 to 2013 (prelim.).

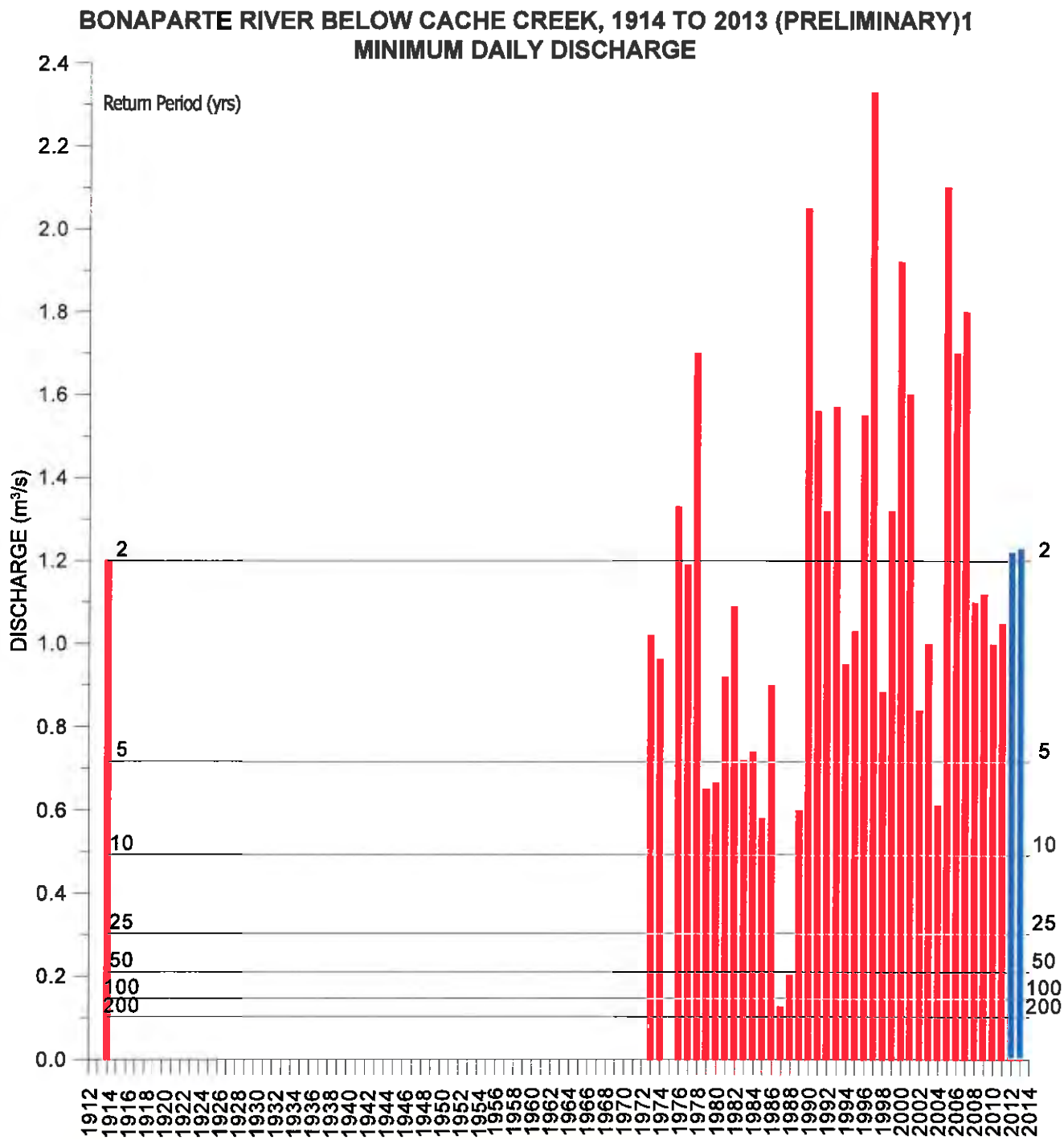


Figure 3.2.6: Historical variation in annual minimum daily discharge, Bonaparte River below Cache Creek, 1912 to 2013 (preliminary).

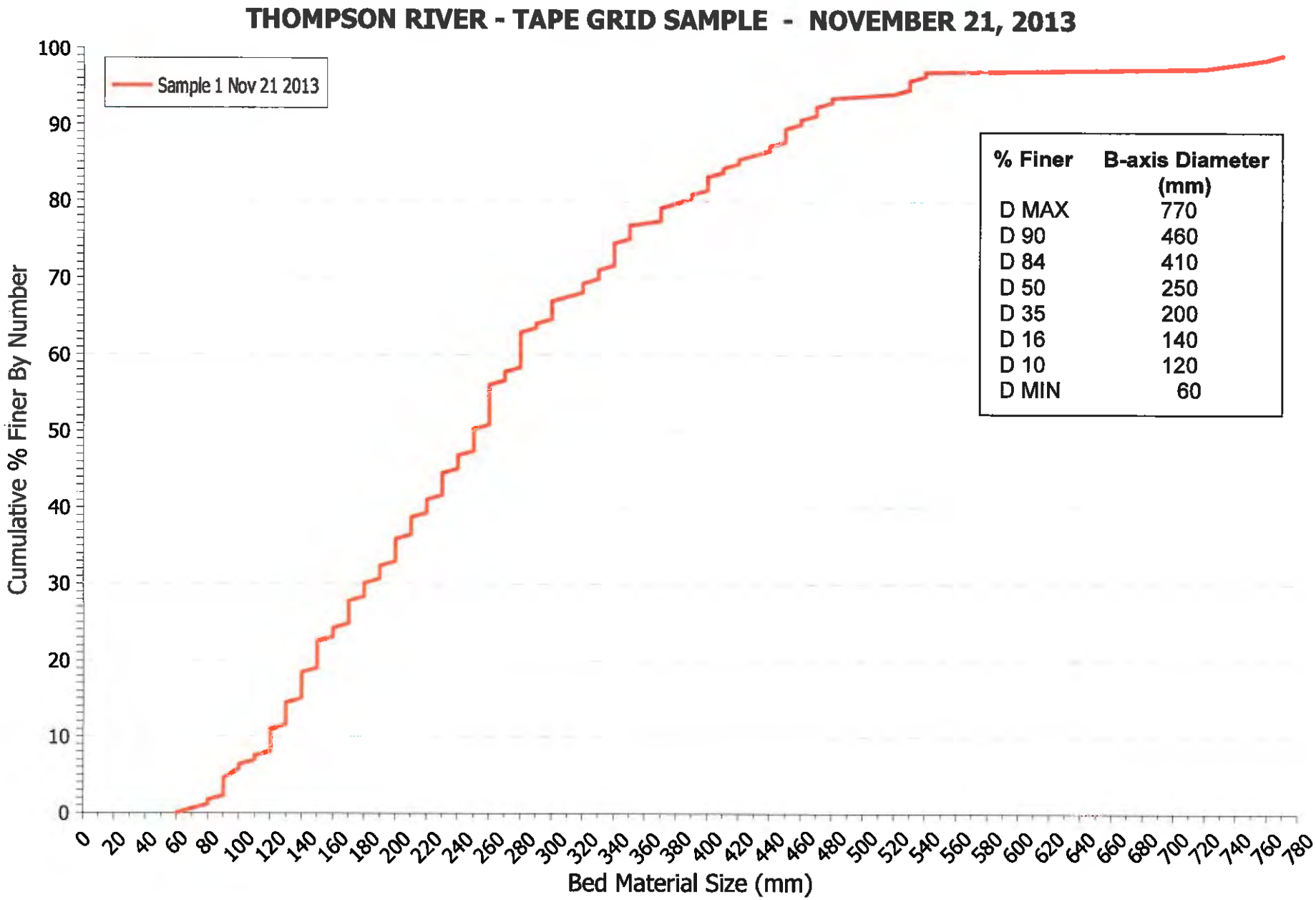
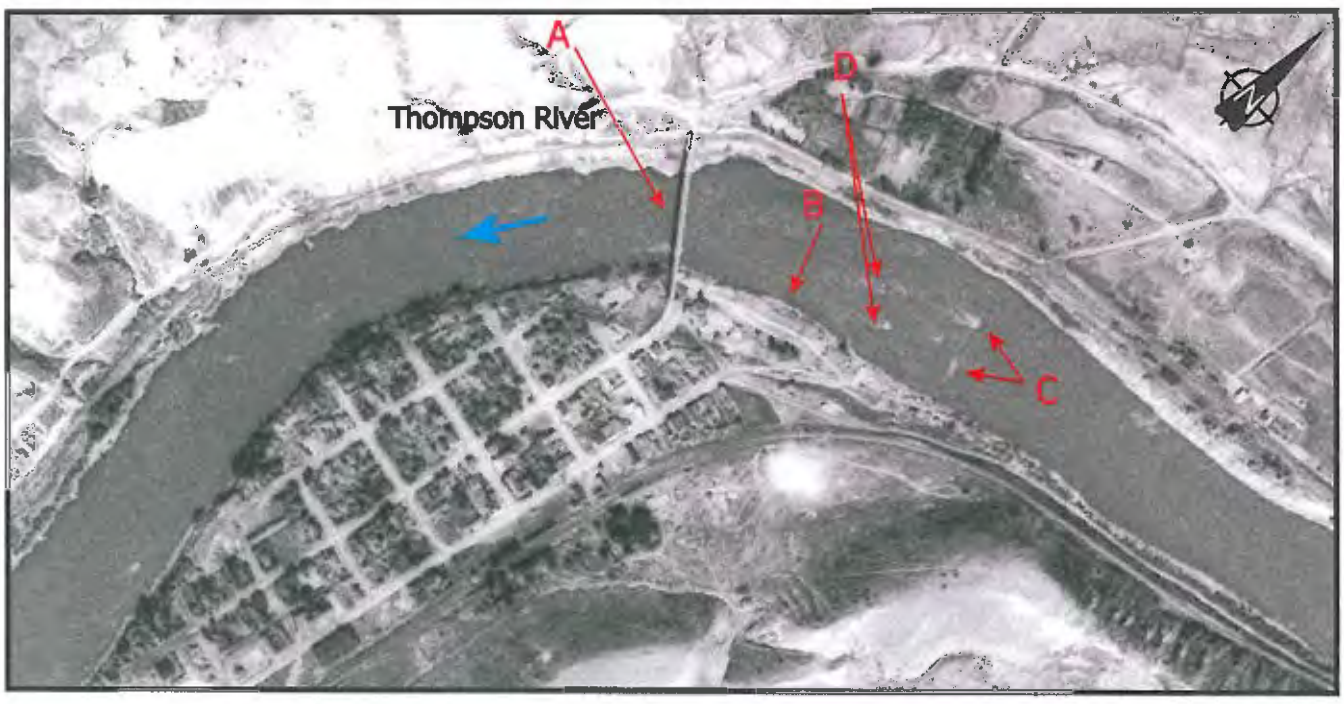


Figure 4.1.1: Bed material size distribution on the left bank downstream of the water supply intake.



(i)

Date: July 31, 1948
BC625 #15

NOTE:

- Highway 97C bridge [A].
- Future location of water supply intake B.
- Pier remnants from former bridge crossing C.
- Large rocks in channel D.

Discharge:

Bonaparte River below Cache Creek	na
Thompson River near Spences Bridge	na

(ii)

Date: July 11, 1959
BC2596 #22

NOTE:

Discharge:

Bonaparte River below Cache Creek	na
Thompson River near Spences Bridge	2,070 m^3/s

(iii)

Date: September 16, 1965
BC5168 #120

NOTE:

- Mid-channel bar [E] beginning to be exposed at a discharge of 518 m^3/s at Spences Bridge.

Discharge:

Bonaparte River below Cache Creek	na
Thompson River near Spences Bridge	518 m^3/s

(iv)

Date: June 18, 1974
BC5597 #261

NOTE:

- Trailer park [F] on bank at future location of water supply intake pumping station.

Discharge:

Bonaparte River below Cache Creek	33.7 m^3/s
Thompson River near Spences Bridge	3,030 m^3/s

Figure 4.2.1A: Historical changes in channel morphology, Thompson River at Ashcroft.

(v)

Date: June 26, 1986
30BC86038 #111

NOTE:

m³/s
Discharge:

Bonaparte River below Cache Creek	5.01 m³/s
Thompson River near Spences Bridge	2,130 m³/s

(vi)

Date: April 11, 1992
30BCC92014 #144

NOTE:

- Mid-channel bar E submerged at a flow of 632 m³/s at Spences Bridge.
- Bank conditions [F] prior to construction of the water supply intake pumping station.

Discharge:

Bonaparte River below Cache Creek	7.51 m³/s
Thompson River near Spences Bridge	632 m³/s

(vii)

Date: July 19, 2000
30BCC00007 #71

NOTE:

- Construction of water supply inlet pumping station and bank protection works [F] which extend into the river channel.

Discharge:

Bonaparte River below Cache Creek	7.50 m³/s
Thompson River near Spences Bridge	2,000 m³/s

(viii)

Date: August 3, 2011
12BCD11304 #264

NOTE:

- Little change in channel or bank conditions over the period since 1948.

Discharge:

Bonaparte River below Cache Creek	9.06 m³/s
Thompson River near Spences Bridge	1,680 m³/s

Figure 4.2.1B: Historical changes in channel morphology, Thompson River at Ashcroft.

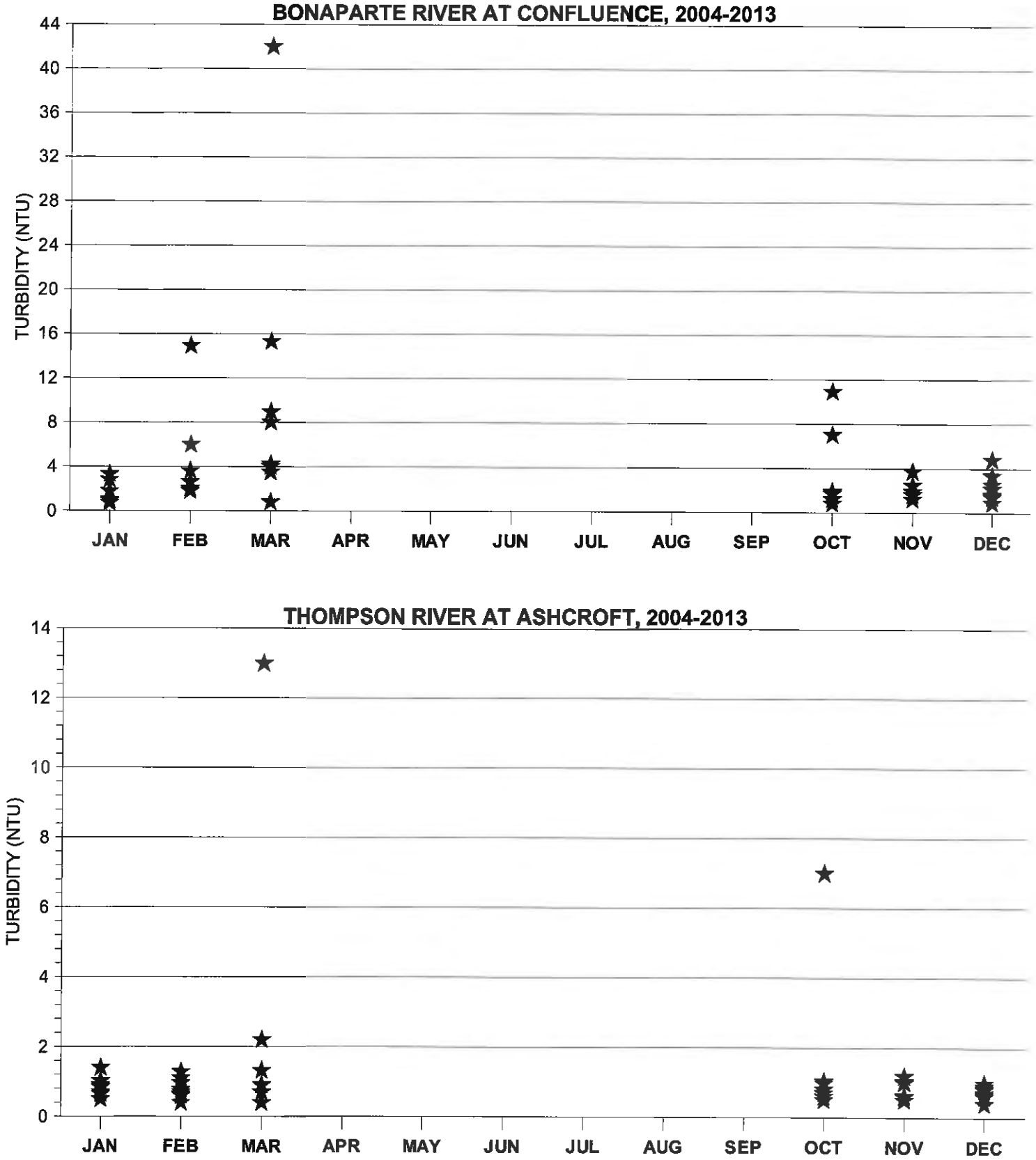


Figure 4.3.1: Seasonal variability in turbidity at Bonaparte River at Confluence and Thompson River at Ashcroft. [Data from BC Ministry of Environment.]

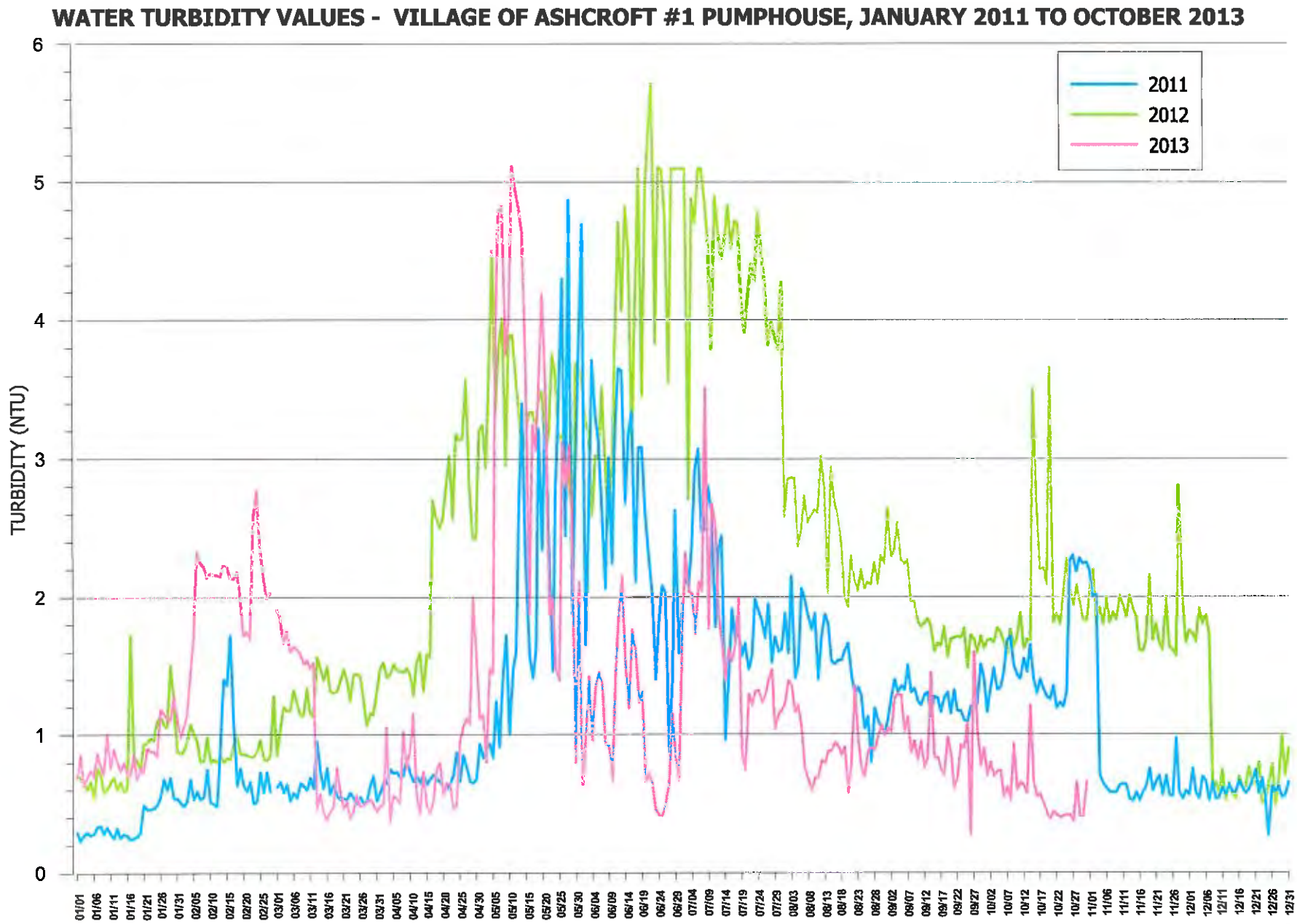


Figure 4.3.2: Water Turbidity Values - Village of Ashcroft #1 Pumphouse, January 1, 2011 to October 31, 2013.

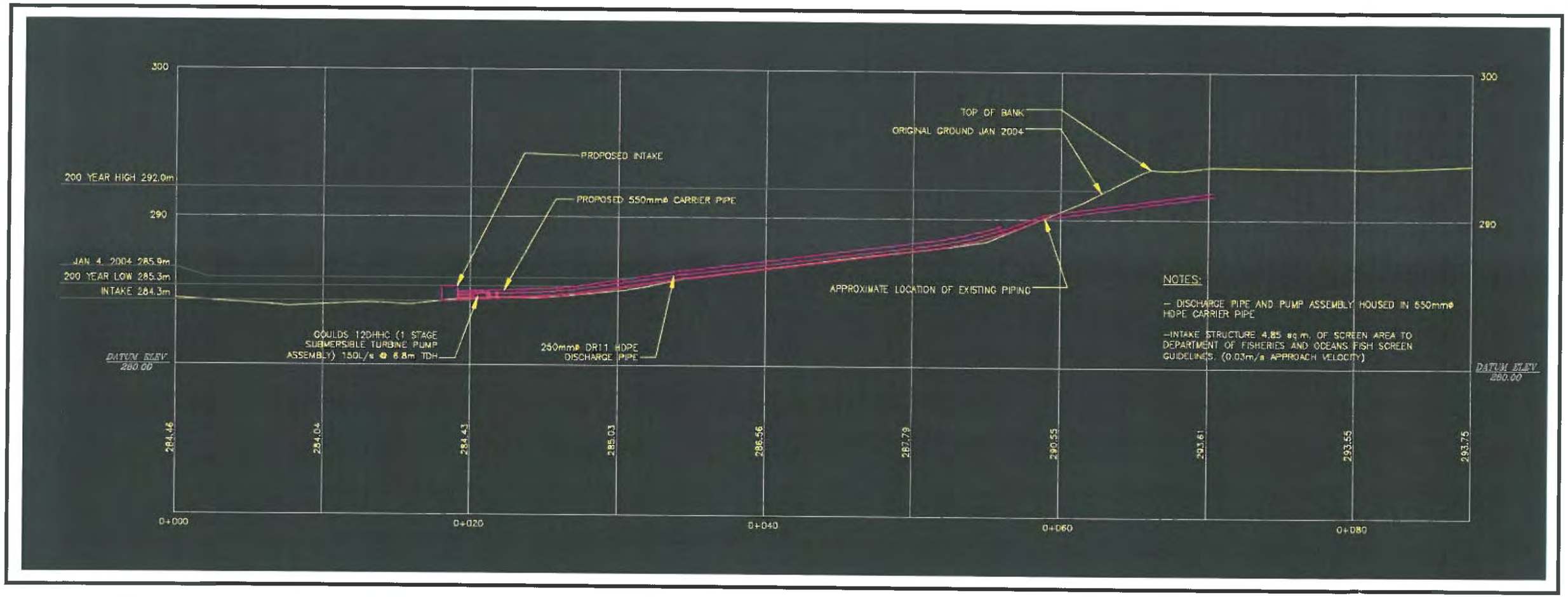


Figure 5.3.1: Design cross-section of the intake pipe installed to augment or replace the infiltration gallery (from Urban Systems Ltd.).

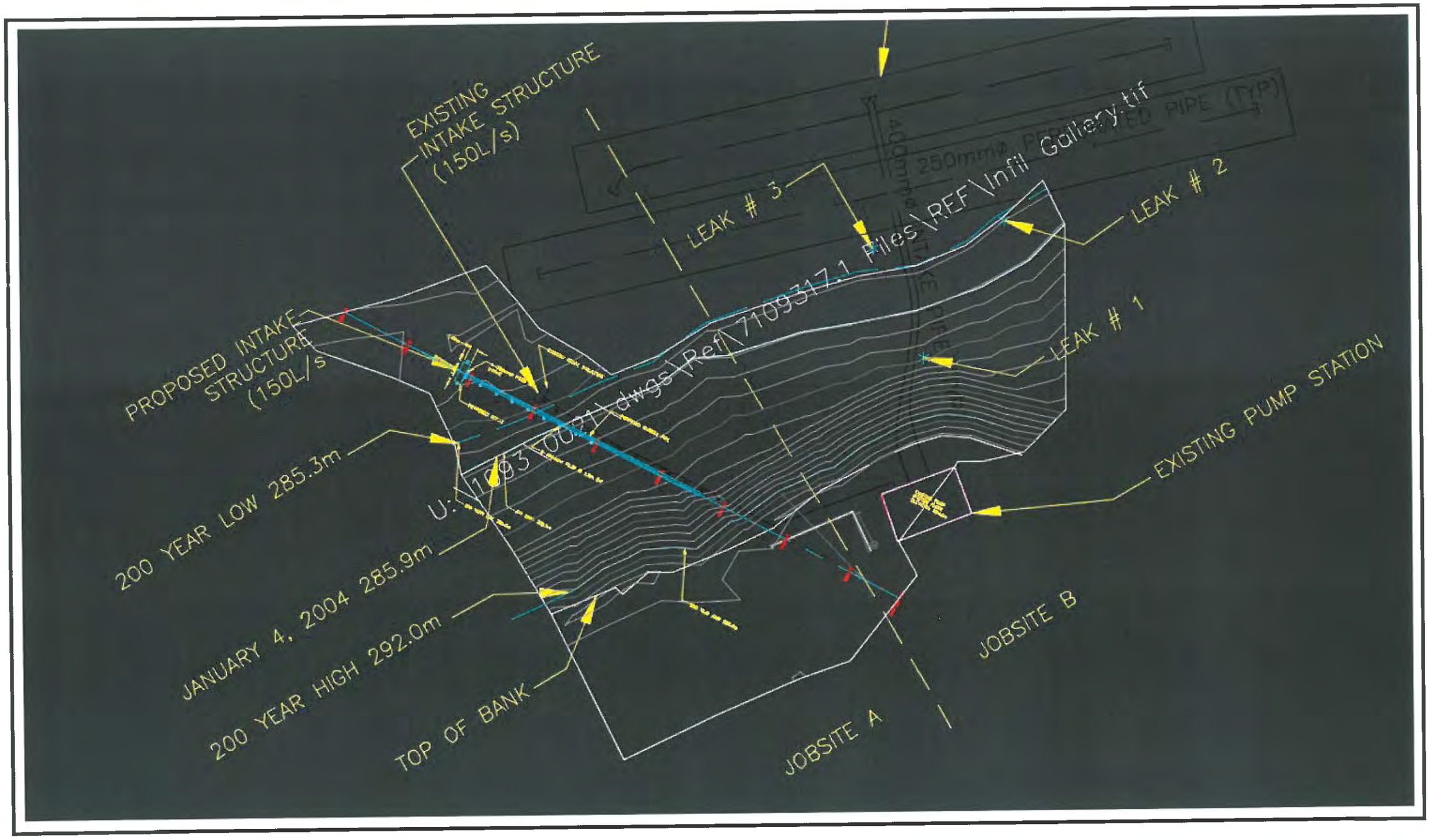


Figure 5.3.2: Design plan of the intake pipe installed to augment or replace the infiltration gallery (from Urban Systems Ltd.).

TABLES

TABLE 3.2.1: FLOOD FREQUENCY ANALYSES - THOMPSON RIVER NEAR SPENCES BRIDGE: MAXIMUM DAILY DISCHARGE, 1952 TO 2013 (PRELIMINARY)

THOMPSON RIVER NEAR SPENCES BRIDGE
MAXIMUM DAILY DISCHARGE

Skew: .374

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1	2
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	See NOTES below	
Log Normal (Maximum Likelihood)	2,700	3,170	3,450	3,770	4,000	4,210	4,420	.0522	.0189
Gumbel (Maximum Likelihood)	2,670	3,180	3,520	3,950	4,260	4,580	4,890	.0622	.0371
Pearson Type III (By Moments)	2,720	3,180	3,440	3,730	3,930	4,110	4,280	.0508	.0139
Log Pearson Type III (By Moments)	2,710	3,170	3,450	3,760	3,980	4,190	4,390	.0523	.0177
Average	2,700	3,180	3,460	3,800	4,040	4,270	4,490		
Adopted Value	2,710	3,170	3,450	3,760	3,980	4,190	4,390		

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	2580	2840	3000	3360	3230	3680	3490	4080	3670	4350	3840	4620	3990	4890
Gumbel (Maximum Likelihood)	2530	2800	2970	3390	3250	3780	3600	4290	3860	4670	4120	5040	4370	5420
Pearson Type III (By Moments)	2590	2850	3010	3350	3230	3650	3480	3980	3640	4210	3790	4430	3930	4630
Log Pearson Type III (By Moments)	2580	2840	3000	3360	3230	3680	3480	4060	3650	4340	3820	4600	3970	4850
Average	2570	2830	2990	3360	3240	3700	3510	4100	3710	4390	3890	4670	4070	4940
Adopted Value	2530	2850	2970	3390	3230	3780	3480	4290	3640	4670	3790	5040	3930	5420

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 3.2.2: FLOOD FREQUENCY ANALYSES - THOMPSON RIVER NEAR SPENCES BRIDGE: MAXIMUM INSTANTANEOUS DISCHARGE, 1952 TO 2013 (PRELIMINARY)

THOMPSON RIVER NEAR SPENCES BRIDGE
MAXIMUM INSTANT. DISCHARGE

Skew: .365

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1 2 See NOTES below
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	
Log Normal (Maximum Likelihood)	2,730	3,200	3,480	3,800	4,030	4,240	4,440	.0485 .0175
Gumbel (Maximum Likelihood)	2,690	3,210	3,550	3,980	4,310	4,620	4,940	.0611 .0367
Pearson Type III (By Moments)	2,750	3,210	3,470	3,760	3,960	4,140	4,310	.0607 .0127
Log Pearson Type III (By Moments)	2,730	3,200	3,480	3,790	4,010	4,220	4,420	.0508 .0165
Average	2,720	3,210	3,490	3,840	4,080	4,310	4,530	
Adopted Value	2,730	3,200	3,480	3,790	4,010	4,220	4,420	

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	2600	2870	3020	3390	3260	3720	3520	4110	3690	4390	3860	4660	4020	4920
Gumbel (Maximum Likelihood)	2550	2830	3000	3420	3280	3820	3630	4340	3890	4720	4150	5100	4400	5480
Pearson Type III (By Moments)	2610	2880	3030	3380	3260	3680	3500	4020	3670	4250	3820	4460	3960	4670
Log Pearson Type III (By Moments)	2600	2870	3020	3390	3250	3720	3510	4100	3680	4380	3840	4640	3990	4890
Average	2590	2860	3020	3400	3260	3740	3540	4140	3730	4430	3920	4720	4090	4990
Adopted Value	2550	2880	3000	3420	3250	3820	3500	4340	3670	4720	3820	5100	3960	5480

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 3.2.3: LOW FLOW FREQUENCY ANALYSIS - THOMPSON RIVER NEAR SPENCES BRIDGE: MINIMUM DAILY DISCHARGE, 1952 TO 2011

THOMPSON RIVER NEAR SPENCES BRIDGE
MINIMUM ANNUAL DISCHARGE

Skew: .695

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1 2 See NOTES below
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	
Log Normal (Maximum Likelihood)	183	159	148	137	130	125	120	.0538 .0239
Gumbel (Maximum Likelihood)	182	158	148	140	136	134	132	.0664 .0412
Pearson Type III (By Moments)	182	159	148	139	133	129	125	.0596 .0308
Log Pearson Type III (By Moments)	183	159	148	137	131	125	120	.0538 .0247
Average	183	159	148	138	133	128	124	
Adopted Value	183	159	148	137	131	125	120	

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	176	191	151	167	140	157	128	147	121	141	115	135	110	131
Gumbel (Maximum Likelihood)	2550	2830	3000	3420	3280	3820	3630	4340	3890	4720	4150	5100	4400	5480
Pearson Type III (By Moments)	175	190	151	167	140	157	129	149	122	144	117	140	113	137
Log Pearson Type III (By Moments)	175	191	152	167	140	157	128	147	121	141	115	136	110	131
Average	770	851	863	981	925	1070	1000	1190	1060	1290	1120	1380	1180	1470
Adopted Value	175	2830	151	3420	140	3820	128	4340	121	4720	115	5100	110	5480

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 3.2.4: FLOOD FREQUENCY ANALYSES - BONAPARTE RIVER BELOW CACHE CREEK: MAXIMUM DAILY DISCHARGE, 1912 TO 2013 (PRELIMINARY)

BONAPARTE RIVER BELOW CACHE CK
MAXIMUM DAILY DISCHARGE

Skew: 1.28

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1 2 See NOTES below
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	
Log Normal (Maximum Likelihood)	22.8	38.3	49.5	64.6	76.5	89.0	102	.0544 .0315
Gumbel (Maximum Likelihood)	23.8	37.6	46.8	58.4	67.0	75.5	84.0	.0496 .0496
Pearson Type III (By Moments)	23.2	39.0	49.6	62.7	72.1	81.4	90.4	.0586 .0352
Log Pearson Type III (By Moments)	23.0	39.0	50.0	63.7	73.7	83.3	92.7	.0583 .0320
Average	23.2	38.5	49.0	62.3	72.3	82.3	92.3	
Adopted Value	23.0	39.0	50.0	63.7	73.7	83.3	92.7	

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	19.0	27.2	31.4	46.3	39.6	61.4	50.1	82.9	57.9	101	65.9	120	73.9	140
Gumbel (Maximum Likelihood)	19.7	27.8	31.4	43.8	38.8	54.7	48.1	68.7	54.9	79.1	61.6	89.4	68.3	99.7
Pearson Type III (By Moments)	18.9	27.4	31.4	46.7	39.2	60.0	48.6	76.8	55.3	89.0	61.9	101	68.3	113
Log Pearson Type III (By Moments)	18.8	27.9	30.0	50.7	36.9	67.7	45.1	90.1	50.7	107	56.0	124	61.1	141
Average	19.1	27.6	31.1	46.9	38.6	61.0	47.9	79.6	54.7	93.9	61.3	108	67.9	123
Adopted Value	18.8	27.9	30.0	50.7	36.9	67.7	45.1	90.1	50.7	107	56.0	124	61.1	141

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 3.2.5: FLOOD FREQUENCY ANALYSES - BONAPARTE RIVER BELOW CACHE CREEK: MAXIMUM INSTANTANEOUS DISCHARGE, 1912 TO 2013 (PRELIMINARY)

BONAPARTE RIVER BELOW CACHE CK
MAXIMUM INSTANT. DISCHARGE

Skew: 1.53

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1	2
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	See NOTES below	
Log Normal (Maximum Likelihood)	22.7	39.1	51.6	69.1	83.4	98.6	115	.0521	.0428
Gumbel (Maximum Likelihood)	24.2	38.3	47.7	59.5	68.2	76.9	85.6	.0731	.0667
Pearson Type III (By Moments)	22.9	40.2	52.2	67.4	78.6	89.7	101	.0601	.0456
Log Pearson Type III (By Moments)	22.9	39.6	51.8	68.2	80.9	94.0	107	.0530	.0443
Average	23.2	39.3	50.8	66.1	77.8	89.8	102		
Adopted Value	22.9	39.6	51.8	68.2	80.9	94.0	107		

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	18.2	28.2	30.7	49.8	39.1	68.1	49.9	95.4	58.2	119	66.7	145	75.5	174
Gumbel (Maximum Likelihood)	19.4	29.0	30.9	45.7	38.2	57.1	47.2	71.7	53.8	82.6	60.4	93.5	66.9	104
Pearson Type III (By Moments)	17.7	28.0	30.1	50.2	37.9	66.5	47.5	87.4	54.5	103	61.3	118	68.1	133
Log Pearson Type III (By Moments)	18.3	28.6	29.8	52.5	37.3	71.9	46.7	99.6	53.6	122	60.3	146	67.0	172
Average	18.4	28.5	30.4	49.6	38.1	65.9	47.8	88.5	55.0	107	62.2	126	69.4	146
Adopted Value	17.7	29.0	29.8	52.5	37.3	71.9	46.7	99.6	53.6	122	60.3	146	66.9	174

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 3.2.6: LOW FLOW FREQUENCY ANALYSIS - BONAPARTE RIVER BELOW CACHE CREEK: MINIMUM DAILY DISCHARGE, 1952 TO 2013 (PRELIMINARY)

BONAPARTE RIVER BELOW CACHE CK
MINIMUM DAILY DISCHARGE

Skew: .326

Frequency Distribution	Estimate of Specified Recurrence Interval Discharge in m3/second							Goodness of fit 1	2
	2 years	5 years	10 years	25 years	50 years	100 years	200 years	See NOTES below	
Log Normal (Maximum Likelihood)	1.13	.739	.549	.357	.238	.135	.0429	.327	.327
Gumbel (Maximum Likelihood)	1.13	.719	.532	.361	.269	.200	.147	.0695	.0594
Pearson Type III (By Moments)	1.13	.737	.546	.354	.235	.132	.0406	.326	.326
Log Pearson Type III (By Moments)	1.20	.716	.492	.303	.211	.147	.103	.324	.324
Average	1.15	.728	.530	.343	.238	.153	.0834		
Adopted Value	1.20	.716	.492	.303	.211	.147	.103		

95% Confidence Limits for Specified Recurrence Interval in m3/second

Frequency Distribution	2 years		5 years		10 years		25 years		50 years		100 years		200 years	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
Log Normal (Maximum Likelihood)	.979	1.29	.580	.905	.374	.733	.162	.563	.0304	.459	-.0843	.369	-.186	.290
Gumbel (Maximum Likelihood)	2550	2830	3000	3420	3280	3820	3630	4340	3890	4720	4150	5100	4400	5480
Pearson Type III (By Moments)	.977	1.28	.573	.901	.361	.732	.139	.568	.51e-5	.470	-.122	.385	-.230	.311
Log Pearson Type III (By Moments)	.960	1.50	.628	.817	.394	.614	.203	.452	.123	.363	.0741	.292	.0449	.236
Average	639	709	750	856	820	957	908	1080	973	1180	1040	1280	1100	1370
Adopted Value	.960	2830	.573	3420	.361	3820	.139	4340	.51e-5	4720	-.122	5100	-.230	5480

1 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for all points.

2 Modified Kolmogorov-Smirnov goodness of fit test based on unbiased plotting positions for 5 largest points.

Analytical procedures used to prepare this summary were made available by the River Forecast Centre, Water Management Branch, B.C. Ministry of Environment. This assistance is gratefully acknowledged.

TABLE 4.3.1: BC MINISTRY OF ENVIRONMENT WATER QUALITY DATA FROM THOMPSON RIVER AT ASHCROFT [0600325]

Sampling Date	Field Temperature	Field Turbidity	Conductivity	Alkalinity (Total as CaCO3)	pH	True Colour	Nitrate plus Nitrite (N)	Total Nitrogen (N)	Total Phosphorus (P)	Dissolved Phosphorus (P)	Orthophosphate (P)	E. Coli
dd-mmm-yy	(°C)	(NTU)	(uS/cm)	(mg/L)	(pH units)	(Col. Unit)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(CFU/100mL)
19-Oct-04	11.7	0.7	82		7.6	< 5	0.072	0.15	0.007	0.004	0.003	< 1
09-Nov-04	10.3	0.62	90		7.8	< 5	0.102	0.16	0.004	< 0.002	< 0.001	< 1
14-Dec-04	5.1	0.64	98		7.7	< 5	0.148	0.24	0.007	0.004	0.004	1
12-Jan-05	2.9	0.65	97		7.7	< 5	0.145	0.20	0.007	0.006	0.002	< 1
22-Feb-05	0.4	1.1	98.7		7.8	5	0.119	0.25	0.009	0.007	0.004	3
15-Mar-05	4.1	0.9	98		7.8	< 5	0.123	0.23	0.006	0.003	0.006	5
24-Oct-05	14.0	0.5	93		7.8	5	0.058	0.12	0.002	0.004	0.005	< 1
22-Nov-05	6.7	0.6	88		7.7	10	0.102	0.17	< 0.002	0.003	0.004	< 1
20-Dec-05	4.7	0.4	98		7.8	5	0.127	0.21	0.004	0.009	0.006	2
24-Jan-06	3.3	0.5	101		7.3	< 5	0.105	0.21	0.011	0.006	0.005	< 1
14-Feb-06	3.2	0.4	100		7.5	5	0.125	0.20	0.009	0.006	0.004	< 1
08-Mar-06	2.9	0.4	106		7.7	< 5	0.118	0.22	0.008	0.006	0.006	< 1
10-Oct-06	9.7	0.6	90		7.9	5	0.062	0.14	0.007	0.003	< 0.001	5
07-Nov-06	9.7	0.5	98		7.7	< 5	0.092	0.19	< 0.002	0.003	< 0.001	
06-Dec-06	4.8	0.7	109		7.8	< 5	0.159	0.24	0.006	0.004	0.003	< 1
04-Jan-07	3.0	0.7	108		7.8	< 5	0.157	0.24	< 0.002	0.003	0.005	1
30-Jan-07	1.7	0.9	108		7.8	< 5	0.174	0.34	0.027	0.027	0.027	< 1
27-Feb-07	2.1	0.6	111		7.8	< 5	0.168	0.22	0.008	0.007	0.003	< 1
27-Mar-07	3.1	0.7	111		7.9	< 5	0.149	0.25	0.007	0.005	0.002	< 1
09-Oct-07			90	34	7.9	< 5	0.061	0.15	0.004	0.005	0.003	3
06-Nov-07			92		7.9	< 5	0.10	0.15	0.005	0.006	0.005	1
04-Dec-07	5.2	0.89	99		7.9	< 5	0.14	0.20	0.005	0.007	0.003	< 1
08-Jan-08	2.0	0.86	100	37	7.8	5	0.14	0.27	0.006	0.005	0.004	1
05-Feb-08	0.5	0.77	100	38	7.9	< 5	0.14	0.25	0.003	0.004	0.002	< 1
27-Feb-08	2.9	0.64	100	42	7.8	5	0.14	0.22	0.006	0.005	0.003	< 1
25-Mar-08	3.1	2.2	100	42	7.8	< 5	0.13	0.20	0.007	0.007	0.003	1
15-Oct-08	12.0	0.8	87	34	7.7	5	0.05	0.12	0.005	0.004	0.001	1
12-Nov-08	9.8	1	93	36	7.8	5	0.09	0.14	0.004	0.003	0.004	1
10-Dec-08	5.3	0.6	97	37	7.5	< 5	0.12	0.18	0.003	0.005	0.001	< 1
06-Jan-09	1.0	1.4	97	38	7.9	< 5	0.21	0.17	0.004	0.003	0.003	92
02-Feb-09	1.3	0.7	99	40	7.9	< 5	0.13	0.20	0.005	0.005	0.002	1
03-Mar-09	2.7	13	100	40	7.9	< 5	0.14	0.25	0.006	0.006	0.001	< 1
31-Mar-09	1.9	<0.1	110	40	7.8	< 5	0.14	0.22	0.005	0.005	0.002	1
28-Oct-09	8.7	0.8	117	38	7.8	10	0.09	0.16	0.005	0.002	0.001	< 1
24-Nov-09	6.4	0.6	100	40	7.7	5	0.10	0.22	0.003	0.003	0.001	< 1
16-Dec-09	3.2	0.8	564	220	7.4	15	0.13	0.27	0.004	0.003	< 0.001	1
19-Jan-10			107	42	7.9	5	0.15	0.29	0.003	0.003	0.001	2
16-Feb-10			110	41	7.7	5	0.15	0.16	0.056	0.046	0.006	1
16-Mar-10			112	45	7.9	5	0.15	0.2	0.005	0.003	0.002	1
26-Oct-10			90	36	7.78	5	0.06	0.14	0.006	< 0.002	0.002	< 2
23-Nov-10			106	40	7.83	5	0.16	0.27	0.004	0.003	0.001	2
14-Dec-10			106	39	7.49	5	0.16	0.23	0.005	0.004	0.002	2
18-Jan-11			487	200	7.87	< 5	0.22	0.42	0.026	0.021	0.018	100
15-Feb-11			106	38	7.37	5	0.17	0.25	0.006	0.002	0.001	57
08-Mar-11			109	39	7.36	5	0.17	0.32	0.005	0.003	< 0.001	< 1
25-Oct-11	9.90	0.98	82	32	7.41	< 5	0.10	0.30	0.004	0.003	< 0.001	2
22-Nov-11	-	-	101	39.4	7.69	< 5	0.14	0.38	0.005	0.004	0.002	1
19-Dec-11	4.50	0.66	98	33	7.27	< 5	0.15	0.29	0.004	0.003	0.002	1
17-Jan-12	2.30	0.84	101	39	7.71	5	0.16	0.16	0.007	0.006	0.002	NA
15-Feb-12	-	0.96	101	40	7.74	-	0.16	0.28	0.013	0.009	0.007	< 1
13-Mar-12	3.40	1.32	107	40	7.84	10	0.15	0.28	0.005	0.003	0.002	< 1
16-Oct-12	14.6	7.00	87	32	7.45	< 5	0.07	0.26	0.015	0.016	0.002	2
13-Nov-12	11	1.18	92	35	7.77	< 5	0.11	0.24	0.007	0.006	0.003	< 1
11-Dec-12	8.3	0.84	96	38	7.80	5	0.15	0.23	0.006	0.003	0.002	< 1
08-Jan-03	5	1.02	99	39	7.82	< 5 (1)	0.15 (1)	0.24	0.006	0.004	0.002 (1)	< 1
05-Feb-13	5.1	1.28	102	41	7.80	5	0.14	0.21	0.009	0.004	0.003	4
05-Mar-13	3.5	0.91	102	41	7.76	10	0.14	0.22	0.005	0.003	0.002	< 1
15-Oct-13	12.3	1.03	86.2	35	7.78	5	0.06	0.15	0.009	0.005	0.002	1
11-Nov-13	12.3	1.03	86.1	34	7.57	5	0.09	0.16	0.004	0.004	0.002	3
10-Dec-13	3.8	0.98	102	41	7.83	5	0.14	0.22	0.006	0.005	0.004	< 1

TABLE 4.3.2: BC MINISTRY OF ENVIRONMENT WATER QUALITY DATA FROM BONAPARTE RIVER AT THE CONFLUENCE WITH THOMPSON RIVER [0600329]

Sampling Date	Field Temperature	Field Turbidity	Conductivity	Alkalinity (Total as CaCO3)	pH	True Colour	Nitrate plus Nitrite (N)	Total Nitrogen (N)	Total Phosphorus (P)	Dissolved Phosphorus (P)	Orthophosphate (P)	E. Coli
dd-mmm-yy	(°C)	(NTU)	(uS/cm)	(mg/L)	(pH units)	(Col. Unit)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(mg/L)	(CFU/100mL)
19-Oct-04	6.0	1.82	359		8.2	< 5	0.006	0.20	0.016	0.006	0.004	290
09-Nov-04	10.5	1.60	386		8.4	5	0.005	0.23	0.010	0.003	0.007	28
14-Dec-04	0.3	3.34	408		8.3	< 5	0.080	0.25	0.021	0.013	0.004	N/R
12-Jan-05	0.0	1.77	415		8.2	10	0.161	0.40	0.028	0.025	0.026	11
22-Feb-05	2.2	2.60	345		8.3	20	0.111	0.41	0.029	0.02	0.017	4
15-Mar-05	3.6	15.30	306		8.2	10	0.049	0.56	0.072	0.018	0.018	18
24-Oct-05	10.0	10.90	312		8.3	15	0.004	0.44	0.032	0.015	0.010	23
22-Nov-05	4.0	3.70	311		8.3	15	0.039	0.26	0.016	0.013	0.013	17
20-Dec-05	1.5	1.40	382		8.3	5	0.060	0.34	0.021	0.017	0.008	36
24-Jan-06	1.2	2.80	375		8.3	5	0.079	0.35	0.058	0.026	0.021	45
14-Feb-06	1.5	1.80	379		8.3	5	0.059	0.40	0.057	0.017	0.017	13
08-Mar-06	2.5	4.00	380		8.4	5	0.066	0.32	0.019	0.016	0.015	110
10-Oct-06		0.80	385		8.5	5	0.004	0.20	0.011	0.008	0.003	13
07-Nov-06		1.20	422		8.4	< 5	0.002	0.19	0.016	0.008	0.009	
06-Dec-06	0.1	1.60	458		8.2	< 5	0.151	0.32	0.026	0.018	0.019	34
04-Jan-07	2.4											22
30-Jan-07	2.0	0.90	450		8.3	< 5	0.179	0.24	0.007	0.006	0.006	17
27-Feb-07	2.4	2.00	427		8.4	5	0.161	0.32	0.031	0.027	0.018	43
27-Mar-07	5.1	8.00	389		8.4	5	0.034	0.41	0.052	0.023	0.023	200
09-Oct-07			355	154	8.4	5	0.002	0.29	0.016	0.011	0.013	22
06-Nov-07			400	180	8.4	< 5	0.002	0.13	0.016	0.014	0.002	1
04-Dec-07	0.5	4.78	420	180	8.45	< 5	0.093	0.24	0.017	0.015	0.015	17
08-Jan-08	0.8	3.30	430	180	8.4	< 5	0.132	0.35	0.030	0.029	0.020	22
05-Feb-08	0.0	1.79	420	180	8.3	< 5	0.163	0.52	0.020	0.02	0.017	24
27-Feb-08	2.2	14.90	390	190	8.4	5	0.171	0.45	0.051	0.052	0.020	20
25-Mar-08	5.1	4.20	390	190	8.6	< 5	<.002	0.18	0.011	0.01	0.010	<1
15-Oct-08	6.4	1.70	400	190	8.4	5	<.002	0.12	0.010	0.009	0.004	22
12-Nov-08	7.6	1.20	430	200	8.5	5	<.002	0.09	0.011	0.01	0.004	34
10-Dec-08	2.0	1.50	420	190	8.5	< 5	0.104	0.09	0.010	0.009	0.006	4
06-Jan-09	1.1	3.30	440	190	8.3	< 5	0.259	0.32	0.022	0.021	0.014	15
02-Feb-09	0.4	6.00	420	180	8.3	< 5	0.158	0.29	0.025	0.024	0.020	13
03-Mar-09	0.8	42	410	190	8.3	5	0.227	0.65	0.101	0.101	0.082	70
31-Mar-09	3.9	0.8	440	180	8.4	< 5	0.082	0.24	0.022	0.021	0.015	210
28-Oct-09	4.5	1.9	420	190	8.3	5	<0.002	0.19	0.007	0.007	0.001	54
24-Nov-09	2.5	1.2	442	42	8.4	< 5	0.005	0.13	0.006	0.006	0.001	2
16-Dec-09			105	42	8.4	10	0.2	0.38	0.003	0.004	0.001	21
19-Jan-10			478	210	8.4	5	0.243	0.39	0.021	0.021	0.02	3
16-Feb-10			502	210	8.5	5	0.067	0.12	0.093	0.18	0.02	2
16-Mar-10			478	210	8.7	15	0.011	0.13	0.003	0.006	0.001	2
26-Oct-10			462	200	8.57	5	<0.002	0.18	0.009	0.006	0.005	6
23-Nov-10			632	250	8.44	5	0.063	0.24	0.009	0.007	0.005	1100
14-Dec-10			533	210	8.3	5	0.16	0.32	0.024	0.02	0.014	33
18-Jan-11			107	42	7.33	5	0.176	0.32	0.005	0.003	0.002	3
15-Feb-11			447	180	8.17	10	0.231	0.39	0.068	0.059	0.044	1
08-Mar-11			478	200	8.19	5	0.24	0.49	0.025	0.017	0.015	21
25-Oct-11	6.40	1.20	427	180	8.32	< 5	0.01	0.37	0.01	0.009	< 0.001	1
22-Nov-11	-	-	503	211	8.31	5	0.11	0.45	0.02	0.02	0.008	18
19-Dec-11	0.70	2.15	505	213	8.26	5	0.16	0.34	0.02	0.02	0.011	51
17-Jan-12	0.90	0.72	598	252	8.28	5	0.25	0.39	0.02	0.02	0.002	NA
15-Feb-12	-	2.02	508	215	8.36	-	0.24	0.43	0.02	0.02	0.021	13
13-Mar-12	2.10	8.94	475	200	8.49	20	0.26	0.47	0.07	0.07	0.039	19
16-Oct-12	10.2	7.00	387	165	8.35	15	0.01	0.30	0.01	0.008	0.004	31
13-Nov-12	2.7	2.46	429	185	8.41	5	0.05	0.33	0.02	0.01	0.01	24
11-Dec-12	1.7	2.54	471	198	8.35	5	0.12	0.30	0.02	0.02	0.01	30
08-Jan-03	0.6	0.98	448	191	8.33	< 5	0.16	0.40	0.02	0.02	0.02	12
05-Feb-13	1.8	3.6	434	190	8.31	10	0.16	0.29	0.03	0.02	0.02	6
05-Mar-13	3.8	3.5	438	191	8.26	15	0.17	0.53	0.03	0.03	0.02	27
15-Oct-13	6.7	1.9	386	173	8.23	10	0.03	0.24	0.02	0.01	0.01	20
11-Nov-13	6.7	1.9	425	192	8.31	10	0.02	0.20	0.02	0.01	0.01	3
10-Dec-13	0.1	0.9	507	217	8.26	5	0.15	0.33	0.02	0.02	0.02	19

TABLE 5.3.1: PROBABILITY OF EXCEEDING DESIGN CRITERIA BASED ON DESIGN RETURN PERIOD AND ANTICIPATED PROJECT LIFESPAN (prepared by M. Miles and Associates Ltd.)

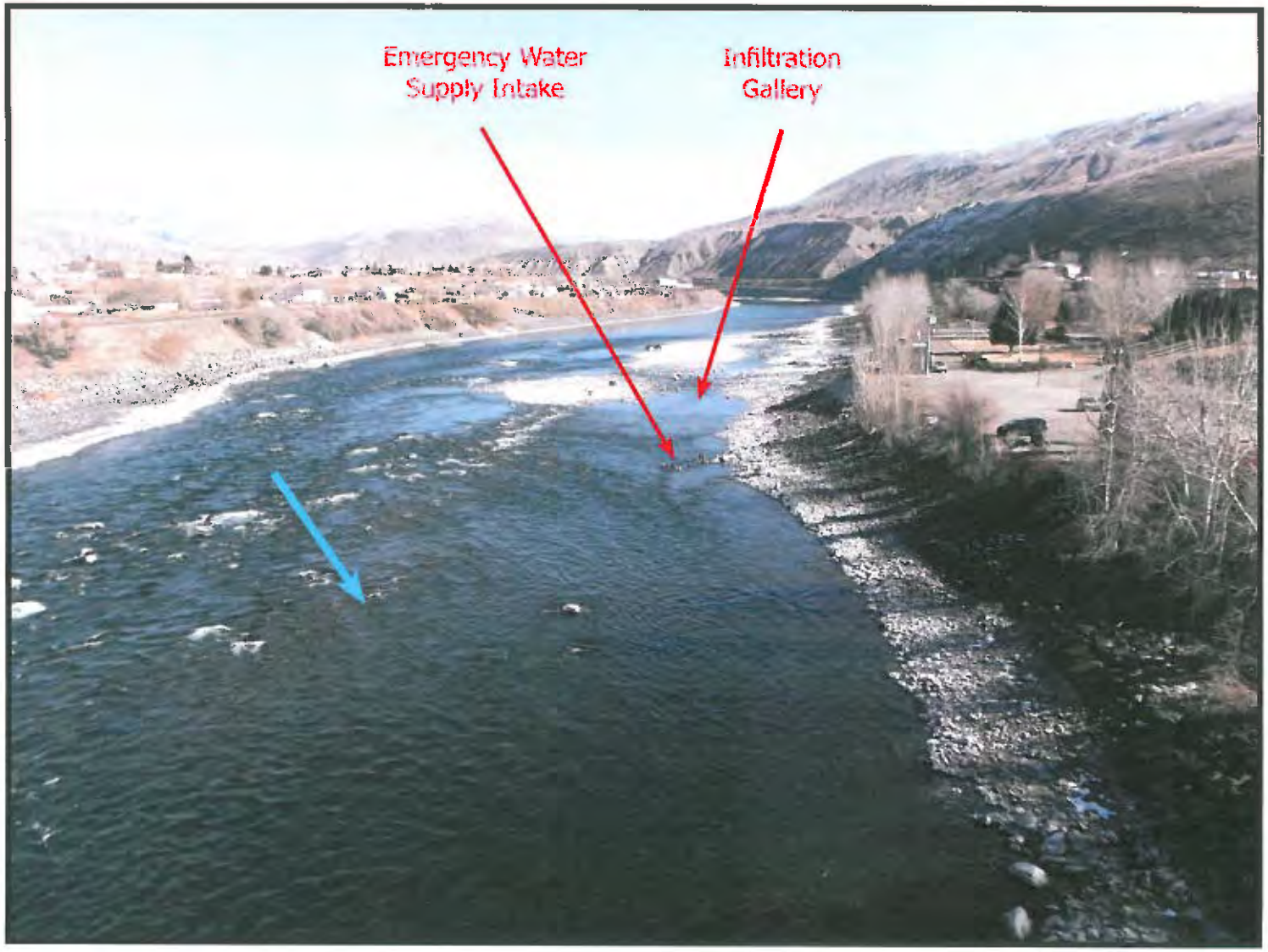
DESIGN CRITERIA	PROBABILITY OF EXCEEDING DESIGN CRITERIA (%)																
	ANTICIPATED PROJECT LIFESPAN (years)																
(average return period in years)	2	5	8	10	15	20	25	30	40	50	60	70	80	90	100	150	200
2	75	97	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
5	36	67	83	89	96	99	100	100	100	100	100	100	100	100	100	100	100
10	19	41	57	65	79	88	93	96	99	99	100	100	100	100	100	100	100
20	10	23	34	40	54	64	72	79	87	92	95	97	98	99	99	100	100
25	8	18	28	34	46	56	64	71	80	87	91	94	96	97	98	100	100
30	7	16	24	29	40	49	57	64	74	82	87	91	93	95	97	99	100
40	5	12	18	22	32	40	47	53	64	72	78	83	87	90	92	98	99
50	4	10	15	18	26	33	40	45	55	64	70	76	80	84	87	95	98
60	3	8	13	15	22	29	34	40	49	57	64	69	74	78	81	92	97
100	2	5	8	10	14	18	22	26	33	39	45	51	55	60	63	78	87
200	1.0	2	4	5	7	10	12	14	18	22	26	30	33	36	39	53	63
500	0.4	1.0	1.6	2	3	4	5	6	8	10	11	13	15	16	18	26	33
1000	0.2	0.5	0.8	1.0	1.5	2	2	3	4	5	6	7	8	9	10	14	18
1500	0.1	0.3	0.5	0.7	1.0	1.3	1.7	2	3	3	4	5	5	6	6	10	12
2000	0.1	0.2	0.4	0.5	0.7	1.0	1.2	1.5	2	2	3	3	4	4	5	7	10

PLATES

November 21, 2013

MM 13 1121 173 to 181

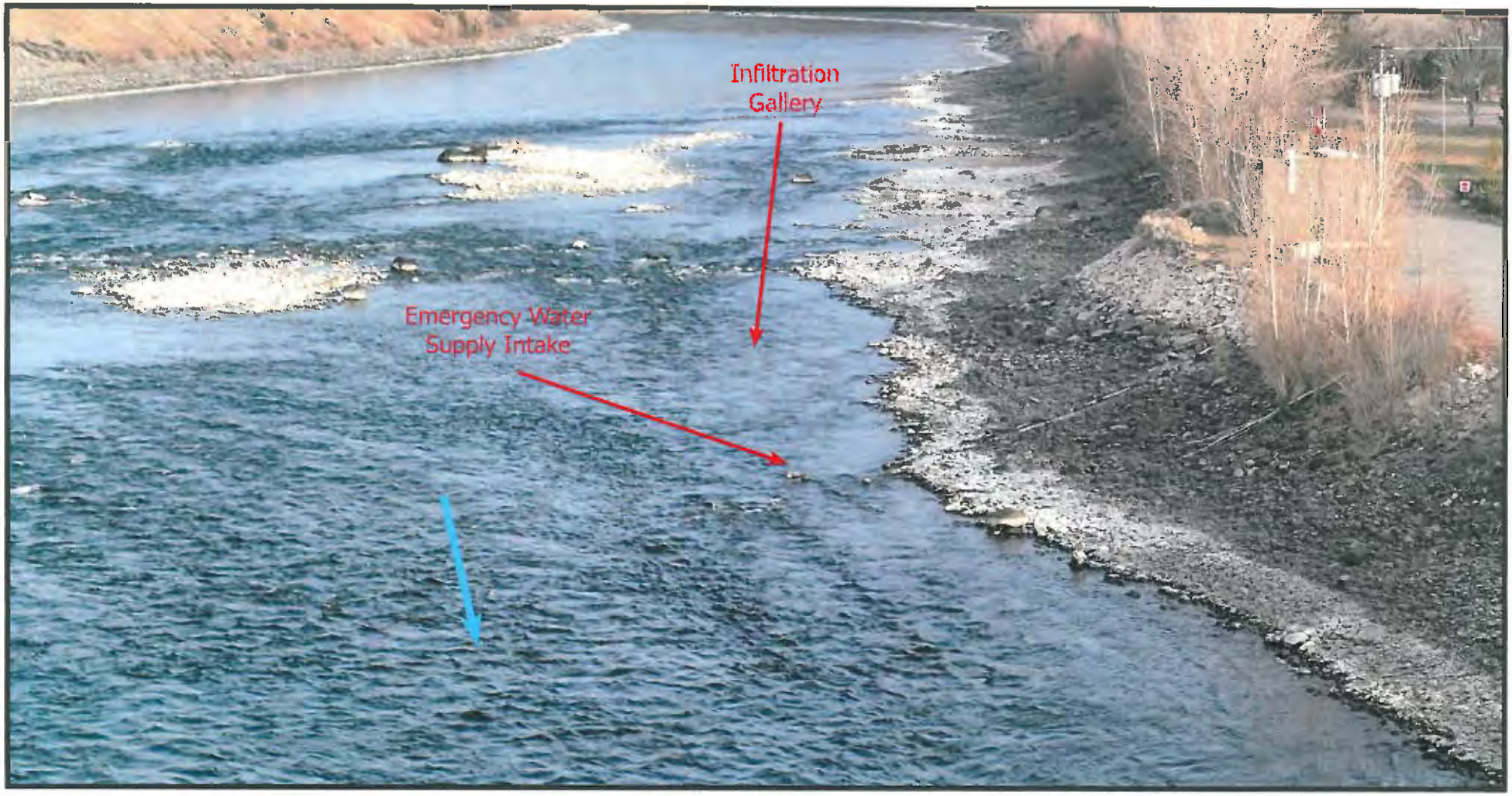
Plate 4.1.1: Looking upstream from the Highway 97C Bridge on November 21, 2013, illustrating the extent of the mid-channel bar at a discharge of 283 m^3/s (measured near Spences Bridge).



February 25, 2014

Photo by Brian Bennewith, Village of Ashcroft

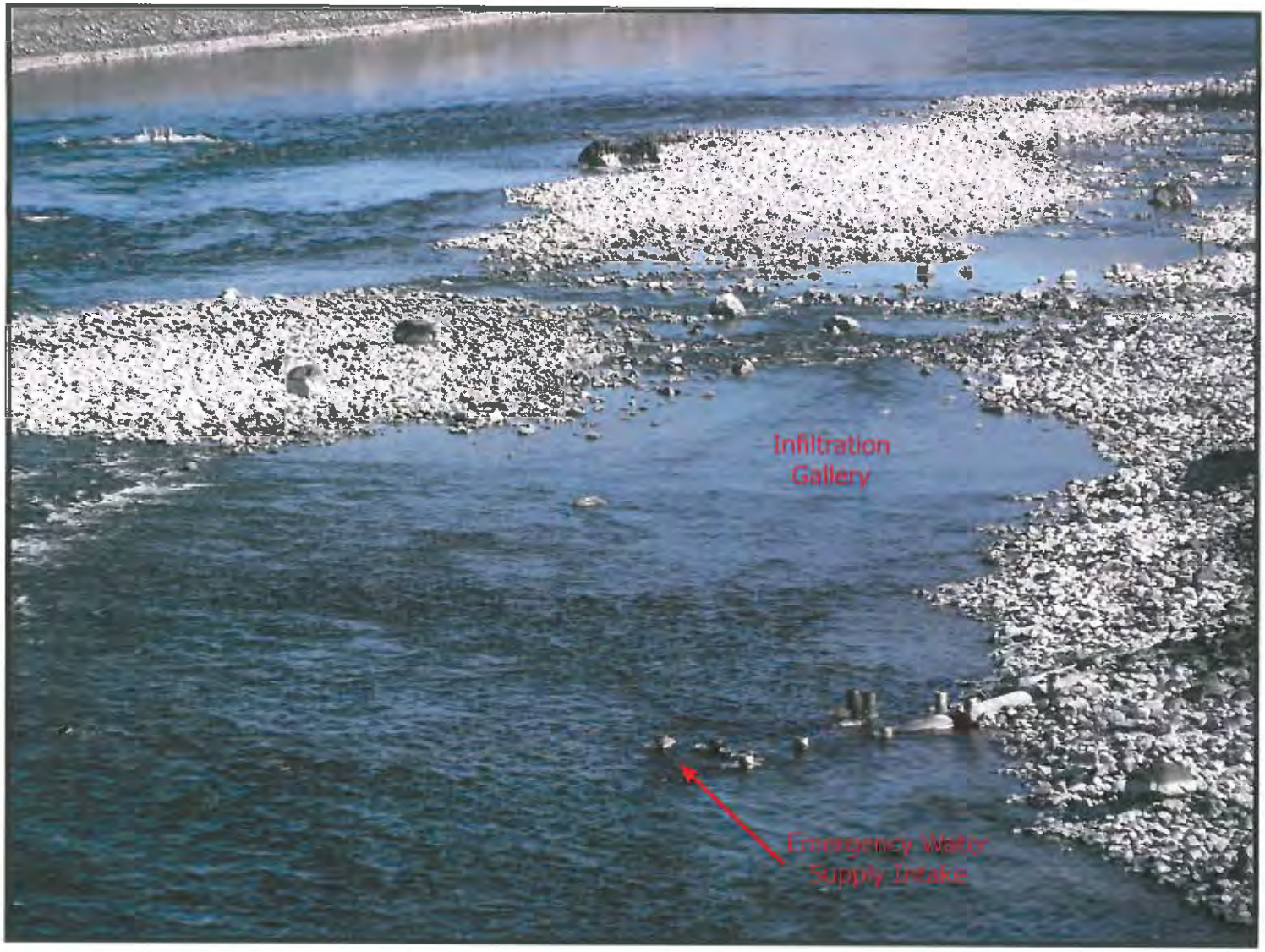
Plate 4.1.2: Looking upstream from the Highway 97C Bridge on February 25, 2014, illustrating the extent of the mid-channel bar at a discharge of 164.5 m^3/s (measured near Spences Bridge).



November 4, 2004

MM 13 1121 182 & 183

Plate 4.1.3: Looking upstream from the Highway 97C Bridge on November 21, 2013, illustrating flow conditions at the emergency intake pipe and the upstream infiltration gallery during a discharge of 283 m^3/s (measured near Spences Bridge).



February 25, 2014

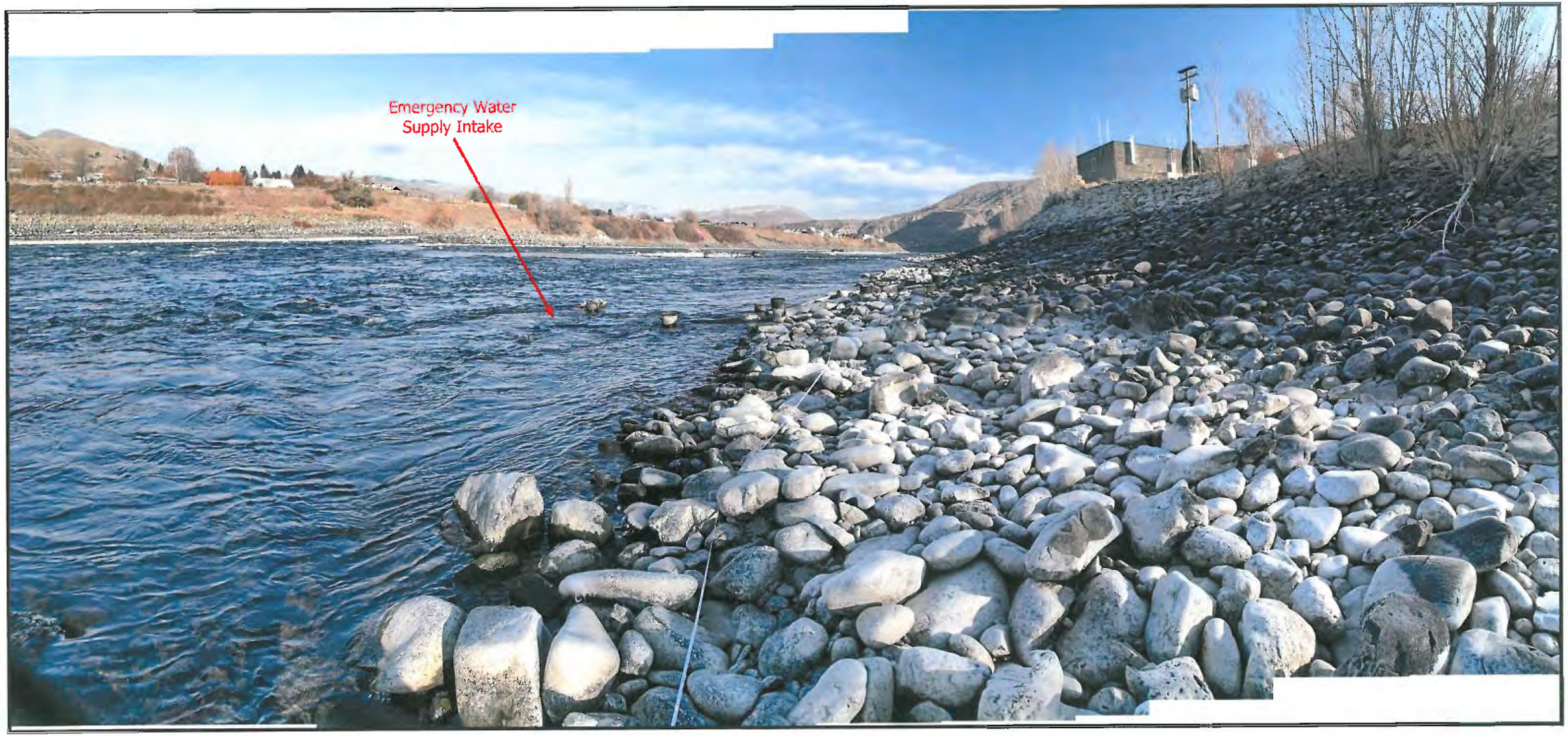
Photo by Brian Bennewith, Village of Ashcroft

Plate 4.1.4: Looking upstream from the Highway 97C Bridge on February 25, 2014, illustrating flow conditions at the emergency intake pipe and the upstream infiltration gallery during a discharge of 164.5 m^3/s (measured near Spences Bridge).

March 2003

Photos provided by Urban Systems Ltd.

Plate 4.1.5: Photograph of the infiltration gallery in 2003 illustrating the limited extent of upstream open water during a discharge of ≥183 m^3/s (measured near Spences Bridge)



November 21, 2013

MM 13 1121 088 to 096

Plate 4.1.6: Looking upstream illustrating the bed material texture on the left bank in the vicinity of the emergency water supply intake.

November 21, 2013

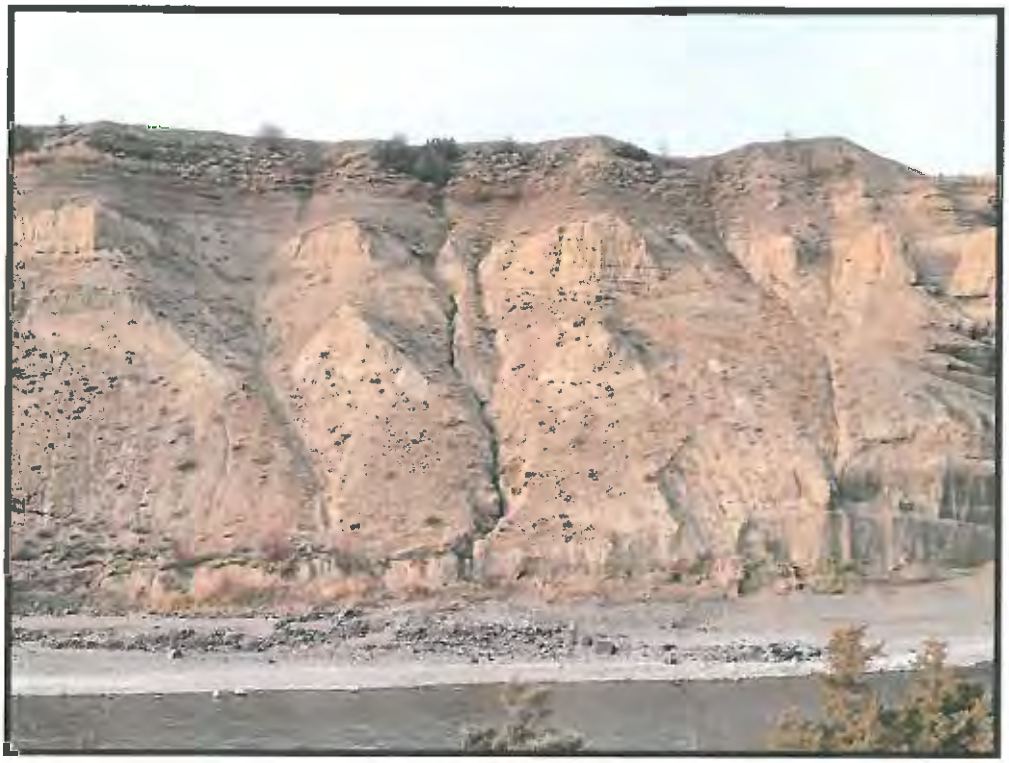
MM 13 1121 166

Plate 4.1.7: Looking upstream from the Highway 97C bridge to the bed material sampling area on the left bank in the vicinity of the emergency water supply intake.

November 21, 2013

MM 13 1121 240 to 248

Plate 4.3.1: Looking upstream to the eroding right bank terrace and active left bank point bar located 4 km upstream of the Ashcroft water supply intake.

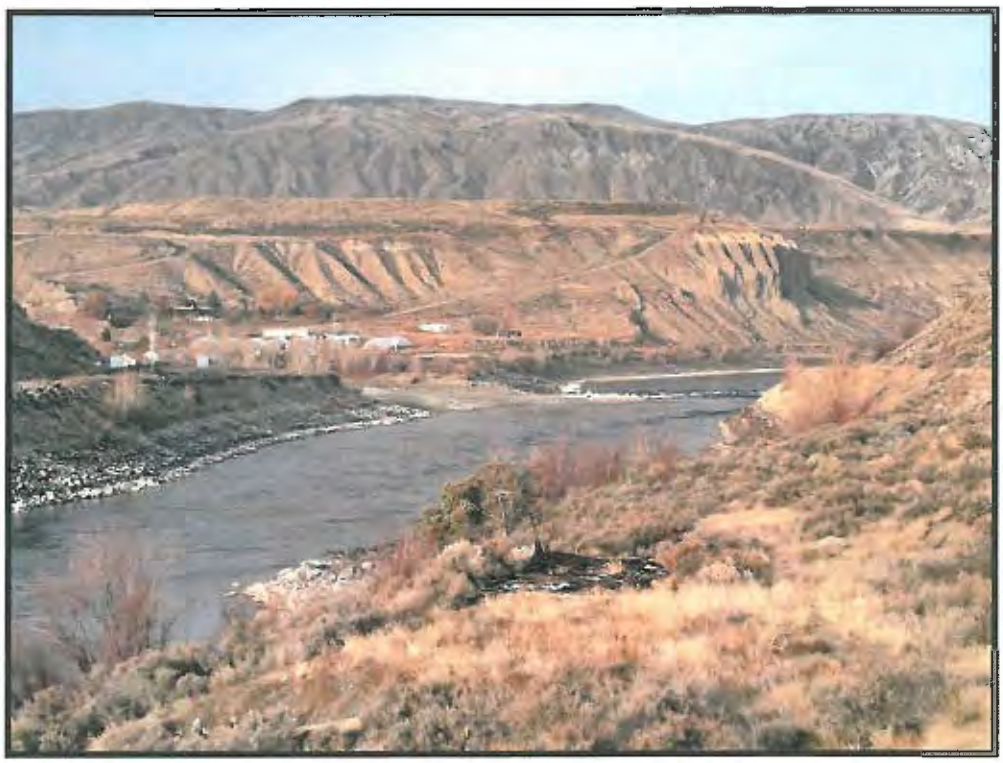


November 21, 2013 MM 13 1121 254

Plate 4.3.2: Illustration of gully formation and sediment production from eroding areas on the terraces bordering Thompson River.

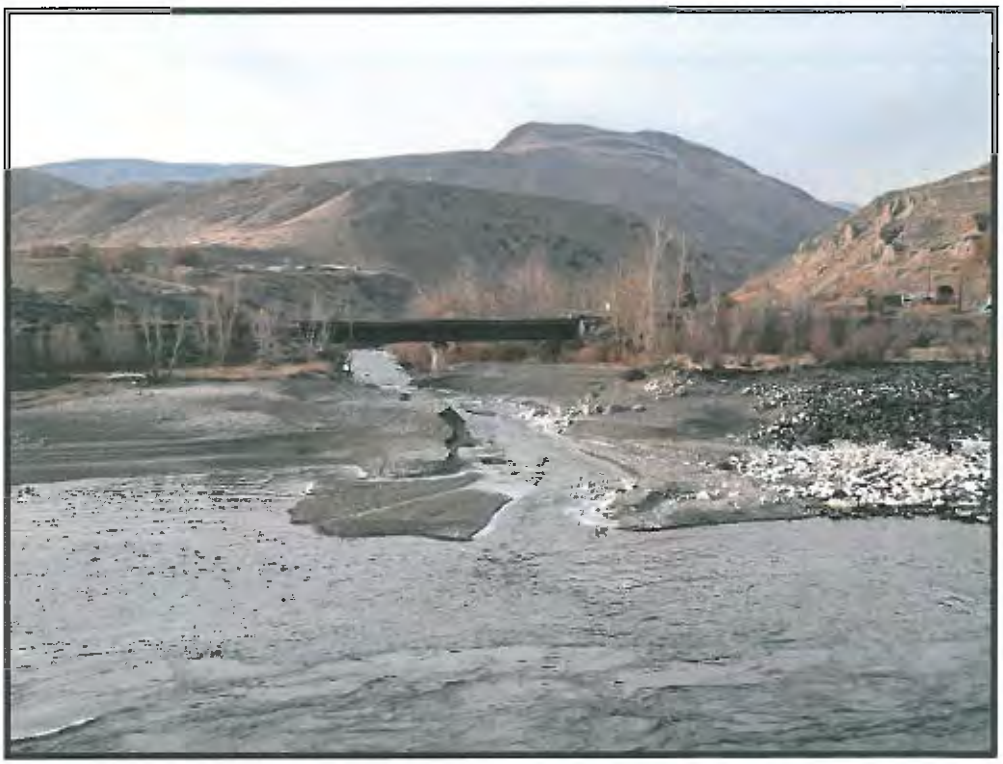
November 21, 2013 MM 13 1121 202

Plate 4.3.3: Looking upstream illustrating sediment production from one of the two small left bank streams located 2 km upstream of the Ashcroft water supply intake.



November 21, 2013 MM 13 1121 209

a) Looking upstream to the confluence



November 21, 2013 MM 13 1121 238

b) Looking west illustrating the sediment deposited along the channel banks.

Plate 4.3.4: Photographs of Bonaparte River at the confluence with Thompson River.

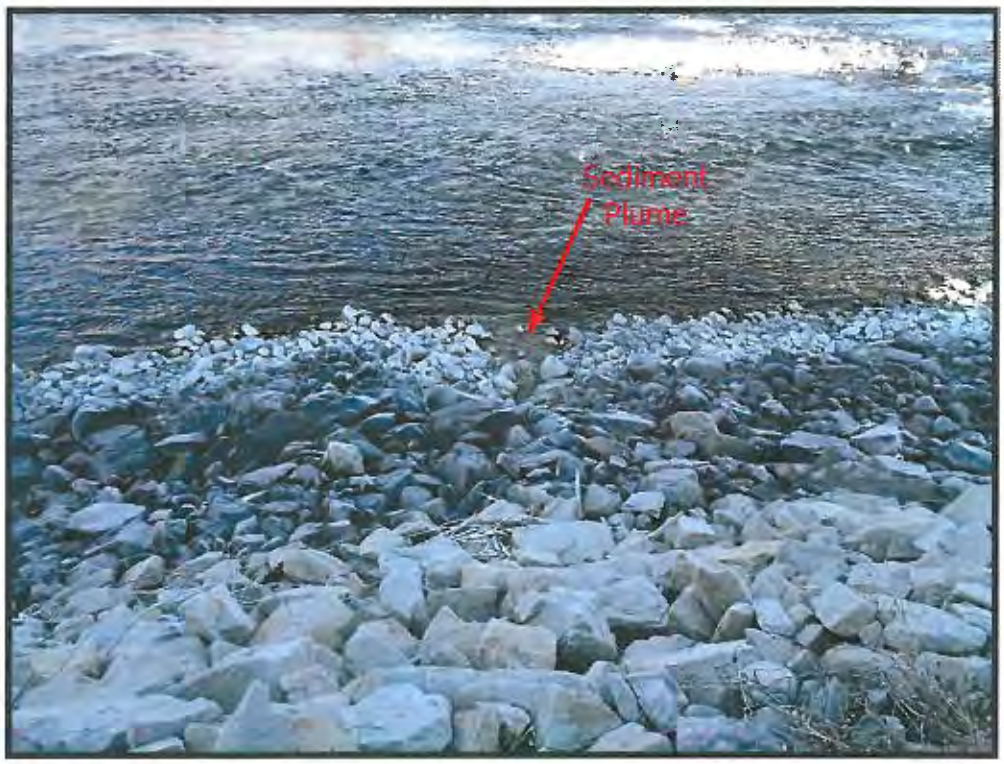
November 21, 2013 MM 13 1121 192

November 21, 2013 MM 13 1121 194

Plate 4.3.5: Looking downstream to the Highway 97C bridge illustrating the coarse textured material on the channel banks and the limited extent of areas which are susceptible to erosion.

March 2003 Photo provided by Urban Systems Ltd.

a) Discharge Thompson River near Spences Bridge ≥183 m^3/s.



November 21, 2013 MM 13 1121 061

a) Discharge Thompson River near Spences Bridge 283 m^3/s.

Plate 5.1.1: Photographs of the infiltration gallery illustrating the fine sediment plumes resulting from back-flushing in March 2003 and November 2013.

THOMPSON RIVER AT ASHCROFT:
CHANNEL STABILITY ASSESSMENT WITH RESPECT TO UPGRADING THE WATER SUPPLY INTAKE

APPENDIX 1
COMPILATION OF HISTORICAL AIR PHOTOS

PREPARED FOR:

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Chief Administrative Officer
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611 Bancroft Street
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PREPARED BY:

Mike Miles, M.Sc., P.Geo.

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645 Island Road
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Phone: 250-595-0653
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November, 2014

BC625 #15 & 16		July 31, 1948				APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge		Discharge: n/a				DATE: November 8, 2013	
REFERENCED DRAWING NO		REFERENCED DRAWING DESCRIPTION				DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

M. MILES AND ASSOCIATES LTD.
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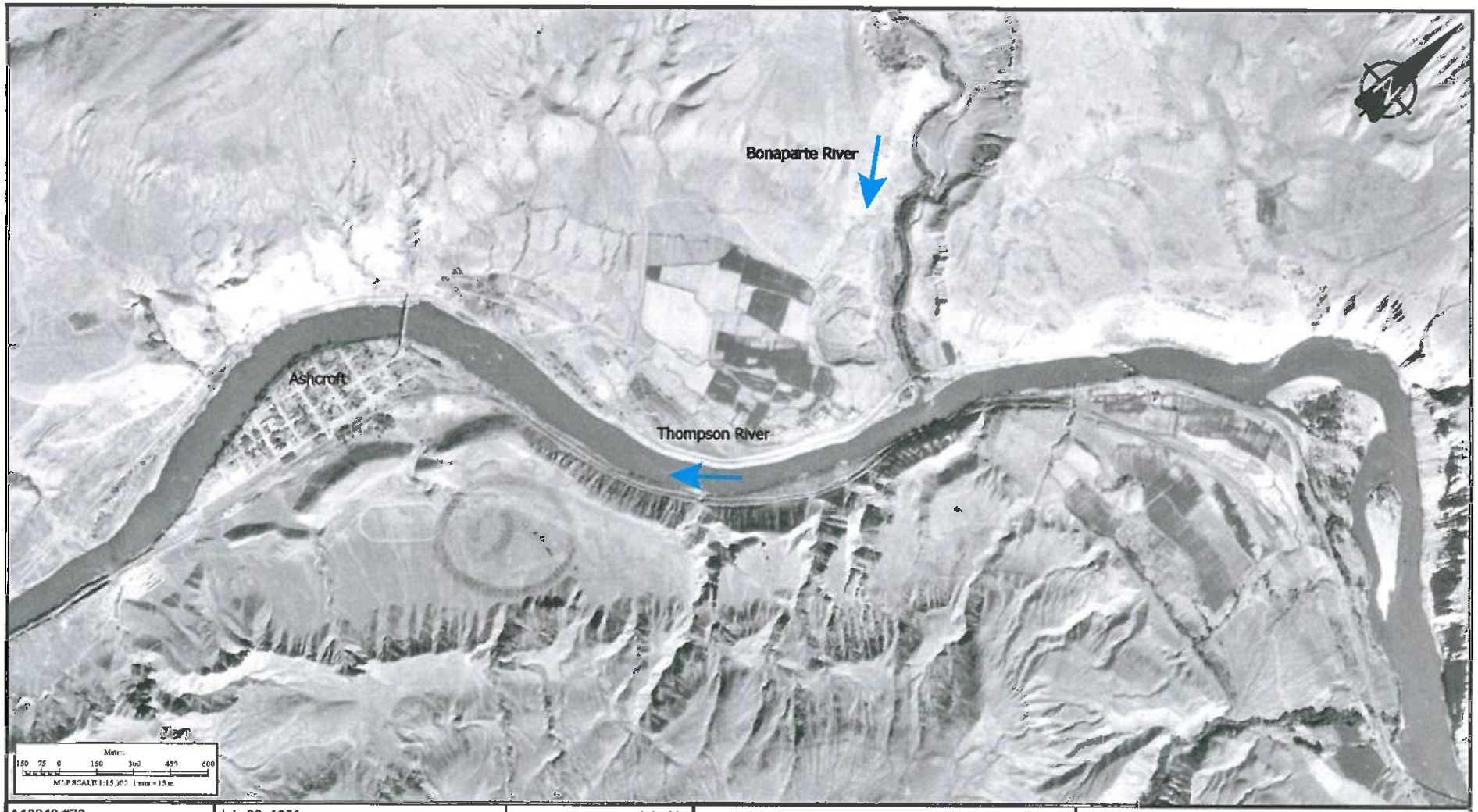
THOMPSON RIVER

1948 AIR PHOTOS

FIGURE A1 - 1

PROJECT # 401
Km #

REV: A



A13246 #78	July 30, 1951				APPROXIMATE SCALE: 1:15,000	M. MILES AND ASSOCIATES LTD. 645 ISLAND ROAD, VICTORIA, BC, V8S 2T7	THOMPSON RIVER
Thompson River near Spences Bridge	Discharge: n/a				DATE: November 8, 2013	Phone: 250-595-0653 Fax: 250-595-7367 email: mikemiles@shaw.ca	1951 AIR PHOTOS
REFERENCED DRAWING NO	REFERENCED DRAWING DESCRIPTION				DRAWN: S. Allegretto		
A	Nov. 8, 2013	Issued for discussion			DESIGNED: S. Allegretto	CLIENT: Michelle Allen, Chief Administrative Officer VILLAGE OF ASHCROFT PO Box 129, Bankcroft Street Ashcroft, BC, V0K 1A0	FIGURE A1 - 2
					CHECKED: M. Miles		PROJECT # 401
					APPROVED:		Km #
							REV: A

BC2596 #22, 23, 26 & 28		July 11, 1959				
REFERENCED DRAWING NO		REFERENCED DRAWING DESCRIPTION				
A	Nov. 8, 2013	Issued for discussion				

APPROXIMATE SCALE:	1:15,000
DATE:	November 8, 2013
DRAWN:	S. Allegretto
DESIGNED:	S. Allegretto
CHECKED:	M. Miles
APPROVED:	

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CLIENT: URBAN SYSTEMS

THOMPSON RIVER

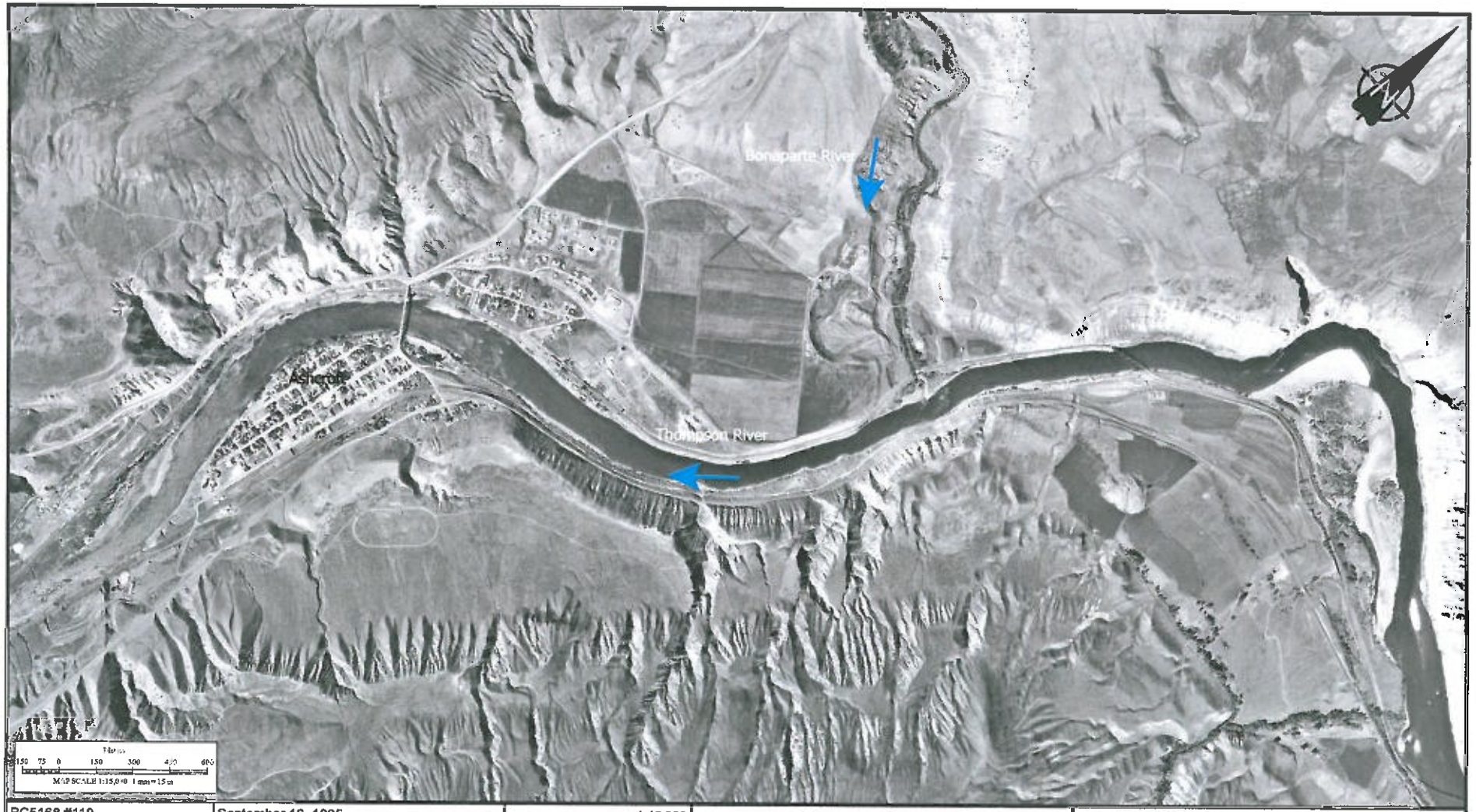
1959 AIR PHOTOS

FIGURE A1 - 3

PROJECT # 401

Km #

REV: A



BC5168 #119		September 16, 1965				APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge		Discharge: 518 m³/s				DATE: November 8, 2013	
REFERENCED DRAWING NO		REFERENCED DRAWING DESCRIPTION				DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

1965 AIR PHOTOS

FIGURE A1 - 4

PROJECT # 401
Km #
REV: A

BC7127 #81, 82 & 89		May 10, 1969				APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge		Discharge: 1,400 m³/s				DATE: November 8, 2013	
REFERENCED DRAWING NO		REFERENCED DRAWING DESCRIPTION				DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

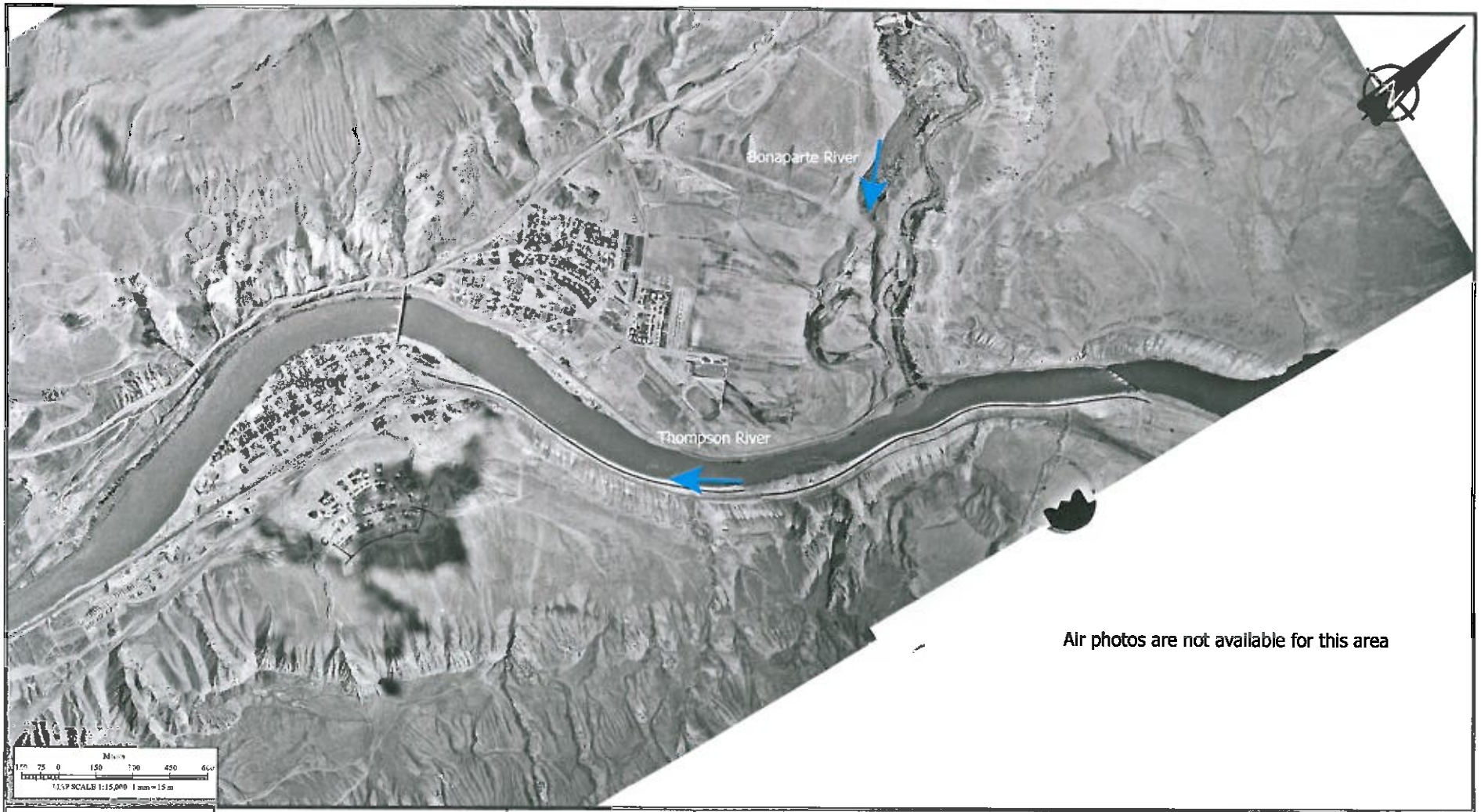
1969 AIR PHOTOS

FIGURE A1 - 5

PROJECT # 401

Km #

REV: A



BC5597 #259 & 261	June 18, 1974					APPROXIMATE SCALE: 1:15,000
						DATE: November 8, 2013
REFERENCED DRAWING NO.	REFERENCED DRAWING DESCRIPTION					DRAWN: S. Allegretto
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto
						CHECKED: M. Miles
						APPROVED:

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CLIENT: URBAN SYSTEMS

THOMPSON RIVER

1974 AIR PHOTOS

FIGURE A1 - 6

PROJECT # 401

Km #

REV: A

30BC80134 #101, 155 & 156	September 10, 1980					APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge	Discharge: 684 m³/s					DATE: November 8, 2013	
REFERENCED DRAWING NO	REFERENCED DRAWING DESCRIPTION					DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

1980 AIR PHOTOS

FIGURE A1 - 7

PROJECT # 401
Km #

REV: **A**

30BC86038 #110-112, 131, 132 & 135	June 28, 1986					APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge	Discharge: 2,130 m³/s					DATE: November 8, 2013	
REFERENCED DRAWING NO.	REFERENCED DRAWING DESCRIPTION					DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

1986 AIR PHOTOS

FIGURE A1 - 8

PROJECT # 401
Km #
REV: A

30BCC92014 #144, 146, 156 & 157	April 11, 1992					APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge	Discharge: 632 m³/s					DATE: November 8, 2013	
REFERENCED DRAWING NO.	REFERENCED DRAWING DESCRIPTION					DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

1992 AIR PHOTOS

FIGURE A1 - 9

PROJECT # 401

Km #

REV: A

30BCC00006 #169 & 170 30BCC00007 #70 & 71		July 19, 2000				APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge		Discharge: 2,000 m³/s				DATE: November 8, 2013	
REFERENCED DRAWING NO		REFERENCED DRAWING DESCRIPTION				DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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CLIENT:
Michelle Allen, Chief Administrative Officer
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Ashcroft, BC, V0K 1A0

THOMPSON RIVER

2000 AIR PHOTOS

FIGURE A1 - 10

PROJECT # 401

Km #

REV: A

15BCC04029 #56 & 75	July 17, 2004					APPROXIMATE SCALE: 1:15,000	
Thompson River near Spences Bridge	Discharge:				1,110 m³/s	DATE: November 8, 2013	
REFERENCED DRAWING NO	REFERENCED DRAWING DESCRIPTION					DRAWN: S. Allegretto	
A	Nov. 8, 2013	Issued for discussion				DESIGNED: S. Allegretto	
						CHECKED: M. Miles	
						APPROVED:	

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THOMPSON RIVER

2004 AIR PHOTO

FIGURE A1 - 11

PROJECT # 401
Km #

REV: **A**

12BCD11301 #530, 531 & 533 12BCD11304 #262 & 264	August 3, 2011				APPROXIMATE SCALE:	1:15,000	
Thompson River near Spences Bridge	Discharge:	1,680 m³/s			DATE:	November 8, 2013	
REFERENCED DRAWING NO	REFERENCED DRAWING DESCRIPTION				DRAWN:	S. Allegretto	
A	Nov. 8, 2013	Issued for discussion			DESIGNED:	S. Allegretto	
					CHECKED:	M. Miles	
					APPROVED:		

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THOMPSON RIVER

2011 AIR PHOTOS

FIGURE A1 - 12

PROJECT #	401	REV:
Km #		A

General Condition Assessment of Village of Ashcroft Reservoirs Ashcroft, B.C.

Prepared for:

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200-286 St. Paul Street
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Job No. K4361

March 07, 2014

General Condition Assessment of
Village of Ashcroft Reservoirs
Ashcroft, B.C.

Table of Contents

1.0 Introduction & Scope

CWMM Consulting Engineers Ltd. has been retained to provide a general condition assessment of three existing concrete reservoirs located in Ashcroft, B.C..The purpose of this assessment is to determine the general condition of these structures and to provide recommendations for further attention or possible remedial works. A limited number of structural drawings for these reservoirs have been submitted to *CWMM* for review, and based on these drawings, an analytical review of the concrete work in these reservoirs has been undertaken. The field review of these reservoirs was primarily limited to a visual examination of those components which could be observed directly, with some hammer soundings carried out on these components as well.

2.0 Field Review & Site Description

Field reviews on these reservoirs were carried out on December 12th and 19th, 2013 by Brendan Murtagh, a P. Eng. with *CWMM*. These field reviews consisted of a visual examination of the interior portions of each structure. All three reservoirs were drained during the time of the field review.

The reservoirs that were field reviewed for this assessment were the Zone 1 Reservoir (approximately 1,620,000L capacity), Zone 2 Reservoir (approximately 205,000L capacity) and Zone 3 Pump Chamber (approximately 480,000L capacity). All three reservoirs collect and store water from the Thompson River, which flows through the middle of Ashcroft in the west direction, and then turns south and flows along the southern region of the village. The Zone 1 and Zone 2 Reservoirs are located in the hills just above the southern region of the village and are approximately 283'-0" and 463'-0" above the Thompson River respectively. The Zone 3 Pump Chamber is located in the northern region of the village and is approximately 187'-0" above the Thompson River. All three reservoirs are located below grade.

3.0 Structural Description

Refer to Appendix A for existing structural drawings for the noted reservoirs.

3.1.1 Zone 1 Reservoir Structure

The Zone 1 Reservoir appears to have been constructed in 1981 with an approximate storage capacity of 1,620,000L that is shared between 2 cells. The inside of these cells is approximately 9.41m wide x 19.52m long x 4.35m high with the cell's short plan dimension running in the north-south direction. The concrete structure consists of 350mm thick concrete exterior walls, a single 350mm thick concrete interior wall, 150mm thick slab-on-grade between these walls in each cell, and 610mm deep precast concrete double 'T' roof members complete with a 75mm concrete topping. The concrete walls bear on strip footings. The precast roof members span in the short plan dimension of each cell and bear 125mm onto the exterior walls and the interior wall. This structure is

located below grade in a localized depression that is located at the edge of a plateau. An approximate 600mm deep layer of earth is specified immediately above the roof concrete topping. The deeper layer of earth that rises above this localized depression is located just beyond the plan area of this reservoir.

3.1.2 Zone 1 Reservoir Construction Joints

The exterior walls contain vertical construction joints which are located approximately at the quarter and three quarter points along each cell's longer plan dimension and approximately at the center of each cell's shorter plan dimension. The interior wall contains vertical construction joints which appear to align with the vertical construction joints in the exterior walls. The slab-on-grade contains construction joints located approximately at the center of each cell's longer plan dimension. Construction joints are also located between the walls and the slab-on-grade, and also at the tops of these walls. The construction joints in the north cell appear to be covered with cementitious sealants whereas the construction joints in the south cell appear to be covered with mastic and cementitious sealants. Refer to Appendix A for a typical joint detail. In addition to the construction joints, small localized cavities on the inside faces of the walls that were left over from the formwork ties appears to have been covered over with a cementitious material.

3.1.3 Zone 1 Reservoir Access

Access into each cell in this reservoir is achieved through an access hatch that corresponds to each cell, and down a fixed ladder that is attached to the interior wall. The access hatch consists of a reinforced aluminum hatch plate member that is mounted to a 900mm tall concrete upstand with inside face to inside face dimensions of 1065mm x 1065mm on all sides. The fixed ladders into each cell are aluminum ladders that are attached to the interior wall at the top and are attached to the slab-on-grade at the bottom. These attachments consist of galvanized metal angles and stainless steel anchor bolts. No information was provided regarding the aluminum ladder's rung and side rail sizes or their grades. Refer to Appendix A for the access hatch and ladder details.

3.1.4 Zone 1 Reservoir Pipework

The pipes located inside each cell appears to consist of metal pipes, which include ductile iron and steel, and pvc pipes. Some of the metal pipes appear to have a coating around them while the pvc pipes are exposed. These pipes appear to be restrained with either galvanized or exposed metal hardware. An overflow pipe appears to limit the water level from exceeding 4.25m above the slab-on-grade. Refer to Appendix A for pipe work information.

3.2.1 Zone 2 Reservoir Structure

The Zone 2 Reservoir appears to have been constructed in 1970 with an approximate storage capacity of 205,000L. The inside of this reservoir is approximately 6m wide x 12m long x 3m high with the reservoirs long plan dimension running in the north-south direction. The concrete structure consists of a concrete suspended roof slab that tapers

down from 275mm along the center ridge to 250mm along the east and west eaves, 250mm thick exterior walls and a 250mm thick raft slab that supports these exterior walls. This structure is located on the face of a hill and is partially located below the natural grade with the remainder of the structure covered with a built up earth grade that extends approximately to the roof slab. The roof slab is fully exposed.

3.2.2 Zone 2 Reservoir Construction Joints

The only construction joints in this structure are located between the exterior walls and roof slab and exterior walls and raft slab. No construction joints are installed within these members. A thin cementitious layer appears to cover the construction joint between the exterior walls and raft slab. A water stop is specified between these members as per Appendix A.

3.2.3 Zone 2 Reservoir Access

Access into this reservoir is achieved through an access hatch and down a fixed ladder which is located in the south-east corner of this reservoir. The access hatch consists of a 9.5mm thick plate mounted to a 100mm tall concrete upstand with inside face to inside face dimensions of 750mm x 750mm all around. The fixed ladder is a galvanized metal ladder that consists of 9.5mm thick x 50mm deep side rails that are attached to the exterior walls with supports spaced at approximately 1200 o/c and also attached to the raft slab. The rungs are 16mm in diameter and are spaced at 300mm on center. The center of the side rails appears to be 300mm beyond the face of the exterior wall. Refer to Appendix A for the access hatch and ladder details.

3.2.4 Zone 2 Reservoir Pipework

The pipes located inside this reservoir appear to be ductile iron pipes and are restrained with metal restraints. An overflow pipe appears to limit the water level from exceeding 2.7m above the raft slab. Refer to Appendix A for pipe work information.

3.3.1 Zone 3 Pump Chamber Structure

The Zone 3 Pump Chamber appears to have been constructed in 1962 with an approximate storage capacity of 480,000L that is shared between 2 cells. The inside of each of these cells is approximately 4.8m wide x 14.4m long x 3.6m high with the cell's short plan dimension running in the north-south direction. The concrete structure consists of a 250mm thick concrete suspended roof slab, 250mm thick concrete exterior walls, a 250mm thick concrete interior wall, and a 300mm thick concrete raft slab which supports the exterior and interior walls. This structure is located below grade with a specified layer of earth fill approximately 650mm thick over the concrete roof slab. A wooden pump chamber structure with masonry foundation walls that extend down through this earth fill is located above the roof slab over the east end of the north cell. The masonry foundation walls bear on the north exterior side wall, east exterior end wall, the interior wall, and the roof slab.

3.3.2 Zone 3 Pump Chamber Construction Joints

The only construction joints located in this structure appears to be located between the exterior walls and roof/raft slabs. No construction joints are installed within these members. There is no sealant material between the exterior walls and the raft slab. A water stop is specified between these members as per Appendix A.

3.3.3 Zone 3 Pump Chamber Access

Access into each cell in this reservoir is achieved through an access hatch and down a fixed ladder which is located in the north-east corner of each cell. The access hatch above the north cell consists of a metal grate, complete with a cover plate, which is mounted to a 275mm tall concrete upstand with inside face to inside face dimensions of 700mm x 700mm all around. The access hatch above the south cell consists of a cover box plate that simply covers a 750mm tall concrete upstand, which extends through the earth fill, with inside face to inside face dimensions of 650mm x 650mm all around. The fixed ladders into each cell are metal ladders that consist of 6.4mm thick x 38mm deep side rails that are attached to the concrete walls with supports spaced at approximately 1600mm o/c, and 19mm diameter rungs spaced at 300 o/c. The center of the side rails appears to be 240mm beyond the face of the concrete walls. Refer to Appendix A for the access hatch and ladder details.

3.3.4 Zone 3 Pump Chamber Pipework

The original pipes appear to be cast iron complete with a bitumastic coating around them as indicated on the original drawings. The material for the newer pipes cannot be confirmed and these pipes appear to have a protective coating around them. An overflow pipe appears to limit the water level from exceeding 3.3m above the raft slab. Refer to Appendix A for pipe work information.

4.0 General Condition Assessment

Refer to Appendix B for Site Photos.

4.1.1 Zone 1 Reservoir Structure

The interior surfaces of the concrete walls, excluding construction joints, generally appeared to be in good structural condition as no major cracks were noted. Furthermore, hammer soundings were also carried out on these walls and no signs of delamination or spalling were noted. There were however several observations noted with these concrete walls:

1. The north exterior wall visually appeared to be slightly bulging inwards.
2. Some areas of wall appeared to exhibit signs of localized honeycombing as shown in Photo 1, possibly as a result of placement issues with the concrete during the construction phase.
3. During the field review of each individual cell, the adjacent cell was approximately full of water. There was no visible leakage observed along the interior concrete

wall, except for a very small localized area on the south face at the east end where the concrete had honeycombed. A thin layer of water could be observed on the exposed coarse aggregate as shown in Photo 1. It should be further noted that the mastic and cementitious patches along this wall did not appear to exhibit signs of visible leakage.

4. A redish-brown residue was observed on the surface of the various wall faces, with this residue more prevalent in the south chamber. This residue could be easily scrapped off and the underlying concrete surface exposed as shown in Photo 2 and 3. It was also observed that this residue consistently terminated a short distance below the top of these walls as shown in Photo 4, such that it may be associated with the water level.
5. The surface of the various wall faces appeared to exhibit a weathered appearance in both cells as shown in Photo 3.
6. A small number of corrosion stains were noted on these walls in both cells.

The condition of the top surface of each slab-on-grade in each cell was difficult to ascertain as there appeared to be a solid coating over this slab followed by a thin layer of residue as shown in Photo 6. Hammer soundings were carried out on this slab and no signs of delamination or spalling were noted. A substantial crack in the slab–on-grade in the north-west corner of the south cell was observed and had been patched with a mastic sealant in the past.

The underside of the precast Double-T concrete roof members generally appeared to be in good structural condition as no major cracks or excessive deflections were noted, however, the metal embed plates in the ends of these roof members and metal embed plates at the top of the concrete walls were corroded in both cells as shown in Photo 5.

4.1.2 Zone 1 Reservoir Construction Joints

The cementitious sealants over the construction joints in the north cell walls generally appeared to be in good condition, while the cementitious sealants over the construction joints between these walls and the slab-on-grade appeared to be cracked, broken, or too thin in some locations as shown in Photo 7. The cementitious sealant over the construction joint in the north cell slab-on-grade generally appeared to be in good condition, however, minor cracks were observed as shown in Photo 8. Hammer soundings were also carried out on these sealants and they sounded solid.

The condition of the mastic sealants over the construction joints in the south cell walls appeared to vary in condition. Some of these sealants appeared to be in good condition while others appeared to exhibit signs of damage or cracks as shown in Photo 9. The mastic sealants over the construction joints between south cell walls and the slab-on-grade and in the south cell slab-on-grade itself appeared to be damaged or missing altogether along numerous sections as shown in Photos 10 & 11.

The cementitious sealants over the small localized cavities in the concrete walls, which were left over from the formwork ties, generally appeared to be in good condition. One of these sealant members appeared to be dislodging itself from the north cell exterior wall.

4.1.3 Zone 1 Reservoir Access

The reinforced aluminum hatch plate members over each cell appeared to be in good condition. The aluminum ladder in each cell appeared to exhibit corrosion build-up over its surface. This corrosion build-up was removed along a small section of the side rail in the south cell and there appeared to be minor surface pitting occurring beneath. The aluminum ladder in the south cell contained more of this corrosion build-up over its surface compared to the aluminum ladder in the north cell as shown in Photo 12 & 13.

The access hatches and the aluminum ladders appeared to meet the requirements of ANSI A14.3-2008 *American National Standard for Fixed Ladders* with respect to minimum dimensions and clearances. The strength of the aluminum ladders was not checked as no information was provided regarding the size or grade of the rungs or side rails.

4.1.4 Zone 1 Reservoir Pipework

Some of the metal pipes appeared to exhibit localized corrosion build-up over their surfaces, as shown on the pipe in Photo 13, while other metal pipes appeared to exhibit uniform corrosion over their surfaces. A metal pipe in the south cell was observed to have corroded completely through its section as water was observed to be spraying out. The PVC pipes in each cell showed signs of discoloration but did not appear to exhibit any signs of surface deterioration. The metal pipe restraints in each cell and below the top of the over-flow pipes appeared to be severely corroding.

4.2.1 Zone 2 Reservoir Structure

The interior surfaces of the concrete walls appeared to have been patched in numerous locations in the past. The condition of these walls and the apparent patch work generally appeared to be in good condition as no major cracks were noted. Furthermore, hammer soundings were carried out and no signs of delamination or spalling were observed. There were however several observations noted with these concrete walls:

1. A light yellowish color strip of residue, possibly efflorescence, was observed along the walls approximately at the same elevation as the top of water elevation as shown Photo 14. Efflorescence indicates leaching of salts from the concrete but typically not indicative of a structural concern.
2. The original concrete surface appeared to exhibit a slightly weathered appearance as shown in Photo 15.
3. Nails were observed to be projecting beyond the concrete surface along the top of the walls and rebar dowels, which were cut flush with the concrete surface, were observed along the bottom of the walls as shown in Photo 16. Small localized rust stains were observed around some of these objects.

The general condition of the top of the raft slab was difficult to ascertain as there appeared to be a solid coating over this slab, and during the time of the field review, a layer of frozen water over this solid coating as shown in Photo 17. Consequently, hammer soundings were not carried out on this slab.

The underside of the concrete suspended roof slab appeared to be in good structural condition as no major cracks or excessive deflections were observed. There were however areas on the underside of this roof slab where localized honeycombing was observed, and in one of these localized honeycombed sections, transverse rebar was exposed as shown in Photo 18. Nails could also be observed projecting below the underside of the roof slab. The top of the roof slab was only observed in a couple of areas as the top of this slab was covered with snow. No major cracks or other major concerns were noted in these observed areas.

4.2.2 Zone 2 Reservoir Construction Joints

The thin cementitious layer covering the construction joint between the exterior walls and raft slab appeared to be deteriorating and debonding from the concrete wall along its top edge as shown in Photo 19.

4.2.3 Zone 2 Reservoir Access

The access hatch appeared to be in satisfactory condition. The fixed galvanized metal ladder components, which include the side rails, rungs and attachments to the concrete wall, have corroded below the top of the overflow pipe as shown in Photo 20.

The fixed ladder components and the access hatch did not to meet the requirements of ANSI A14.3-2008 *American National Standard for Fixed Ladders* in terms of minimum member sizes and clearances. The side rails should be a minimum of 9.5mm thick x 64mm wide and the rungs should be a minimum of 19mm in diameter. The minimum clearances for the access hatch and ladder termination should conform to Appendix C.

4.2.4 Zone 2 Reservoir Pipework

The pipework and their corresponding metal restraints inside this reservoir were corroded as shown in Photo 21.

4.3.1 Zone 3 Pump Chamber Structure

The interior surfaces of the concrete walls generally appeared to be in good structural condition as no major cracks were noted. Furthermore, hammer soundings were also carried out on these walls and no signs of delamination or spalling were noted. There were however several observations noted with these concrete walls:

1. A uniform redish-brown residue was observed on the surface of the concrete walls in both cells and consistently terminated a short distance below the top of these walls as shown in Photo 22, such that it may be associated with the water level. This film could be easily scraped off and the underlying concrete surface exposed as shown in Photo 23.
2. The surface of the concrete walls appeared to exhibit a weathered appearance in both cells as shown in Photo 24.
3. Wood embedded in the concrete could be observed on the surface of these walls along the bottom and at the corners in both cells as shown in Photos 25 & 26.

The condition of the top surface of the raft slab was difficult to ascertain in both cells as there appeared to be a solid coating over the entire slab followed by a thin layer of residue in some areas as shown in Photos 27 & 28. Divots were observed in the top surface of the raft slab in the south cell, possibly a result from an impact during the construction phase.

The underside of the concrete suspended roof slab over both cells generally appeared to be in good structural condition as no major cracks or excessive deflections were observed. Small localized rust stains were observed throughout this surface and a small section of rebar appeared to be exposed in the north cell.

4.3.2 Zone 3 Pump Chamber Construction Joints

No water-proofing sealants were installed over the construction joints in this structure, therefore, there were no observations noted regarding these sealants.

4.3.3 Zone 3 Pump Chamber Access

The access hatches on both cells appeared to be in good condition while the fixed ladders in both cells were corroded below the top of the overflow pipe as shown in Photo 29.

The side rails of both fixed ladders and both access hatches did not meet the requirements of ANSI A14.3-2008 *American National Standard for Fixed Ladders* in terms of minimum member size and clearances. The side rails should be a minimum of 9.5mm thick x 64mm wide and the minimum clearances for the access hatch and ladder termination should conform to per Appendix C.

4.3.4 Zone 3 Pump Chamber Pipework

The original cast iron pipes appeared to exhibit uniform corrosion over their surfaces as shown in Photo 30, while the newer pipes appeared to exhibit localized corrosion build-up over their surfaces as shown in Photo 31.

5.0 Limited Analytical Review

Based on the structural drawings submitted to *CWMM*, an analytical review could only be undertaken on the Zone 1 Reservoir and the Zone 2 Reservoir. This analytical review was limited to basic strength checks of the existing structural components noted in this section and also to determine if these structural components met minimum reinforcing requirements per Code. A full analytical review of these reservoirs was not undertaken as this is beyond the scope of this general condition assessment. This limited analytical review was conducted in accordance with the provisions set forth in the *2012 BC Building Code* (BCBC), and *Code Requirements for Environmental Engineering Concrete Structures* (ACI 350-06) and commentary (ACI 350R-06). The following design parameters were used in this analytical review:

Gravity Loads:

Snow & Rain Loads

S_s (ground snow)	= 1.7 kPa
S_r (rain)	= 0.1 kPa
I_s	=1.25 (ULS)
I_s	= 0.9 (SLS)

A superimposed dead load of 11.4 kPa was applied over the roof of the Zone 1 Reservoir as the existing drawings specified a 600mm +/- earth cover.

Lateral Loads:

Hydrostatic loading/m	= 9.8 kN/m^3 x h		
Lateral Earth Pressure loading	= (K_o * γ_{soil} x h) +(k_o * surcharge)		
	K_o	= 0.42	(assumed)
	γ_{soil}	= 19.0 KN/m^3	(assumed)
	h	= height (m)	

The surcharge load for the Zone 1 Reservoir included a snow load component and an earth cover load component. The earth cover component for this reservoir was taken as 24.7kPa, as there appeared to be approximately 1.3m of earth above the top support for the exterior side walls immediately beyond the plan area of this reservoir. The surcharge load for the Zone 2 Reservoir only included a snow load component and not an earth cover load component as the grade around this reservoir never extended beyond the top of the concrete roof slab.

The concrete compressive strength was assumed to be 20.7 MPa (3000 psi) and the concrete reinforcement was assumed to be intermediate grade with an f_y equal to 276 MPa for both reservoirs.

5.1 Zone 1 Reservoir

The existing concrete walls appeared to have been designed using two-way bending analysis with the concrete reinforcement designed to more economically resist the resulting forces from this type of analysis compared to one-way bending analysis. Due to the variability in the amount of reinforcement in these walls, only a meter wide strip down the center of the exterior side walls and interior wall was analyzed, as these strips displayed the highest flexural forces. These walls were found to have sufficient flexural reinforcement in resisting hydrostatic loads but significantly insufficient flexural reinforcement in resisting the lateral earth pressure loads based on the assumptions indicated. These walls were found to have moderately insufficient horizontal shrinkage and temperature reinforcement.

The existing slab-on-grade was found to have significantly insufficient temperature and shrinkage reinforcement in each direction. Also, the spacing of the wires in the 152x152

MW18.7/MW18.7 welded wire mesh in this slab was found to exceed the maximum wire spacing of 102mm.

The existing precast roof Double-T panel members could not be analyzed as structural information was not provided for these members.

5.2 Zone 2 Reservoir

The existing concrete suspended roof slab appeared to have been designed as a one-way slab member. This roof slab was found to have sufficient flexural reinforcement and concrete shear strength against the gravity loads noted above, but significant insufficient temperature and shrinkage reinforcement in the longitudinal direction. Since it is feasible for a vehicle to drive onto this roof slab from the west side of this reservoir, a minimum vehicular live load corresponding to vehicles weighing a maximum of 4000 kg was further applied over this roof slab. This vehicular live load was taken as 2.4 kPa or 18 kN, which ever produced the most adverse effects. This roof slab was found to have sufficient flexural reinforcement, when analyzed as a two-way slab, to support the vehicular load noted above but insufficient flexural reinforcement to support vehicles weighing more than 4000 kg.

The existing concrete walls appear to have been designed as a one-way member spanning between the raft and the suspended roof slab. These walls were found to have sufficient flexural reinforcement against the hydrostatic loading but deficient flexural reinforcement against lateral earth pressure loading since this reinforcement terminates 900mm below the underside of the roof slab and the ACI manual does not permit unreinforced concrete. The walls along the longer plan dimension were found to have moderately insufficient temperature and shrinkage reinforcement in the horizontal direction due to the restrained length of the wall.

The existing concrete raft slab appeared to have been designed as a one-way member spanning in the short plan dimension between the concrete walls. This raft slab was found to have sufficient flexural reinforcement against the applied bearing pressure and sufficient temperature and shrinkage reinforcement in the short plan dimension but significant insufficient temperature and shrinkage reinforcement in the long plan dimension. A subgrade modulus of 27.1 MPa/m was assumed, which is a typical value in modeling the springiness of the bearing material.

6.0 Discussions & Recommendations

Based on our observations, we have concluded that the concrete work in all 3 reservoirs generally appeared to be in good condition as no major cracks were noted and the various concrete surfaces appeared to be sound throughout.

In regards to the Zone 1 Reservoir exterior walls being significantly deficient in terms of the amount of flexural reinforcement required to resist the applied lateral earth pressure loads, the geotechnical parameters commonly assumed to calculate these applied lateral

earth pressure loads generally tend to be conservative compared to precise geotechnical parameters obtained from a geotechnical engineer. It also needs to be noted that this structure has been in operation for more than 30 years and the concrete work in this structure did not appear to exhibit any signs of catastrophic overload. *CWMM* recommends that the slight bulge of the north exterior wall be measured and further, and the lateral earth pressure parameters be further investigated as precautionary measures. These precautionary measures can likely be incorporated into a future maintenance inspection.

To help ensure longevity of these reservoirs, many of the items noted in this assessment should receive maintenance attention at some point in the future, especially the leaking honeycombed section and corroding embed plates noted in the Zone 1 Reservoir.

Although the Zone 2 Reservoir was found to be able to support the weight of a maximum 4000 kg vehicle, signage should be clearly posted or bollards installed around this reservoir to deter any kind of vehicular traffic since design load restrictions can be easily misunderstood and/or abused. Signage and bollards should also be installed around the Zone 1 Reservoir and Zone 3 Pump Chamber to prevent vehicular overstress of those roof slabs altogether.

The fixed ladders and their corresponding supports in the Zone 2 Reservoir and the Zone 3 Pump Chamber should be replaced with stainless steel ladders and supports that meet the requirements of ANSI A14.3-08. The fixed aluminum ladders and their supports in the Zone 1 Reservoir should be cleaned of any corrosion build-up and painted with an approved corrosion inhibiting coating. If the supports on the bottom are of dissimilar metals, they should be replaced with similar galvanic metals to the aluminum. An issue with the access hatches in the Zone 2 Reservoir and the Zone 3 Pump Chamber is that their clearances would likely complicate the rescue of a person if an incident were to occur. As a minimum, a reduced clearance deflector plate should be installed in these reservoirs as shown in Appendix C.

CWMM cannot make any conclusions on the water tightness of these reservoirs. Even though each of these reservoirs contain structural members that have insufficient temperature and shrinkage reinforcement, which are intended to help limit the amount of cracking the concrete undergoes, significant cracking was not observed and leakage concerns are low. If complete assurance for water tightness of these reservoirs is desired, then an approved leak test will need to be carried out.

We trust this is satisfactory to you, and we are available to discuss any of the above at your convenience.

Yours truly,

CWMM CONSULTING ENGINEERS LTD.

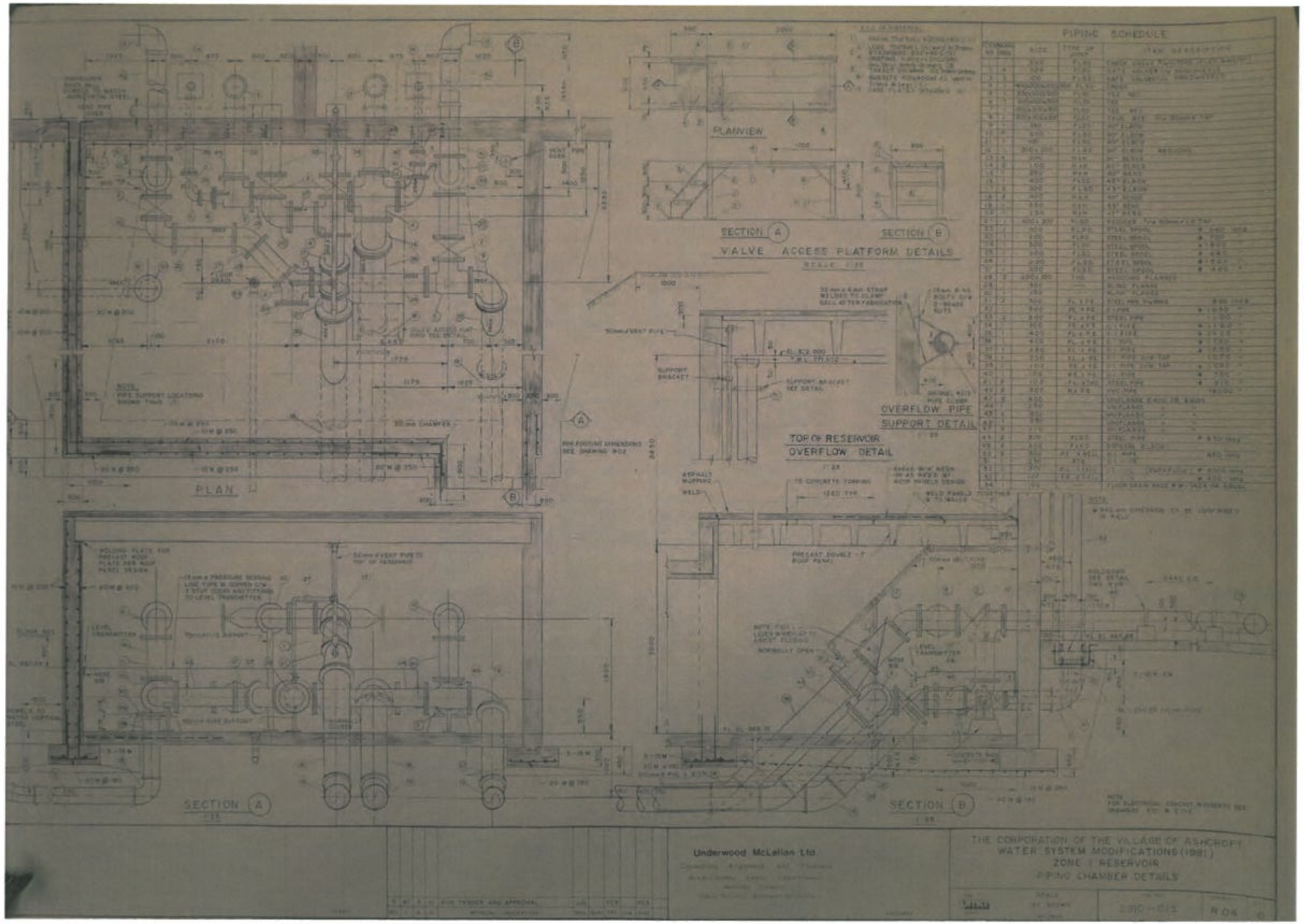
Prepared by:

Brendan Murtagh, P. Eng.

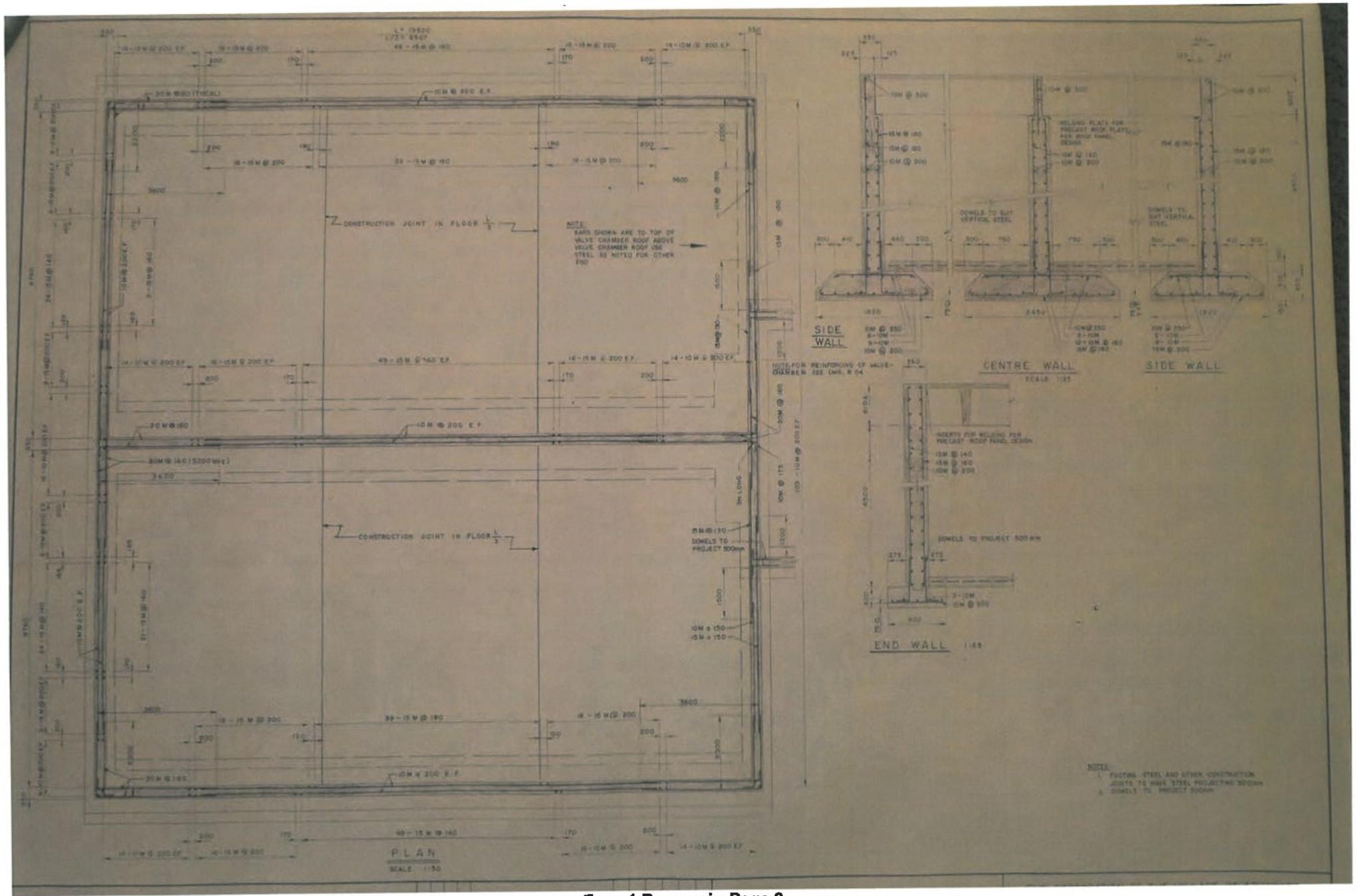
Reviewed By:

Michael Weilmeier, P. Eng., Associate

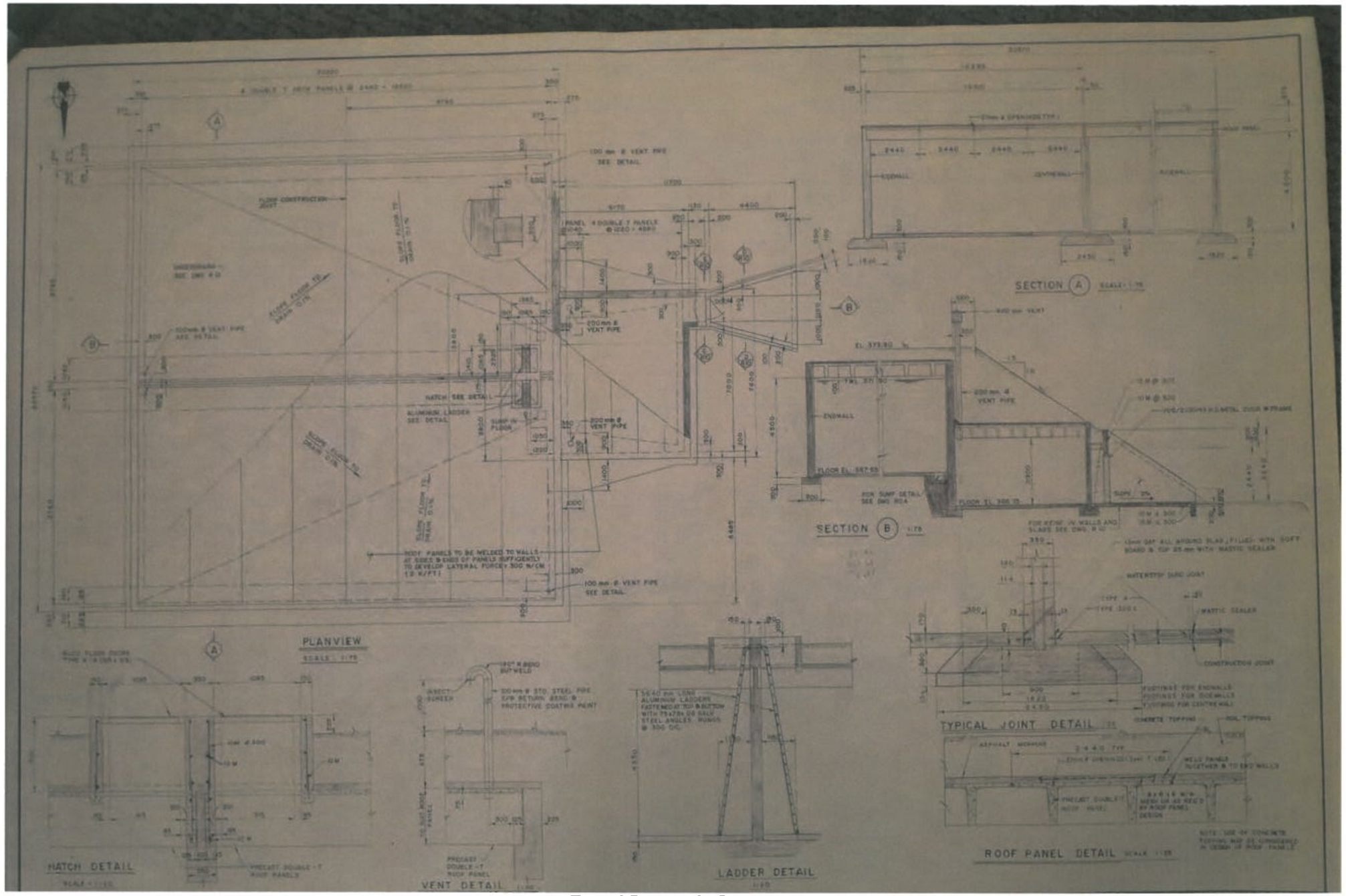
Appendix A – Existing Structural Drawings



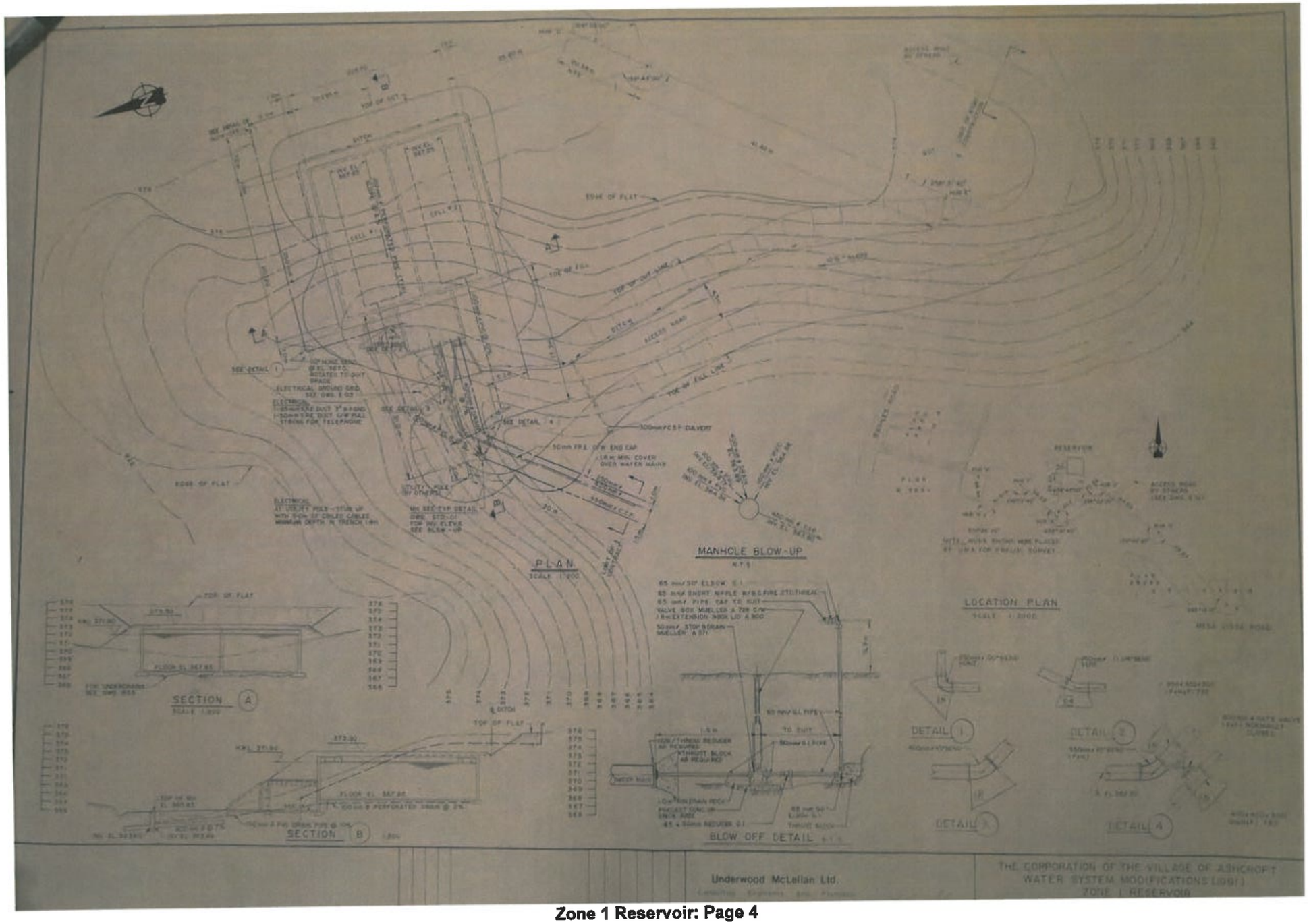
Zone 1 Reservoir: Page 1



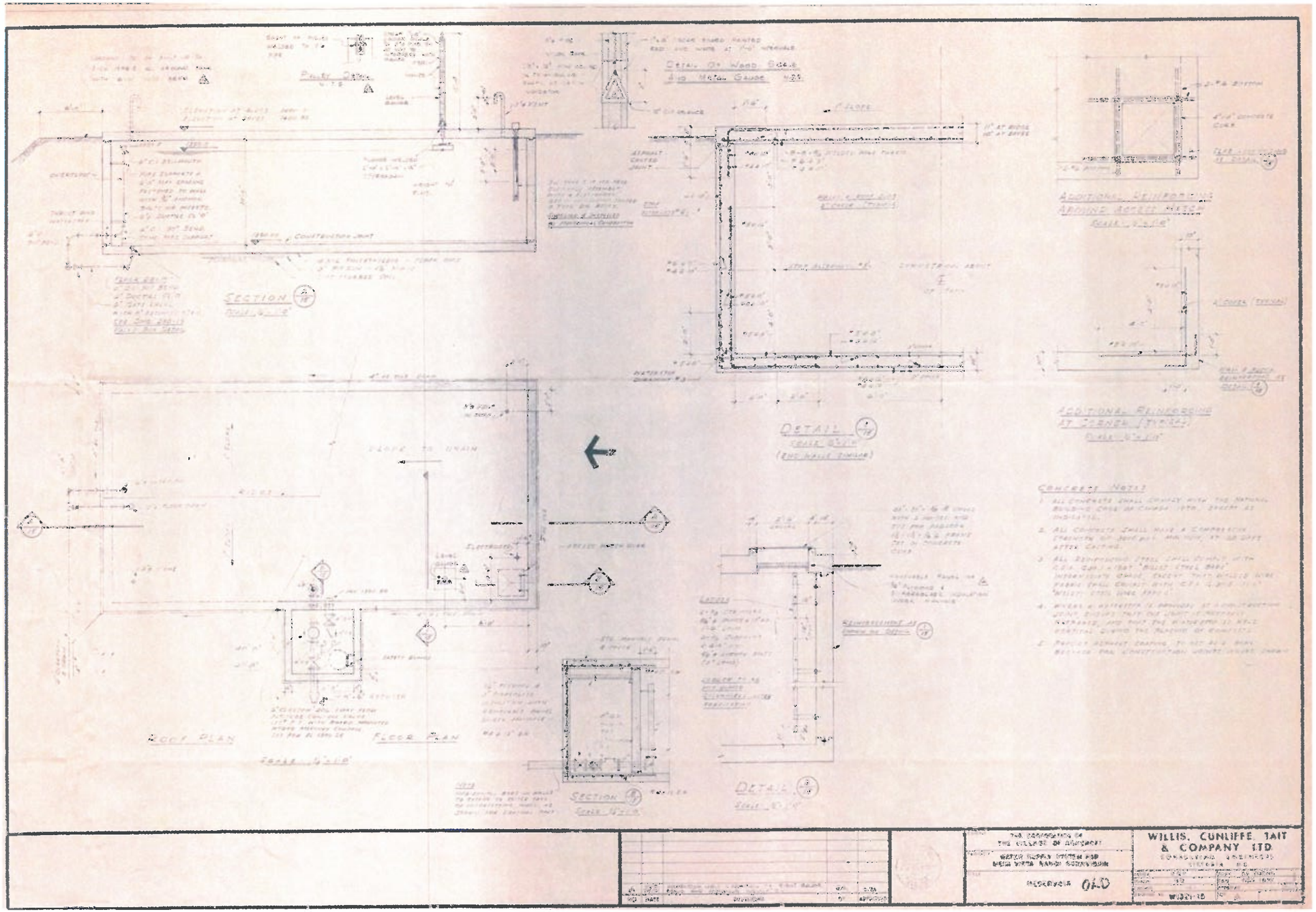
Zone 1 Reservoir: Page 2



Zone 1 Reservoir: Page 3



Zone 1 Reservoir: Page 4



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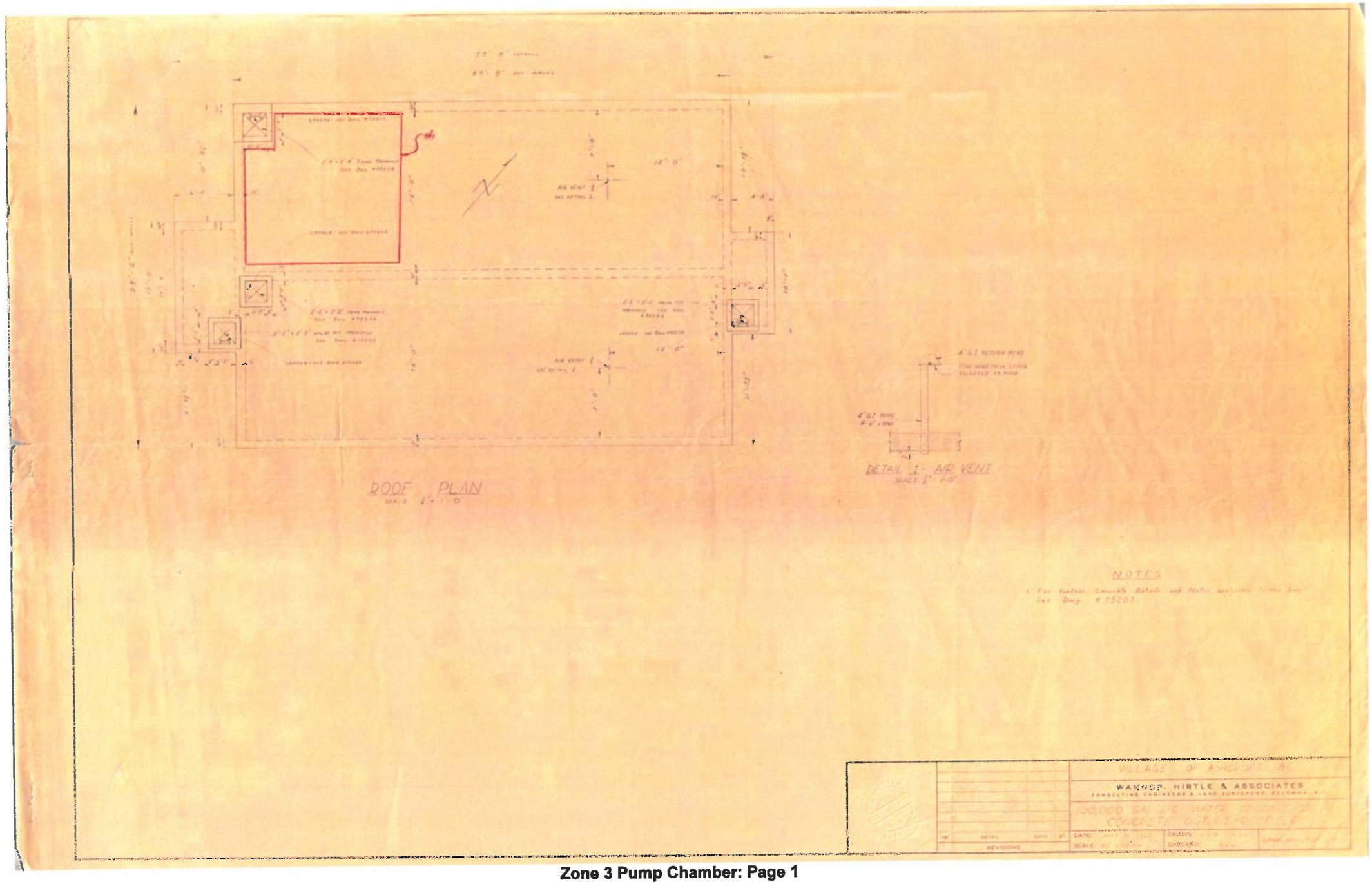
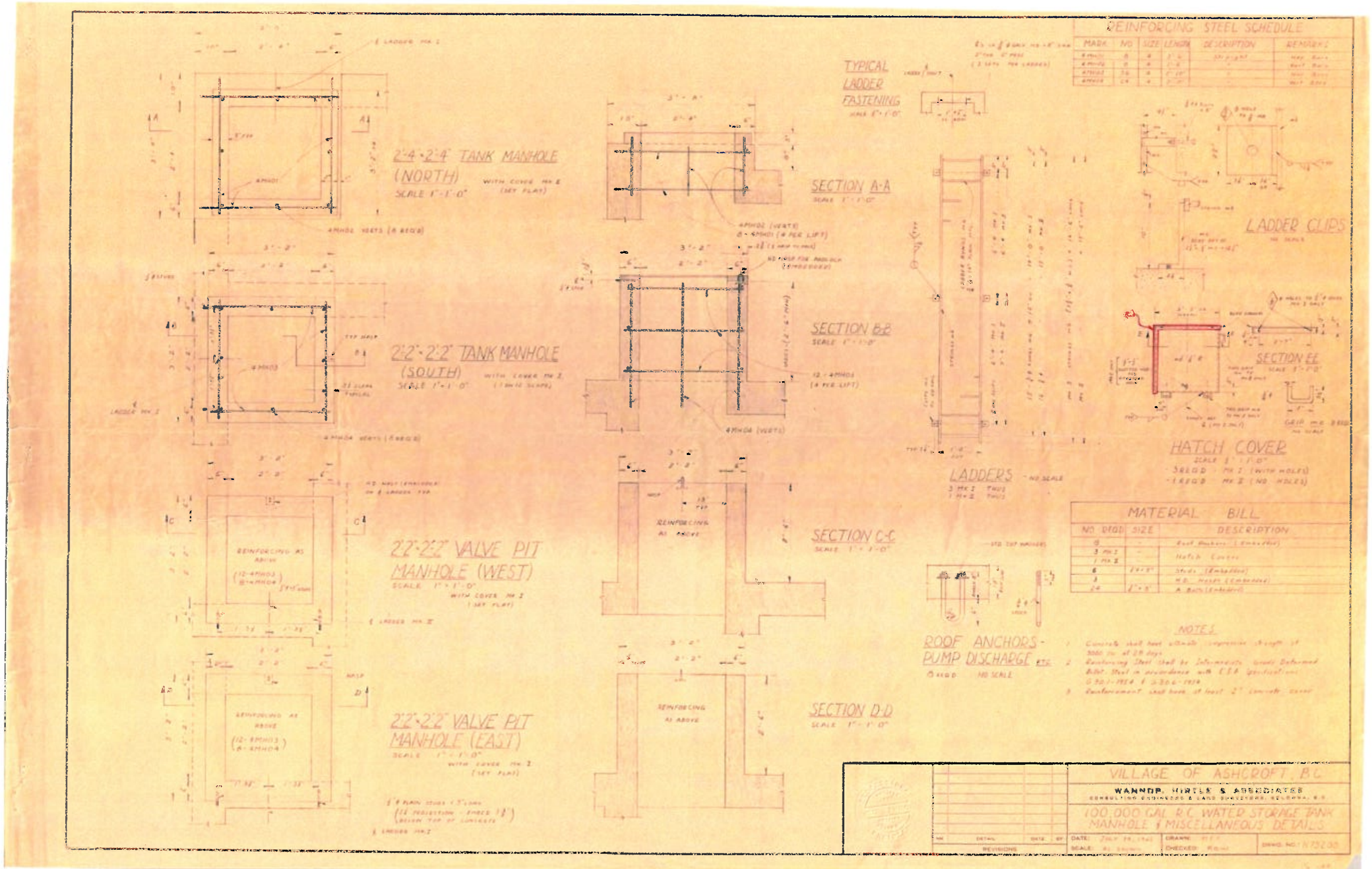
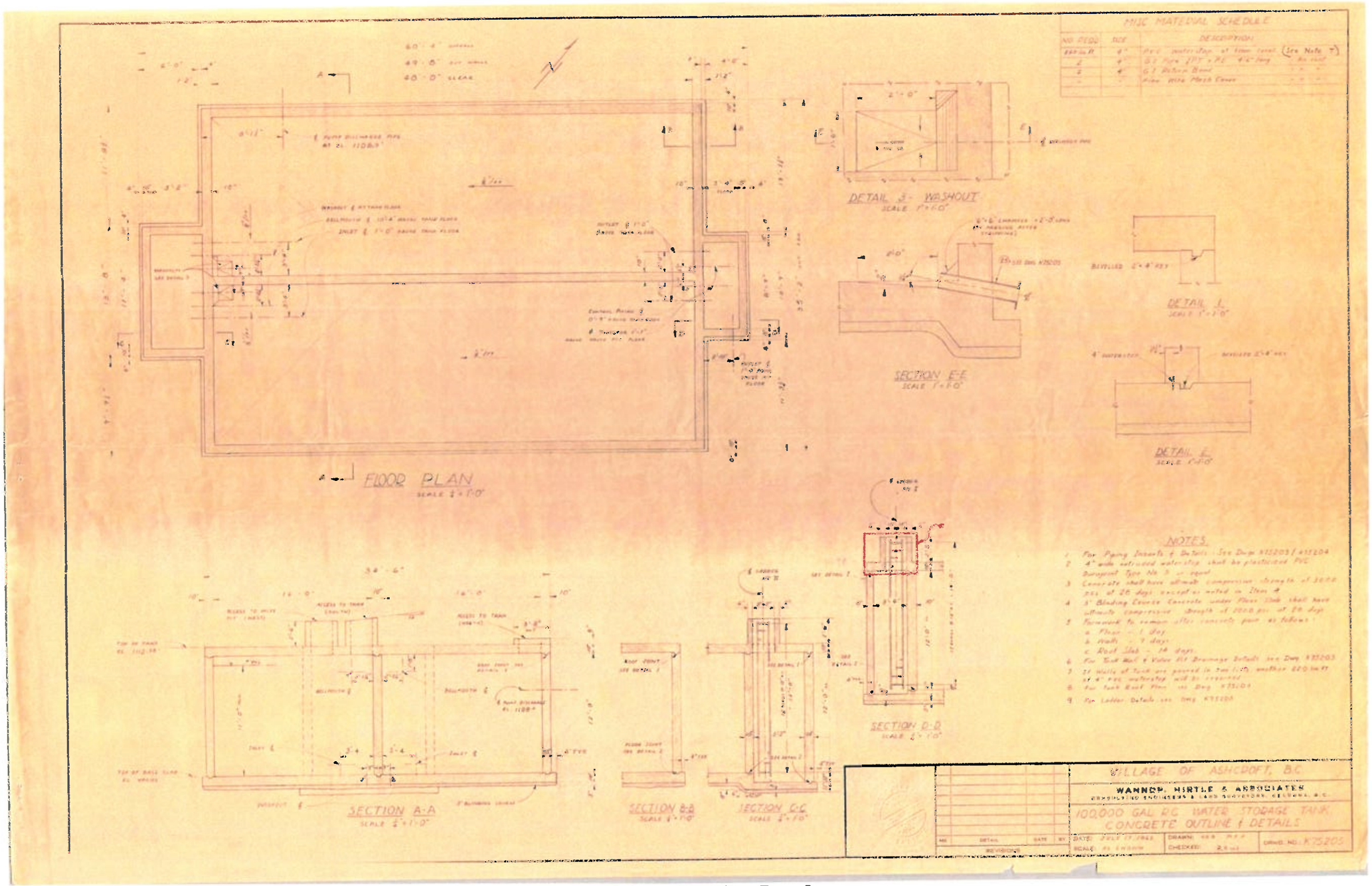


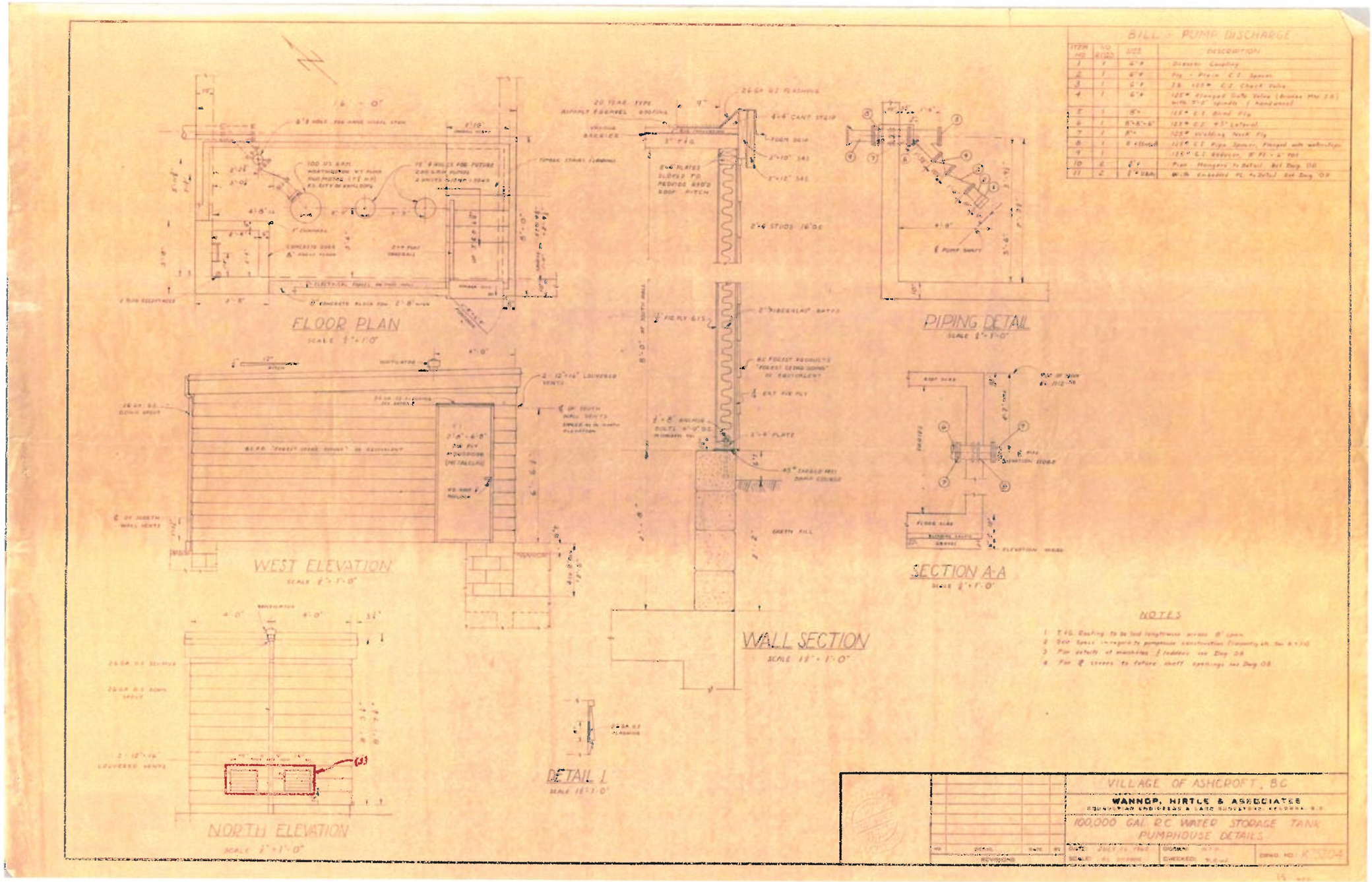
Zone 3 Pump Chamber: Page 1



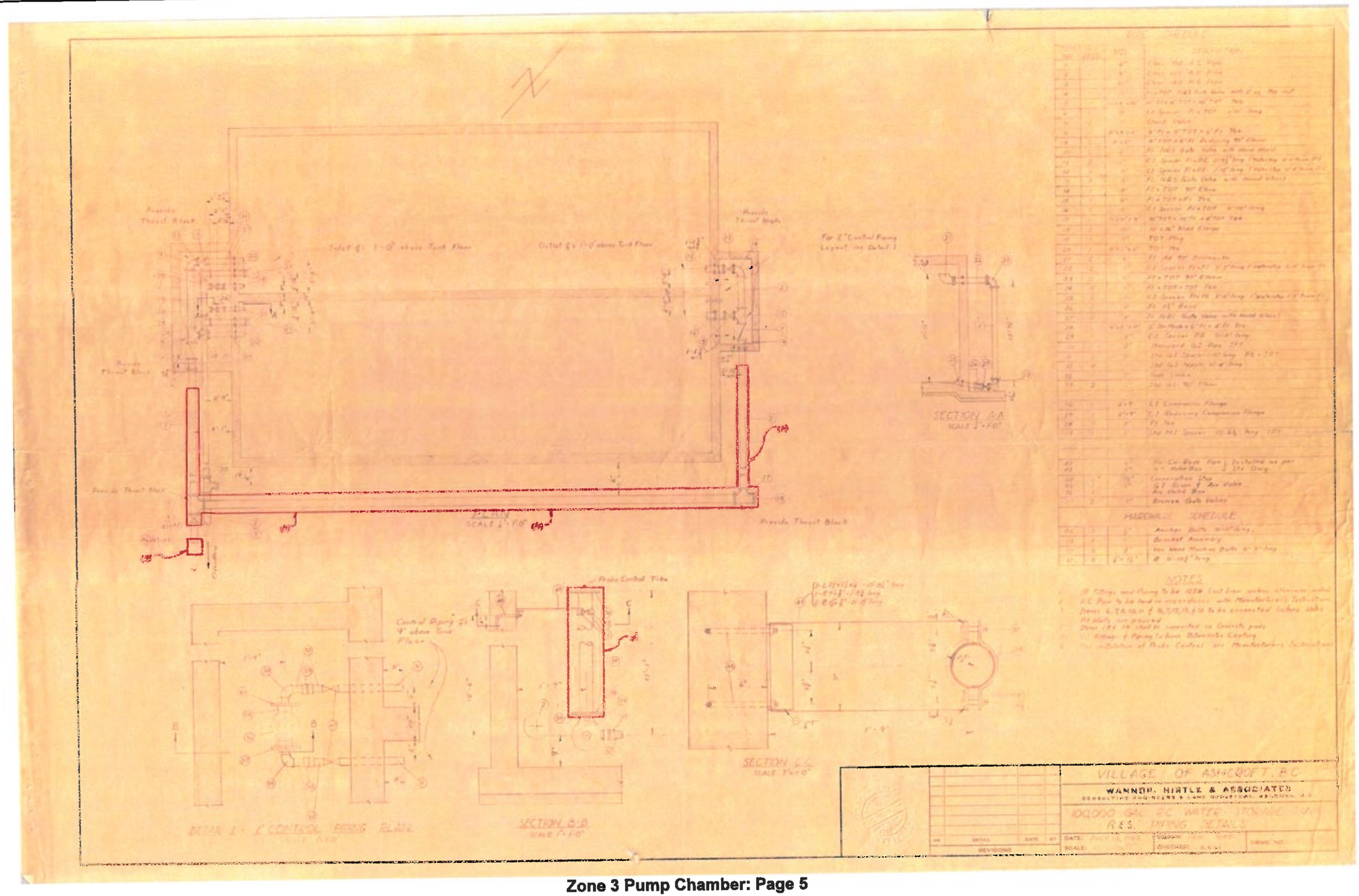
Zone 3 Pump Chamber: Page 2



Zone 3 Pump Chamber: Page 3



Zone 3 Pump Chamber: Page 4



Zone 3 Pump Chamber: Page 5

Appendix B – Site Photos

Photo 1 (Zone 1 Reservoir)

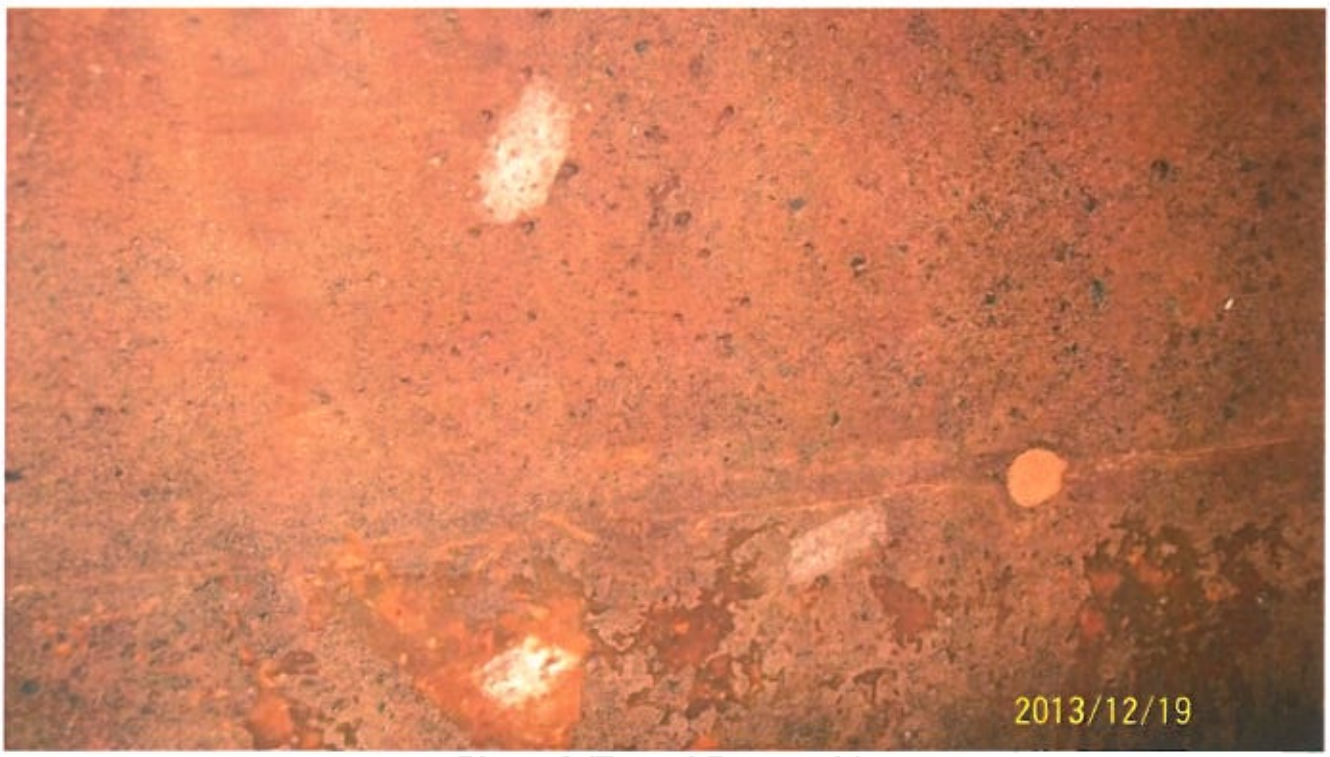


Photo 2 (Zone 1 Reservoir)

Photo 3 (Zone 1 Reservoir)

Photo 4 (Zone 1 Reservoir)

Photo 5 (Zone 1 Reservoir)

Photo 6 (Zone 1 Reservoir)

Photo 7 (Zone 1 Reservoir)

Photo 8 (Zone 1 Reservoir)

Photo 9 (Zone 1 Reservoir)

Photo 10 (Zone 1 Reservoir)

Photo 11 (Zone 1 Reservoir)

Photo 12 (Zone 1 Reservoir)

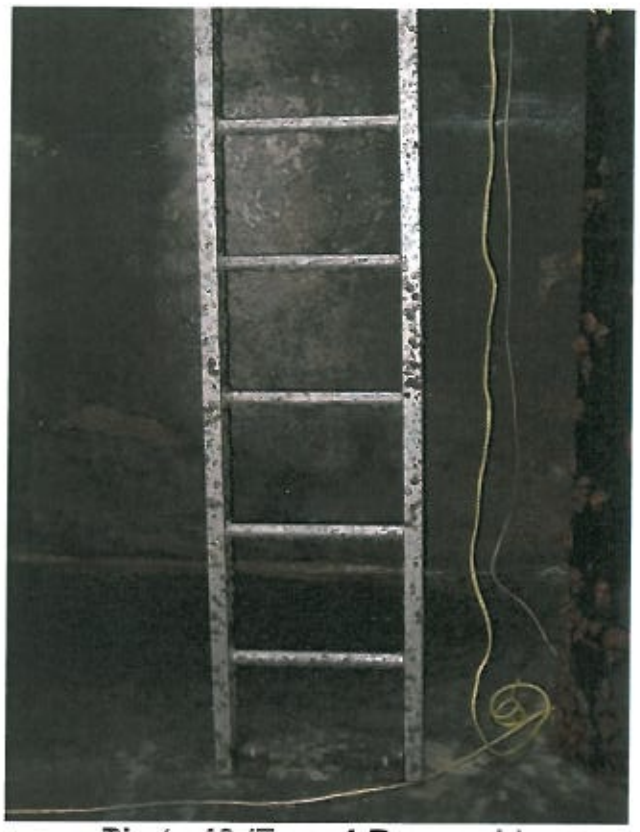


Photo 13 (Zone 1 Reservoir)

Photo 14 (Zone 2 Reservoir)

Photo 15 (Zone 2 Reservoir)

Photo 16 (Zone 2 Reservoir)

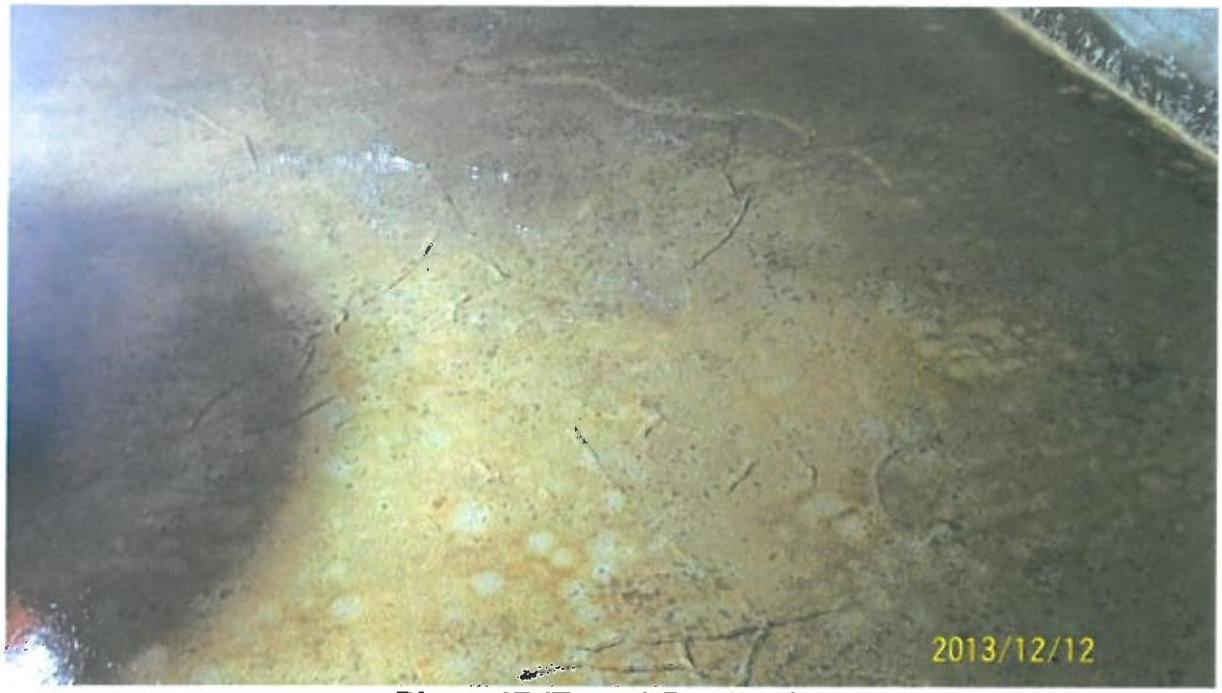


Photo 17 (Zone 2 Reservoir)

Photo 18 (Zone 2 Reservoir)

Photo 19 (Zone 2 Reservoir)

Photo 20 (Zone 2 Reservoir)

Photo 21 (Zone 2 Reservoir)

Photo 22 (Zone 3 Pump Chamber)

Photo 23 (Zone 3 Pump Chamber)

Photo 24 (Zone 3 Pump Chamber)

Photo 25 (Zone 3 Pump Chamber)

Photo 26 (Zone 3 Pump Chamber)

Photo 27 (Zone 3 Pump Chamber)

Photo 28 (Zone 3 Pump Chamber)

Photo 29 (Zone 3 Pump Chamber)

Photo 30 (Zone 3 Pump Chamber)

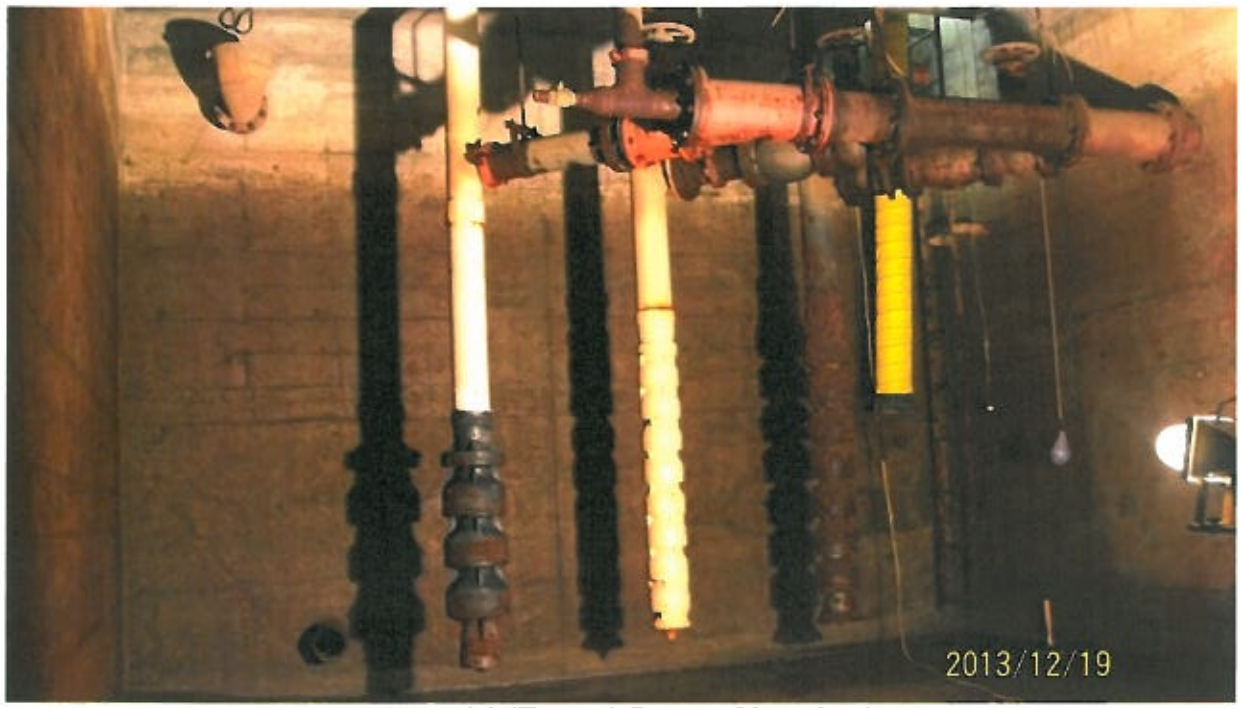


Photo 31 (Zone 3 Pump Chamber)

Appendix C – ANSI A14-3-2008

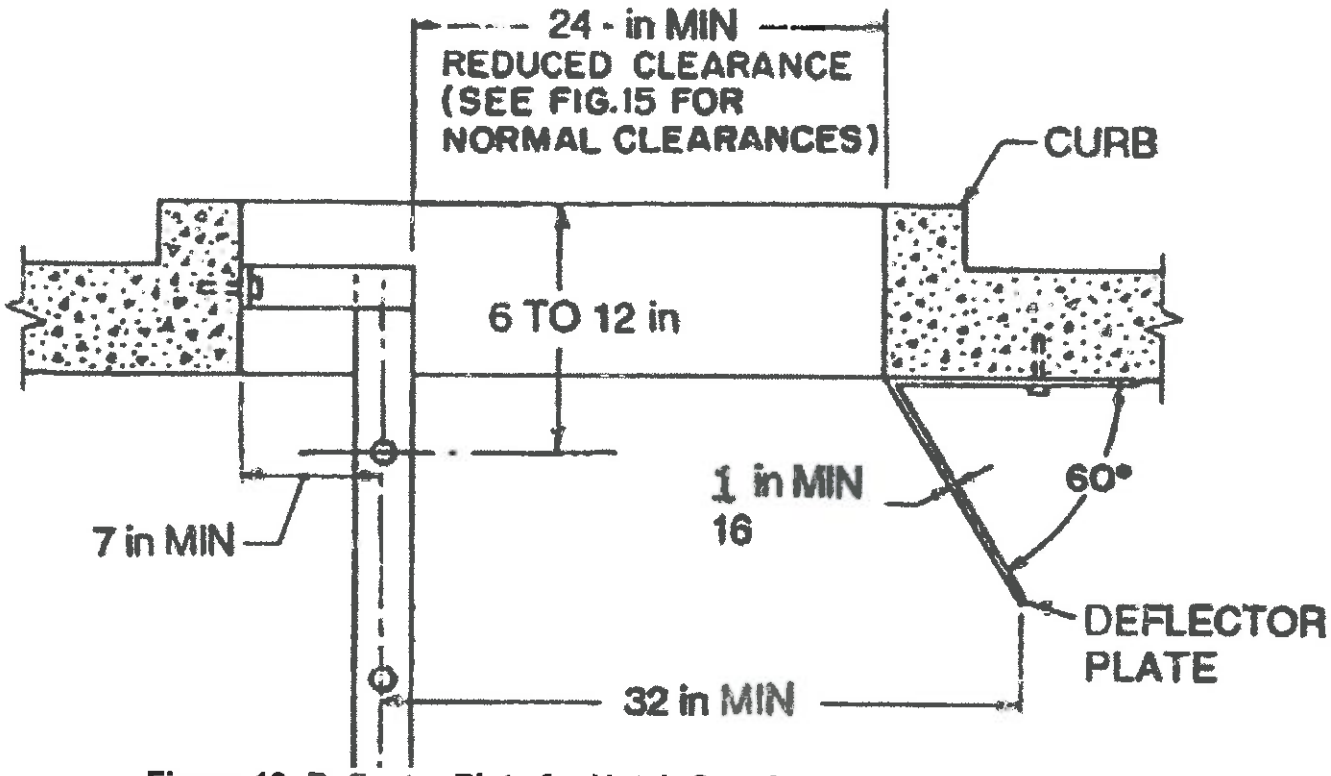


Figure 13: Deflector Plate for Hatch Opening with Reduced Clearance

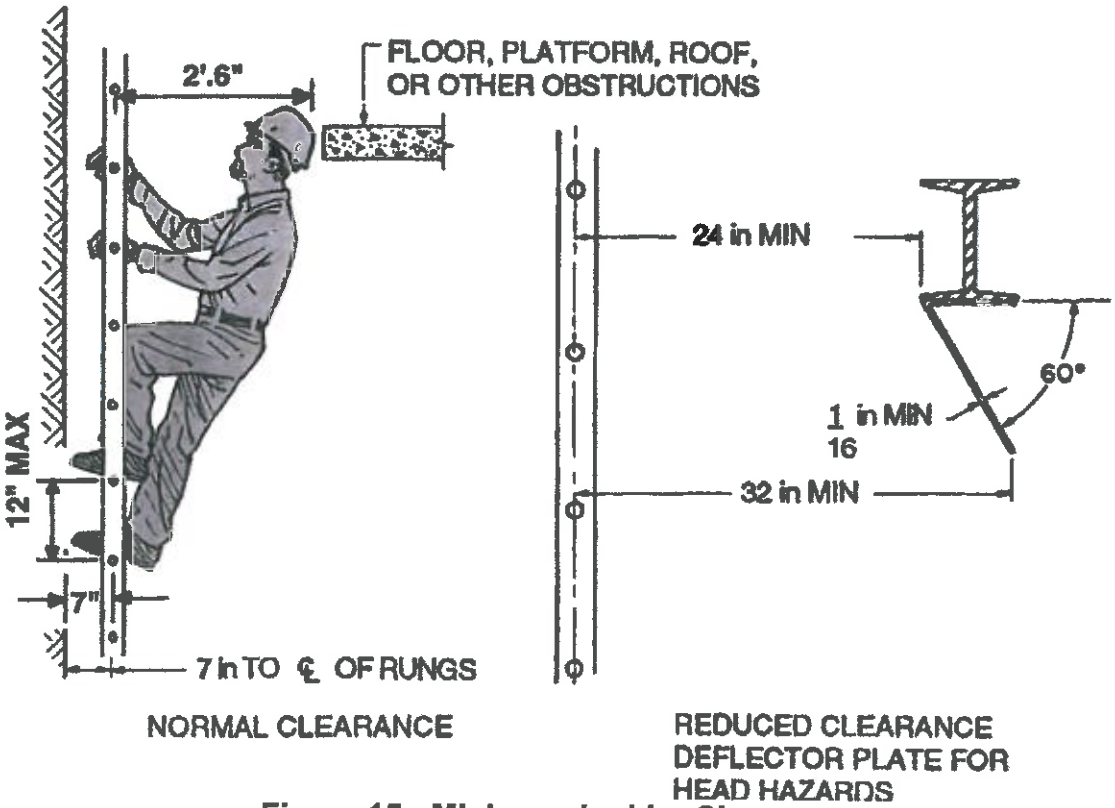


Figure 15 : Minimum Ladder Clearance

Appendix B

Drinking Water Treatment Objectives (Microbiological) for Surface Water Supplies in British Columbia

Prepared by BC Ministry of Health

Issued November 2012

DRINKING WATER TREATMENT OBJECTIVES (MICROBIOLOGICAL) FOR SURFACE WATER SUPPLIES IN BRITISH COLUMBIA

VERSION 1.1 / NOVEMBER 2012

1. Objective

Provide a general overview of microbiological drinking water treatment objectives for surface water supplies in British Columbia.

2. Background and Regulatory Framework

There are three main types of micro-organisms (pathogens) that pose risks to human health in drinking water: viruses, bacteria and protozoa. The B.C. *Drinking Water Protection Act* (DWPA) (2001) and Drinking Water Protection Regulation (DWPR) (2003) specify water quality standards, monitoring schedules, applicability and recommended treatment aimed at reducing the risks from these pathogens.

Schedule A of the DWPR specifies bacteriological water quality standards for potable water[1] for the protection of human health. These standards represent partial drinking water treatment goals and are consistent with the *Guidelines for Canadian Drinking Water Quality: Guideline Technical Document — Escherichia coli* and total coliform (Health Canada, 2006).

Schedule B of the DWPR outlines the monitoring schedule and its applicability based on population served. Section 5 of the regulation requires that surface water sources must, as a minimum, receive disinfection. Reducing risks from virus and protozoa through disinfection of drinking water are dealt with through the application of best management principles as outlined in this document and detailed in the Guidelines for Canadian Drinking Water Quality (GCDWQ). As no one type of treatment system is effective in treating all hazards, a multi-barrier approach is usually required to adequately address all risks, which typically includes two or more forms of treatment.

The DWPA and the DWPR give drinking water officers (DWOs) the flexibility and discretion to address public health risks through treatment requirements in operating permits to deal with pathogenic risks. Discretion of the drinking water officer also includes, but is not limited to, understanding the source water characterization, effectiveness of system-specific treatment technologies, operational management issues and reasonable time frames to achieve incremental improvements in existing systems. With respect to water quality analyses, the issuing official should ensure that he/she has

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[1] Potable water is defined under the *Drinking Water Protection Act* as water provided by a domestic water system that (a) meets the standards prescribed by regulation, and (b) is safe to drink and fit for domestic purposes without further treatment.

adequate data to determine that the proposed treatment is adequate to address public health risks in relation to relevant microbiological and chemical/physical parameters.

Existing water supply systems may have some appreciable risk for certain parameters without treatment in place. In such cases, it is acceptable from a public health perspective for water supply systems to present drinking water officers with a continuous improvement plan that addresses implementing treatment for these parameters within a reasonable time period.

3. Purpose and Scope

Under the DWPA, water suppliers are responsible for providing potable water to all users of their systems. Drinking water treatment requirements are site specific, risk based and dependent on a number of factors, including source water quality and efficacy of treatment technology.

This document provides the basic, minimum framework towards goals for drinking water treatment for pathogens in surface water supply systems in British Columbia. It may also be used as a general reference for assessing progress towards updating or improving existing water supply systems. This document does not address the treatment of groundwater or disinfection of distribution systems.

These objectives use the *Guidelines for Canadian Drinking Water Quality* (Health Canada, 2012) as a primary reference for potability. However, given site-specific conditions of water systems in various regions of B.C., it is necessary to apply these guidelines in consideration of a risk assessment of individual cases. In all cases, the drinking water officer must be contacted to confirm the necessary treatment objectives for microbiological parameters when planning or upgrading water supply systems.

4. Treatment Objectives

These objectives provide treatment requirements that address the following microbiological parameters: enteric viruses, pathogenic bacteria, *Giardia* cysts and *Cryptosporidium* oocysts. The general objectives are as follows and described in more detail below:

- 4-log reduction or inactivation of viruses.
- 3-log reduction or inactivation of *Giardia* and *Cryptosporidium.*
- Two treatment processes for surface water.
- Less than or equal to (≤) one nephelometric turbidity unit (NTU) of turbidity.
- No detectable *E. Coli*, fecal coliform and total coliform.

These drinking water treatment objectives provide a minimum performance target for water suppliers to treat water to produce microbiologically safe drinking water. Depending on specific situations, the actual amount of treatment required will depend on the risks identified and may require greater levels of treatment. Water treatment is only one part of the multi-barrier approach to providing safe drinking water. Choosing an appropriate water source, protecting that source and reducing distribution system risks can be essential complementary steps to providing treatment when dealing with microbiological risks.

While there are numerous precautionary treatment steps available to reduce the risk of microbiological contamination of drinking water supplies, no system is fail-safe. Risk management is based on applying

scientific evidence that documents the quality and variability of the water source and the efficacy of management measures selected to achieve acceptable public health outcomes.

4.1. 4-log Inactivation of Viruses

Viruses are micro-organisms that are incapable of replicating outside a host cell. In general, viruses are host specific, which means that viruses that infect animals or plants do not usually infect humans, although a small number of enteric viruses have been detected in both humans and animals (Health Canada, 2010). Viruses are ubiquitous and often species-specific. Viruses of concern in drinking water are those that cause human illness or are capable of cross-species transfer. The role of nonhuman viruses as facilitators of pathogens or in transmitting genetic material that could be pathogenic is not clearly understood; hence, overall reductions of viruses in source water are preferred.

Health Risk Management Outcomes for Enteric Viruses

The level of risk deemed tolerable or acceptable by Health Canada for enteric viruses has been adopted from the World Health Organization's (WHO) *Guidelines for Drinking-Water Quality* (WHO, 2004; cited in Health Canada, 2010) based on the Disability Adjusted Life Year (DALY) as a unit of measure for risk.

The basic principle of the DALY is to calculate a value that considers both the probability of experiencing an illness or injury and the impact of the associated health effects (Murray and Lopez, 1996a; Havelaar and Melse, 2003; cited from Health Canada, 2010). The WHO (2004) guidelines adopt 10^{-6} DALY/person per year as a health risk management target. Table 1 describes the relationship between viruses in source water and the level of treatment necessary to achieve this health risk management goal.

Table 1: Overall treatment requirements for virus log reduction as a function of approximate source water concentration to meet a level of risk of 1×10^{-6} DALY/person per year (Health Canada, 2010)

Source water virus concentration (no./100 L)	Overall required treatment reduction for viruses ($\log_{10}$)
1	4
10	5
100	6
1000	7

Treatment Objectives for Enteric Virus

A minimum 4-log reduction of enteric viruses is recommended for all surface water sources. Depending on the surface water source, especially those subject to human fecal contamination, a greater than 4-log reduction may be necessary (See Table 1).

Reductions can be achieved through physical removal processes, such as filtration, and/or through inactivation processes, such as disinfection (Health Canada, 2010). Disinfection of water systems is recommended as a means to provide safeguards to the water system. Enteric viruses are readily inactivated by the use of chemical disinfection such as chlorine.

Ultraviolet (UV) light disinfection systems may be used to reduce viruses in water, but the effectiveness of UV varies significantly among different types of viruses. Double-stranded DNA viruses, such as adenoviruses, are more resistant to UV radiation than single-stranded RNA viruses, such as HAV (Meng and Gerba, 1996; cited in Health Canada, 2010).

Because of their high level of resistance to UV treatment and because some adenoviruses can cause illness, particularly in children and immunocompromised adults, adenoviruses have been used by the U.S. EPA as the indicator pathogen for establishing UV light inactivation requirements for enteric viruses in the *Long Term 2 Enhanced Surface Water Treatment Rule* (LT2ESWTR) (U.S. EPA, 2006). Accordingly, the LT2ESWTR requires aUV dose of 186 mJ/cm^2 to achieve 4-log inactivation of viruses (U.S. EPA, 2006).

For water supply systems in Canada, UV disinfection is commonly applied, most often in combination with chlorine disinfection or other physical removal barriers such as filtration (Health Canada, 2010). A UV dose of 40 mJ/cm^2 is considered to be protective of human health as most enteric viruses are inactivated at this dosage; however, this dosage would provide only a 0.5-log inactivation of adenovirus. Additional log removal credits may be obtained through the addition of free chlorine.

For drinking water sources considered to be less vulnerable to human fecal contamination, the drinking water officer may accept an enteric virus such as rotavirus as the target pathogen to determine the UV dose required for 4-log inactivation of viruses. Where a system relies solely on UV disinfection for pathogen control and the source water is known or suspected to be contaminated with human sewage[2], either a higher UV dose such as that stated in the LT2ESWTR or a multi-barrier treatment strategy should be adopted.

The physical removal of viruses can be partially achieved by clarification and filtration processes. Clarification is generally followed by the filtration process. Some filtration systems, however, are used without clarification (direct filtration). Many treatment processes are interdependent and rely on optimal conditions upstream in the treatment process for efficient operation of subsequent treatment steps.

Drinking water treatment plants that meet the turbidity limits established in the *Guidelines for Canadian Drinking Water Quality: Supporting Documentation — Turbidity* (Health Canada, 2003) can apply the estimated physical removal credits for enteric viruses. For example, for conventional filtration, the virus credit is 2-log and for direct filtration the virus credit is 1-log.

Alternatively, log removal rates can be established on the basis of demonstrated performance or pilot studies. The physical log removal credits can be combined with the disinfection credits to meet overall treatment goals. In all cases, the drinking water officers must be consulted when planning treatment for a water supply system.

It is recommended that water supply systems should provide, as a minimum, 4-log reduction of viruses for all surface water systems.

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[2] The Ministry of Health is awaiting further clarification from Health Canada as to what constitutes as *human fecal contamination*. In lieu of clarification, it is best to use as much available information as possible to make an informed decision on a case-by-case basis.

4.2. 3-log Inactivation of *Giardia* and *Cryptosporidium*

Protozoa such as *Giardia* and *Cryptosporidium* are relatively large pathogenic micro-organisms that multiply only in the gastrointestinal tract of humans and other animals. They cannot multiply in the environment, but their cysts/oocysts can survive in water longer than intestinal bacteria, and they are more infectious and resistant to disinfection than most other micro-organisms (Health Canada, 2004).

Health Risk Management Outcomes for Giardia and Cryptosporidium

While *Giardia* and *Cryptosporidium* can be responsible for severe and, in some cases, fatal gastrointestinal illness, the *Guidelines for Canadian Drinking Water* have not established maximum acceptable concentrations for these protozoa in drinking water. Routine methods available for the detection of cysts and oocysts have low recovery rates and do not provide any information on their viability or human infectivity. Until better monitoring data and information on the viability and infectivity of cysts and oocysts present in drinking water are available, measures should be implemented to reduce the risk of illness as much as possible.

Treatment Objectives for *Giardia* and *Cryptosporidium*

The goal of surface water treatment is to reduce the presence of disease-causing organisms and associated health risks to an acceptable safe level.

Treatment of drinking water is another integral part of the multi-barrier approach. In addition to disinfection, where warranted by source water conditions, physical treatment of surface supplies should be included. Because *Giardia* and *Cryptosporidium* are ubiquitous in surface waters in Canada and more resistant to disinfection than most other infectious organisms, it is desirable that treatment achieves at least a 99.9% (3-log) reduction of *Giardia* and *Cryptosporidium* (Health Canada, 2004).

Giardia may be partially inactivated by large doses of free chlorine, ozone or chlorine dioxide. Filtration can be effective in removing *Giardia* cysts and *Cryptosporidium* oocysts, but the performance is significantly dependant on the methods of filtration and operational performance. *Giardia* and *Cryptosporidium* may also be inactivated using UV disinfection. Many commercially available UV systems have undergone testing to verify that the dosage provided under design operating conditions achieves the 3-log inactivation required.

It is recommended that water supply systems should provide, as a minimum, 3-log reduction of *Giardia* and *Cryptosporidium* for systems that have a water source considered to have low risk of these parasites and have not had an outbreak of the disease. A higher level of reduction may be required if the situation justifies it.

4.3. Two Methods of Treatment (Dual Treatment)

Health Risk Management Outcomes for Dual Treatment of Drinking Water

Some microbiological agents of concern are more resistant to certain forms of treatment than others. Ultimately, the best approach to ensure complete disinfection of water intended for human use is a multi-barrier one, which begins with collecting water from the cleanest source possible.

As most disinfection systems require clear water to ensure maximum efficiency, it may be necessary to combine multiple specific treatment technologies. To provide the most effective protection, the *Guidelines for Canadian Drinking Water* recommend that filtration and one form of disinfection be used to meet the treatment objectives.

Alternatively, two forms of disinfection (for example, chlorination and UV disinfection) may be considered if certain criteria are met.

A water supply system may be permitted to operate without filtration if the following conditions for exclusion of filtration are met, or a timetable to implement filtration has been agreed to by the drinking water officer:

1. Overall inactivation is met using a minimum of two disinfections, providing 4-log reduction of viruses and 3-log reduction of *Cryptosporidium* and *Giardia*.
2. The number of *E. coli* in raw water does not exceed 20/100 mL (or if *E. coli* data are not available less than 100/100 mL of total coliform) in at least 90% of the weekly samples from the previous six months. The treatment target for all water systems is to contain no detectable *E. coli* or fecal coliform per 100 ml. Total coliform objectives are also zero based on one sample in a 30-day period. For more than one sample in a 30-day period, at least 90% of the samples should have no detectable total coliform bacteria per 100 ml and no sample should have more than 10 total coliform bacteria per 100 ml.
3. Average daily turbidity levels measured at equal intervals (at least every four hours) immediately before the disinfectant is applied are around 1 NTU, but do not exceed 5 NTU for more than two days in a 12-month period.
4. A watershed control program is maintained that minimizes the potential for fecal contamination in the source water. (Health Canada, 2003)

Applying the exclusion of filtration criteria does not mean filtration will never be needed in the future. A consistent supply of good source water quality is critical to the approach, but source quality can change. Therefore, the exclusion of filtration must be supported by continuous assessment of water supply conditions.

Changing source water quality can occur with changes in watershed conditions. Increased threats identified through ongoing assessment and monitoring may necessitate filtration. Maintaining the exclusion condition relies on known current and historic source water conditions, and provides some level of assurance to water suppliers that a filtration system may not be necessary unless the risk of adverse source water quality increases.

It is recommended that dual water treatment should be applied to all surface water.

4.4 ≤1 NTU in Turbidity

Events such as sedimentation from road surfaces, higher surface runoff peak flows, landslides and debris flows increase a condition commonly referred to as "turbidity." Turbidity in water is caused by suspended organic and colloidal matter, such as clay, silt, finely divided organic and inorganic matter, bacteria, protozoa and other microscopic organisms. It is measured in nephelometric turbidity units (NTU) and is generally acceptable when less than 1 NTU, as per the exclusion criteria in section 4.3, and becomes visible when above 5 NTU.

Health Risk Management Outcomes for Turbidity

Turbidity is an indicator of the potential presence of human pathogens such as bacteria and protozoa. Furthermore, a greater concentration of organic and/or microbiological matter in source water has the potential to disrupt or overload drinking water disinfection processes, such as UV light and chlorination, to the point that they may no longer effectively control pathogens in the water. In

addition, organic matter in the water can react with disinfectants such as chlorine to create byproducts that may cause adverse health effects (Health Canada, 2003).

Treatment Objectives for Turbidity

In general, turbidity is caused by particles in water and can be effectively reduced by filtration. Depending on the filtration technologies applied to the water, filtered water from well operated filtration systems could have turbidity ranges from 0.1 to 1.0 NTU. The Canadian guideline on turbidity applies to filtered surface water and is categorized by the type of filtration technology: conventional and direct filtration; slow sand or diatomaceous earth filtration; and membrane filtration. To comply with the Canadian guideline on turbidity, continuous monitoring of turbidity is required.

Turbidity is effectively reduced through filtration, using one of a number of common technologies. The goal of treating water for turbidity is to reduce its level to as low as possible and minimize fluctuation. For this reason, when filtration technology is employed, the system should strive to achieve a treated water turbidity target from individual filters or units of less than 0.1 NTU at all times. Where this is not achievable, the treated water from filters or units should be less than or equal to 0.3 NTU for conventional and direct filtration; less than or equal to 1.0 NTU for slow sand or diatomaceous earth filtration; and less than or equal to 0.1 NTU for filtration systems that use membrane filtration. Inability to achieve these objectives in filtered systems indicates a breakdown of the treatment train and potential health impacts to users.

For nonfiltered surface water to be acceptable as a drinking water source supply, average daily turbidity levels should be established through sampling at equal intervals (at least every four hours) immediately before the disinfectant is applied. Turbidity levels of around 1.0 NTU but not exceeding 5.0 NTU for more than two days in a 12-month period should be demonstrated in the absence of filtration. In addition, source water turbidity should not show evidence of harbouring microbiological contaminants in excess of the exemption criteria in section 4.3 of this document.

It is recommended that turbidity of treated surface water should be maintained at less than 1 NTU. Where filtration is part of the treatment process, the turbidity levels should comply with the Canadian guideline on turbidity, entitled *Guidelines for Canadian Drinking Water Quality: Guideline Technical Document — Turbidity (Health Canada, 2003)* (expected turbidity reduction depends on the filtration methods). Continuous monitoring of turbidity should be required for water systems with filtration to verify compliance with system performance objectives. Systems that meet the criteria for exclusion from the requirement for filtration should be monitored to verify that the system continues to meet the exclusion criteria.

4.5. No Detectable *E. Coli*, Fecal Coliform and Total Coliform

E. coli and other fecal coliforms are members of the total coliform group of bacteria, but *E. coli* is the only member found exclusively in the feces of humans and other animals. Other members of the total coliform group (including fecal coliforms) are found naturally in water, soil, and vegetation, as well as in feces. The presence of *E. coli* and other fecal coliforms in water indicates not only recent fecal contamination, but also the possible presence of intestinal disease-causing bacteria, viruses, and protozoa.

Health Risk Management Outcome for *E. Coli* and Total Coliform

The absence of *E. coli*, fecal coliform and total coliform is used as an indicator that treated water is free from intestinal disease-causing bacteria. Their presence in drinking water distributed from a treatment plant indicates a serious failure and that corrective action is necessary. The presence of total coliform bacteria in the water distribution system indicates that the system may be vulnerable to contamination or experiencing bacterial regrowth.

Treatment Objectives for *E. coli*, Fecal Coliform and Total Coliform

E. coli, fecal coliform and total coliform are easily controlled with disinfection processes such as chlorine or UV light and can also be reduced by filtration. The DWPR calls for water suppliers to provide water with nondetectable *E. coli*, fecal coliform and total coliform based on sampling frequency established by the DWPR or through agreement with the drinking water officer.

In summary, according to Schedule A of the DWPR (updated 2008), the treatment target for all water systems is to contain no detectable *E. coli* or fecal coliform per 100 ml. Total coliform objectives are also zero based on one sample in a 30-day period. For more than one sample in a 30-day period, at least 90% of the samples should have no detectable total coliform bacteria per 100 ml and no sample should have more than 10 total coliform bacteria per 100 ml.

5. Conclusion

These objectives are intended to provide general requirements for surface water supply treatment systems in B.C. and rely on the *Guidelines for Canadian Drinking Water Quality* (Health Canada, 2012) as a primary reference for potability and treatment. However, given site-specific physical, chemical and biological conditions of water supplies throughout various regions in B.C., it may be necessary to apply these guidelines based on risk assessment of individual cases.

In all cases, the treatment objectives for microbiological parameters in specific water supply systems must be developed in consultation with a drinking water officer when planning or upgrading drinking water supply systems in the province.

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Appendix C

Cost Estimates

Village of Ashcroft
Water Master Plan

Physical Details						Cost Information						Budget Requirements	
Facility	Description	Quantity	Year Installed or Renewed	Service Life	Age	Unit Cost	Replacement Value	Loss in Value	Remaining Value	Expected Remaining Life	Infrastructure Deficit (Backlog)	20 Year Total	Average Annual Life Cycle Investment (AALCI)
Pipe													
Mains - Direct from L3		25,632					$ 12,704,240	$ 8,265,624	$ 4,438,616	35%	$ -	$ 4,584,860	$ 176,433
Mains - 5 Year Blocks of Time -USE THIS FOR CAPITAL ESTIMATES AND DEFICIT CALC.		25,632					$ 12,704,240	$ 8,265,624	$ 4,438,616	35%	$ -	$ 4,584,860	$ 176,433
Mains - 7 Year Moving Average		25,632					$ 12,704,240	$ 8,265,624	$ 4,438,616	35%	$ -	$ 4,584,860	$ 176,433
sing only the 5 Years Blocks of Time		**25,632**					**$ 12,704,240**	**$ 8,265,624**	**$ 4,438,616**	**$ 0**	**$ -**	**$ 4,584,860**	**$ 176,433**
Supply													
River Intake 1	- Pump - 25 Hp	1	2011	10	4	$ 15,000	$ 15,000	$ 6,000	$ 9,000	60%	$ -	$ 30,000	$ 1,500
	- Screens	1	1999	25	16	$ 20,000	$ 20,000	$ 12,800	$ 7,200	36%	$ -	$ 20,000	$ 800
	- Piping	1	1999	40	16	$ 40,000	$ 40,000	$ 16,000	$ 24,000	60%	$ -	$ -	$ 1,000
	- Electrical	1	1999	30	16	$ 10,000	$ 10,000	$ 5,333	$ 4,667	47%	$ -	$ 10,000	$ 333
River Intake 2	- Pump - 25 Hp	1	2012	10	3	$ 15,000	$ 15,000	$ 4,500	$ 10,500	70%	$ -	$ 30,000	$ 1,500
	- Screens	1	2004	25	11	$ 20,000	$ 20,000	$ 8,800	$ 11,200	56%	$ -	$ 20,000	$ 800
	- Piping	1	2004	40	11	$ 40,000	$ 40,000	$ 11,000	$ 29,000	73%	$ -	$ -	$ 1,000
	- Electrical	1	2004	30	11	$ 10,000	$ 10,000	$ 3,667	$ 6,333	63%	$ -	$ 10,000	$ 333
Infiltration Gallery	(Cost not entered as gallery will not be replaced)	1	1994	30	21	$ 1	$ 1	$ 1	$ 0	30%	$ -	$ 1	$ 0
River (Main) Pump Station	- Pump 1 - 200 Hp	1	1994	25	21	$ 40,000	$ 40,000	$ 33,600	$ 6,400	16%	$ -	$ 40,000	$ 1,600
	- Pump 2 - 200 Hp	1	1994	25	21	$ 40,000	$ 40,000	$ 33,600	$ 6,400	16%	$ -	$ 40,000	$ 1,600
	- Building and Piping	1	1994	50	21	$ 300,000	$ 300,000	$ 126,000	$ 174,000	58%	$ -	$ -	$ 6,000
	- Wet Well	1	1994	50	21	$ 435,000	$ 435,000	$ 182,700	$ 252,300	58%	$ -	$ -	$ 8,700
	- Chlorine Gas System	1	1994	25	21	$ 25,000	$ 25,000	$ 21,000	$ 4,000	16%	$ -	$ 25,000	$ 1,000
	- Electrical, Controls & SCADA	1	1994	30	21	$ 100,000	$ 100,000	$ 70,000	$ 30,000	30%	$ -	$ 100,000	$ 3,333
Total							**$ 1,110,001**	**$ 535,001**	**$ 575,000**	**52%**	**$ -**	**$ 325,001**	**$ 29,500**
Reservoirs													
Zone 1	- Reservoir Structure complete with Piping Systems	1	1981	79	34	$ 1,215,000	$ 1,215,000	$ 522,911	$ 692,089	57%	$ -	$ -	$ 15,380
concrete	- Electrical, Controls and SCADA	1	1981	35	34	$ 40,000	$ 40,000	$ 38,857	$ 1,143	3%	$ -	$ 40,000	$ 1,143
	- Site Works and Fencing	1	1981	50	34	$ 30,000	$ 30,000	$ 20,400	$ 9,600	32%	$ -	$ 30,000	$ 600
Zone 2 (Mesa Vista)	- Reservoir Structure complete with Piping Systems	1	1981	81	34	$ 1,023,750	$ 1,023,750	$ 429,722	$ 594,028	58%	$ -	$ -	$ 12,639
concrete	- Electrical, Controls and SCADA	1	1981	35	34	$ 40,000	$ 40,000	$ 38,857	$ 1,143	3%	$ -	$ 40,000	$ 1,143
	- Site Works and Fencing	1	1981	50	34	$ 30,000	$ 30,000	$ 20,400	$ 9,600	32%	$ -	$ 30,000	$ 600
Old Zone 2 (Mesa Vista)	- Reservoir Structure complete with Piping Systems	1	1970	80	45	$ 1	$ 1	$ 1	$ 0	44%	$ -	$ -	$ 0
concrete - abandonned	- Electrical, Controls and SCADA	1	1970	35	45	$ 1	$ 1	$ 1	$ -	0%	$ 1	$ 1	$ 0
no cost as not to be replaced	- Site Works and Fencing	1	1970	50	45	$ 1	$ 1	$ 1	$ 0	10%	$ -	$ 1	$ 0
Upper Mesa Vista Balancing Tank	- Reservoir Structure complete with Piping Systems	1	1970	80	45	$ 350,000	$ 350,000	$ 196,875	$ 153,125	44%	$ -	$ -	$ 4,375
	- Electrical, Controls and SCADA	1	1970	35	45	$ 20,000	$ 20,000	$ 20,000	$ -	0%	$ 20,000	$ 20,000	$ 571
	- Site Works and Fencing	1	1970	50	45	$ 15,000	$ 15,000	$ 13,500	$ 1,500	10%	$ -	$ 15,000	$ 300
Zone 3 (North Ashcroft)	- Reservoir Structure complete with Piping Systems	1	1981	80	34	$ 855,000	$ 855,000	$ 363,375	$ 491,625	58%	$ -	$ -	$ 10,688
concrete	- Electrical, Controls and SCADA	1	1981	35	34	$ 40,000	$ 40,000	$ 38,857	$ 1,143	3%	$ -	$ 40,000	$ 1,143
	- Site Works and Fencing	1	1981	50	34	$ 30,000	$ 30,000	$ 20,400	$ 9,600	32%	$ -	$ 30,000	$ 600
Old Zone 3 (North Ashcroft)	- Reservoir Structure complete with Piping Systems	1	1967	80	48	$ 1	$ 1	$ 1	$ 0	40%	$ -	$ -	$ 0
steel - seasonal use	- Electrical, Controls and SCADA	1	1967	35	48	$ 1	$ 1	$ 1	$ -	0%	$ 1	$ 1	$ 0
no cost as not to be replaced	- Site Works and Fencing	1	1967	50	48	$ 1	$ 1	$ 1	$ 0	4%	$ -	$ 1	$ 0
Total							**$ 3,688,756**	**$ 1,724,160**	**$ 1,964,596**	**53%**	**$ 20,002**	**$ 245,004**	**$ 49,181**
Booster Stations													
Mesa Vista	- Pump 1	1	1985	25	30	$ 15,000	$ 15,000	$ 15,000	$ -	0%	$ 15,000	$ 15,000	$ 600
	- Pump 2	1	2011	25	4	$ 15,000	$ 15,000	$ 2,400	$ 12,600	84%	$ -	$ -	$ 600
	- Pump 3 (backup)	1	1985	35	30	$ 10,000	$ 10,000	$ 8,571	$ 1,429	14%	$ -	$ 10,000	$ 286
	- Mechanical and Controls	1	1970	35	45	$ 60,000	$ 60,000	$ 60,000	$ -	0%	$ 60,000	$ 60,000	$ 1,714
	- Building	1	1970	60	45	$ 100,000	$ 100,000	$ 75,000	$ 25,000	25%	$ -	$ 100,000	$ 1,667
Upper Mesa Vista	- Pumps	1	1980	25	35	$ 15,000	$ 15,000	$ 15,000	$ -	0%	$ 15,000	$ 15,000	$ 600
	- Mechanical and Controls	1	1980	35	35	$ 35,000	$ 35,000	$ 35,000	$ -	0%	$ 35,000	$ 35,000	$ 1,000
	- Building/Chamber	1	1980	60	35	$ 25,000	$ 25,000	$ 14,583	$ 10,417	42%	$ -	$ -	$ 417
North Ashcroft	- Pump - 50 HP	1	2002	25	13	$ 20,000	$ 20,000	$ 10,400	$ 9,600	48%	$ -	$ 20,000	$ 800
	- Pump - 50 HP	1	2014	25	1	$ 20,000	$ 20,000	$ 800	$ 19,200	96%	$ -	$ -	$ 800
	- Pump - 25 HP	1	1980	25	35	$ 15,000	$ 15,000	$ 15,000	$ -	0%	$ 15,000	$ 15,000	$ 600
	- Wet Well	1	1962	80	53	$ 300,000	$ 300,000	$ 198,750	$ 101,250	34%	$ -	$ -	$ 3,750
	- Mechanical and Controls	1	1962	35	53	$ 50,000	$ 50,000	$ 50,000	$ -	0%	$ 50,000	$ 50,000	$ 1,429
	- Building	1	1962	60	53	$ 60,000	$ 60,000	$ 53,000	$ 7,000	12%	$ -	$ 60,000	$ 1,000
Total							**$ 740,000**	**$ 553,505**	**$ 186,495**	**25%**	**$ 190,000**	**$ 380,000**	**$ 15,262**
Other Facilities													
PRV - Mesa Vista to Zone 1	- Station	1	1985	50	30	$ 130,000	$ 130,000	$ 78,000	$ 52,000	40%	$ -	$ -	$ 2,600
	- PRV	1	1985	35	30	$ 30,000	$ 30,000	$ 25,714	$ 4,286	14%	$ -	$ 30,000	$ 857
PRV - North Ascroft to Zone 1	- Station	1	1981	50	34	$ 130,000	$ 130,000	$ 88,400	$ 41,600	32%	$ -	$ 130,000	$ 2,600
	- PRV	1	1981	35	34	$ 30,000	$ 30,000	$ 29,143	$ 857	3%	$ -	$ 30,000	$ 857
Total							**$ 320,000**	**$ 221,257**	**$ 98,743**	**31%**	**$ -**	**$ 190,000**	**$ 6,914**
Total Water							**$ 18,562,997**	**$ 11,299,546**	**$ 7,263,451**	**39%**	**$ 210,002**	**$ 5,724,865**	**$ 277,290**

VILLAGE OF ASHCROFT WATER SYSTEM IMPROVEMENTS							
Cost Estimate - Rapid Sand Filtration, UV Disinfection and Chlorination							
2014 Water Master Plan							
				System Capacity	125 L/s	100 L/s	
Item Description					***Costs***	***Costs***	***Comments/Assumptions***
1	**Conceptual Design**						
	0.1	Conceptual Design			$50,000	$50,000	
				Subtotal:	$50,000	$50,000	
2	**Pilot Testing and Predesign**						
	0.1	Water Quality Monitoring & Pilot Testing			$50,000	$50,000	piloting desirable, but may not be necessary
	0.2	Geotechnical Investigation			$15,000	$15,000	
	0.3	Surveying			$10,000	$10,000	
	0.4	Predesign			$150,000	$150,000	
	0.5	Environmental/Approvals			$35,000	$35,000	
				Subtotal:	$260,000	$260,000	
3	**DETAILED DESIGN**						
	0.1	Detailed Design & Tendering			$600,000	$600,000	
				Subtotal:	$600,000	$600,000	
4	**CONSTRUCTION**						
	0.1	General Requirements			$200,000	$185,000	assumed at approximately 2.5%
			Insurance & Bonding				
			Survey & Layout				
			Mobilization & Demobilization				
			Commissioning				
	0.2	General Sitework			$500,000	$460,000	
			Access Road				
			Dewatering				
			Site preparation				
			Landscaping & lighting, fencing				
	0.3	Site Piping			$250,000	$250,000	
	0.4	River Intake Pumps (2)			$50,000	$40,000	replace for higher head loss - pump to filters
	0.5	Building			$1,250,000	$1,090,000	350 and 300 m² respectively
			Excavation & Backfill				
			Structural				
			Clearwell				
			Office/Laboratory				
			HVAC				
	0.6	Rapid Sand Filtration					
			Chemical Feed System & Storage				
			Flocculation Equipment				
			Piping & Valving				
			Filters (includes tanks, media)		$1,400,000	$1,220,000	including ss tanks, chemical feed, controls, blowers, flocculators, media
	0.7	Water Quality Monitoring Equipment			$50,000	$50,000	
	0.8	Process Piping & Valving			$300,000	$300,000	
	0.9	Chlorination System			$50,000	$50,000	assume sodium hypochlorite, eyewash/shower
	0.10	UV Disinfection			$250,000	$220,000	quote for medium pressure UV
	0.11	Electrical & Controls, SCADA					per estimate from ICI
			Low & high lift pump controls		$130,000	$130,000	
			Treatment System controls/instruments		$310,000	$270,000	
			Electrical Service		$150,000	$150,000	
			Main Control Systems		$100,000	$100,000	
			SCADA system		$90,000	$90,000	
			Instrument Air System		$80,000	$80,000	
			General Overhead		$40,000	$40,000	
				subtotal electrical	*$900,000*	*$860,000*	
	0.12	Standby Power			$160,000	$160,000	per ICI estimate
	0.13	Retrofit existing River Pump station			$200,000	$200,000	would need to isolate raw/treated water, could have UV/chlorination at this location
	0.14	Solids Handling			$200,000	$170,000	
	0.15	Uni-directional flushing of distribution system			$60,000	$60,000	
	0.16	Engineering - Construction & Post Construction			$350,000	$325,000	
				Subtotal:	$6,170,000	$5,640,000	
				Contingency on Construction Costs (20%):	$1,234,000	$1,128,000	
				PST (5%)	$308,500	$282,000	
				Construction Subtotal	**$7,712,500**	**$7,050,000**	
				TOTAL FOR ALL ABOVE COSTS (rounded)	**$8,620,000**	**$7,960,000**	

Notes:

1) Water treatment plant sized for 125 L/s (10.8 ML/d) or 100 L/s
2) Proposed treatment system includes direct filtration, UV disinfection and chlorination
3) Pilot Testing recommended to optimize treatment process selection (e.g. filter type and loading rate).
4) Process/system configuration and site plan to be reviewed during Conceptual Design.
   - there are several options for system configuration which mainly depend on:
      a) treatment plant location: existing River site or at zone 1 reservoir
      b) type of filters: pressure or gravity
   - this will affect the approach to pumping and controls:
      a) a treatment plant at the River site with gravity filters would require -- replacing the river intake pumps and pumping directly to the gravity filters. The
      b) a treatment plant at the Zone 1 reservoir with gravity filters would mean keeping the existing river intake/high lift pump configuration, but possibly
      c) pressure filters at the River site could mean using the river intake pumps to pump to the existing pump station, new low lift pumps to pump to WTP
5) For this cost estimate, assume WTP at River site with gravity filters
6) Estimate assumes that adequate land is available and does not need to be purchased
7) For this cost estimate assume:
   - River intake pumps will be replaced so that they can pump directly to the gravity filters
   - Existing River pump station to be retrofitted and used as clearwell/high lift pump station
   - Use existing high lift pumps
   - Clearwell for pumping only - not contact time as there is a dedicated main to the reservoir
4) Estimate in 2015 $ - does not include inflation. Does not include GST

Village of Ashcroft
O&M costs - for <u>increase</u> from current Village water system costs with addition of direct filtration and UV disinfection

Item No.	AVERAGE ANNUAL O & M COSTS Description	Based on 40 L/s ADD	Comments
1.0	**Chemical Systems**		assume no pH adjustment
	Sodium Hypochlorite	$ 10,000	assume increase from current - could be reduced chlorine demand with filtration, but converting to sodium hypochlorite and this will be more expensive
	Coagulant	$ 24,000	
	Coagulant Aid	$ 2,800	
	Sodium Hypochlorite Feed Pump	per current	assume current costs applicable - just calculating differential
	Coagulant Feed Pump	$ 400	
	Coagulant Aid Feed Pump	$ 400	
2.0	**Main Pumps**		
	Intake Pump Power	per current	assume current costs applicable - just calculating differential
	High-Lift Pump Power	per current	assume current costs applicable - just calculating differential
3.0	**UV Disinfection**	$ 18,000	includes power and lamp replacement
4.0	**Media Filtration**		
	Sand Media Replacement	$ 1,500	$15000, replacement frequencey every 10 years, annual O&M cost calculated by dividing the cost over 10 years
5.0	**Water quality testing, Recording, Monitoring**	per current	
6.0	**General Maintenance Labour**		
	Operator Full Time (incl. benefits)	$ 90,000	As per Michelle Allen May 30 - assume one new FTE. Does not include new truck
	Water quality monitoring/record keeping	per current	these are included in full-time operator's duties
	Routine equip. maintenance/calibration	per current	these are included in full-time operator's duties
	Periodic equipment maintenance	per current	these are included in full-time operator's duties
	Electrical system inspection and maintenance (By Contractor)	$ 5,000	
7.0	**Miscellaneous**		
	Telephone/Internet	$ 1,200	for new building
	Heating and General Building Electrical	$ 4,500	for new building
	Average Annual O&M Subtotal	**$ 157,800**	

Village of Ashcroft
Water Master Plan
20 Year Capital Plan (and Operations and Maintenance Increases Compared to Current Activities and Costs)

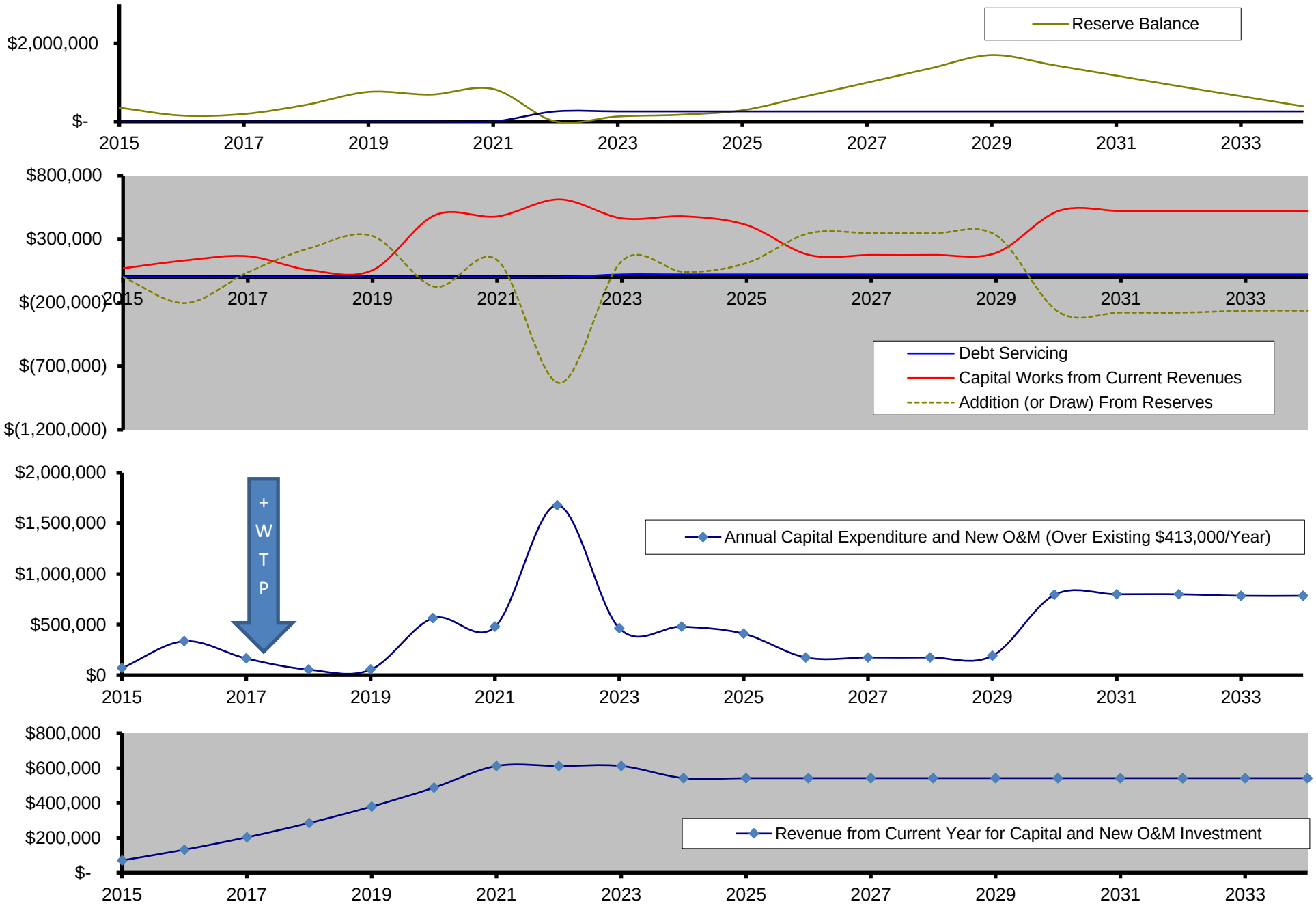
									Annual Capital Plan (Municipal Portion)																			
Title	**Year**	**Cost**	**Grants (%)**	**Grants (Amount)**	**Net Project Cost**	**Recover from Developers / LAS**	**Recoverable Amount**	**Municipal Contribution**	**2015**	**2016**	**2017**	**2018**	**2019**	**2020**	**2021**	**2022**	**2023**	**2024**	**2025**	**2026**	**2027**	**2028**	**2029**	**2030**	**2031**	**2032**	**2033**	**2034**
Capital Works																												
Zone 2 Reservoir Concrete Barriers and Signage (No Vehicle Traffic on Reservoir)	2015	$5,000	0%	$0	$5,000	0%	$0	$5,000	$5,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Zone 2 Reservoir and Zone 3 Pump Chamber Ladders	2016	$5,000	0%	$0	$5,000	0%	$0	$5,000	--	$5,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
New North Ashcroft PRV to Serve Zone 1 (Capital Project 2A - Confirm if 2B will not work first)	2016	$250,000	0%	$0	$250,000	0%	$0	$250,000	--	$250,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
2nd Cell - North Ashcroft (Zone 3)	2022	$1,200,000	0%	$0	$1,200,000	0%	$0	$1,200,000	--	--	--	--	--	--	--	$1,200,000	--	--	--	--	--	--	--	--	--	--	--	--
Mesa Vista Service Connection Backflow Preventers (Incl. Communications with Residents)	2016	$11,000	0%	$0	$11,000	0%	$0	$11,000	--	$11,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Mesa Vista Heights - Revising Fire Pumping Booster System	2020	$50,000	0%	$0	$50,000	0%	$0	$50,000	--	--	--	--	--	$50,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Mesa Vista 200 mm Dia. Main - Reservoir to Vista Heights	2025	$125,000	0%	$0	$125,000	0%	$0	$125,000	--	--	--	--	--	--	--	--	--	--	$125,000	--	--	--	--	--	--	--	--	--
Connect Supply Main (WTP to Zone 1 Reservoir) to Adjacent Main Along 1st Ave.	2020	$30,000	0%	$0	$30,000	0%	$0	$30,000	--	--	--	--	--	$30,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Treatment Plant (Conceptual Stage Engineering)	2015	$50,000	0%	$0	$50,000	0%	$0	$50,000	$50,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Treatment Plant (Pilot Testing and Predesign) - Excluded from Cash Flow Model	2017	$260,000	67%	$174,200	$85,800	0%	$0	$85,800	--	--	$85,800	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Treatment Plant (Detailed Design and Tendering) - Excluded from Cash Flow Model	2018	$600,000	67%	$402,000	$198,000	0%	$0	$198,000	--	--	--	$198,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Treatment Plant (Construction) - Excluded from Cash Flow Model	2019	$6,850,000	67%	$4,589,500	$2,260,500	0%	$0	$2,260,500	--	--	--	--	$2,260,500	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Treatment Plant (Possible Emergency Power Generator) - Excl from Cash Flow Model	2019	$200,000	67%	$134,000	$66,000	0%	$0	$66,000	--	--	--	--	$66,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Metering - Zone Meters	2016	$30,000	0%	$0	$30,000	0%	$0	$30,000	--	$30,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Metering - Industrial, Commercial, Institutional Investigation	2016	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Metering - Industrial, Commercial, Institutional	2017	$150,000	0%	$0	$150,000	0%	$0	$150,000	--	--	$150,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Metering - Residential Meters - Review of Water Use + Benefit/Costs Analysis	2019	$20,000	0%	$0	$20,000	0%	$0	$20,000	--	--	--	--	$20,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Metering - Residential Meters - Capital Cost Not Included. First Determine if Benefit/Cost Analysis Proves Out																												
Replace Zone 3 PRV - Use Existing Station	2018	$40,000	0%	$0	$40,000	0%	$0	$40,000	--	--	--	$40,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
SCADA Upgrades/Replacement	2025	$100,000	0%	$0	$100,000	0%	$0	$100,000	--	--	--	--	--	--	--	--	--	--	$100,000	--	--	--	--	--	--	--	--	--
Replace Intake Pump 1	2021	$15,000	0%	$0	$15,000	0%	$0	$15,000	--	--	--	--	--	--	$15,000	--	--	--	--	--	--	--	--	--	--	--	--	--
Replace Intake Pump 1	2031	$15,000	0%	$0	$15,000	0%	$0	$15,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$15,000	--	--	--
Replace Intake Screen 1	2024	$6,000	0%	$0	$6,000	0%	$0	$6,000	--	--	--	--	--	--	--	--	--	$6,000	--	--	--	--	--	--	--	--	--	--
Replace Intake Pump 2	2022	$15,000	0%	$0	$15,000	0%	$0	$15,000	--	--	--	--	--	--	--	$15,000	--	--	--	--	--	--	--	--	--	--	--	--
Replace Intake Pump 2	2032	$15,000	0%	$0	$15,000	0%	$0	$15,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$15,000	--	--
Replace Intake Screen 2	2029	$6,000	0%	$0	$6,000	0%	$0	$6,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$6,000	--	--	--	--	--
Replace Mesa Vista PRV - Use Existing Station	2035	$40,000	0%	$0	$40,000	0%	$0	$40,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Hydrant Replacements	2024-'28	$50,000	0%	$0	$50,000	0%	$0	$50,000										$10,000	$10,000	$10,000	$10,000	$10,000						
Other Miscellaneous General Mechanical Capital Works (Upgrades and Replacements) - Already Included in Existing Operations and Maintenance Budget																												
Valve Replacements and Leak Repairs - Already Included in Existing Operations and Maintenance Budget																												
Water Main Replacement (Taken from Reinvestment Model)	2020	$298,000	0%	$0	$298,000	0%	$0	$298,000	--	--	--	--	--	$298,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2021	$298,000	0%	$0	$298,000	0%	$0	$298,000	--	--	--	--	--	--	$298,000	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2022	$298,000	0%	$0	$298,000	0%	$0	$298,000	--	--	--	--	--	--	--	$298,000	--	--	--	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2023	$298,000	0%	$0	$298,000	0%	$0	$298,000	--	--	--	--	--	--	--	--	$298,000	--	--	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2024	$298,000	0%	$0	$298,000	0%	$0	$298,000	--	--	--	--	--	--	--	--	--	$298,000	--	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2025	$0	0%	$0	$0	0%	$0	$0	--	--	--	--	--	--	--	--	--	--	$0	--	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2026	$0	0%	$0	$0	0%	$0	$0	--	--	--	--	--	--	--	--	--	--	--	$0	--	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2027	$0	0%	$0	$0	0%	$0	$0	--	--	--	--	--	--	--	--	--	--	--	--	$0	--	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2028	$0	0%	$0	$0	0%	$0	$0	--	--	--	--	--	--	--	--	--	--	--	--	--	$0	--	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2029	$0	0%	$0	$0	0%	$0	$0	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$0	--	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2030	$619,000	0%	$0	$619,000	0%	$0	$619,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$619,000	--	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2031	$619,000	0%	$0	$619,000	0%	$0	$619,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$619,000	--	--	--
Water Main Replacement (Taken from Reinvestment Model)	2032	$619,000	0%	$0	$619,000	0%	$0	$619,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$619,000	--	--
Water Main Replacement (Taken from Reinvestment Model)	2033	$619,000	0%	$0	$619,000	0%	$0	$619,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$619,000	--
Water Main Replacement (Taken from Reinvestment Model)	2034	$619,000	0%	$0	$619,000	0%	$0	$619,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$619,000
Water Capital Total		**$14,733,000**		**$5,299,700**	**$9,433,300**		**$0**	**$9,433,300**	**$55,000**	**$306,000**	**$235,800**	**$238,000**	**$2,346,500**	**$378,000**	**$313,000**	**$1,513,000**	**$298,000**	**$314,000**	**$235,000**	**$10,000**	**$10,000**	**$10,000**	**$6,000**	**$619,000**	**$634,000**	**$634,000**	**$619,000**	**$619,000**
Water - Operations																												
Annual Operations (Current Dollars - No Inflation) - $413,000/year																												
Water Conservation (Cost to right rielates to $5,00/year from 2015 to 2034)	ongoing	$100,000	0%	$0	$100,000	0%	$0	$100,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000
Leak Detection - Phase 1	2015	$10,000	0%	$0	$10,000	0%	$0	$10,000	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Leak Detection - Phase 2	2020	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	--	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Fire Flow and Field Investigations Regarding Optimizing North Ashcroft Pressure Zones	2016	$15,000	0%	$0	$15,000	0%	$0	$15,000	--	$15,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Cross Connection Control Program - Phase 1	2016	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Cross Connection Control Program - Phase 2	2017	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Cross Connection Control Program - Phase 3	2018	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Master Plan - 20 Year Capital Plan Review and Cash Flow Update	2020	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	--	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Water Master Plan - 20 Year Capital Plan Review and Cash Flow Update	2025	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	--	--	--	--	--	--	--	$10,000	--	--	--	--	--	--	--	--	--
Water Master Plan - 20 Year Capital Plan Review and Cash Flow Update	2030	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$10,000	--	--	--	--
Pipe Condition Assessments	2019	$20,000	0%	$0	$20,000	0%	$0	$20,000	--	--	--	--	$20,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Pipe Condition Assessments	2029	$20,000	0%	$0	$20,000	0%	$0	$20,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	$20,000	--	--	--	--	--
Treatment Plant - Operator Training	2019	$10,000	0%	$0	$10,000	0%	$0	$10,000	--	--	--	--	$10,000	--	--	--	--	--	--	--	--	--	--	--	--	--	--	--
Treatment Plant O&M (Starting in 2020. Cost to right is for 2020 to 2034)	after WTF	$2,400,000	0%	$0	$2,400,000	0%	$0	$2,400,000						$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000	$160,000
Water Operations - NEW, In Addition to Exisitng $413,000/Year Total		**$2,645,000**		**$0**	**$2,645,000**		**$0**	**$2,645,000**	**$15,000**	**$30,000**	**$15,000**	**$15,000**	**$35,000**	**$185,000**	**$165,000**	**$165,000**	**$165,000**	**$165,000**	**$175,000**	**$165,000**	**$165,000**	**$165,000**	**$185,000**	**$175,000**	**$165,000**	**$165,000**	**$165,000**	**$165,000**
Water Total (Capital and New Operations)		**$17,378,000**		**$5,299,700**	**$12,078,300**		**$0**	**$12,078,300**	**$70,000**	**$336,000**	**$250,800**	**$253,000**	**$2,381,500**	**$563,000**	**$478,000**	**$1,678,000**	**$463,000**	**$479,000**	**$410,000**	**$175,000**	**$175,000**	**$175,000**	**$191,000**	**$794,000**	**$799,000**	**$799,000**	**$784,000**	**$784,000**

Water Capital Plan - 20 Year Budgetary Cash Flow Projection - Capital and O&M

DOES NOT INCLUDE WATER TREATMENT PLANT LOAN AS THAT IS CALCULATED SEPARATELY

	Year	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034
FINANCIAL SUMMARY																					
Debt Servicing		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529
Debt Load (Serviced by 20 yr Debt)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579	$ 257,579
Capital Works from Current Revenues		$ 70,000	$ 131,950	$ 165,000	$ 55,000	$ 55,000	$ 487,691	$ 478,000	$ 612,294	$ 463,000	$ 479,000	$ 410,000	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765
Addition (or Draw) From Reserves		$ -	$ (204,050)	$ 38,192	$ 230,121	$ 324,340	$ (75,309)	$ 134,294	$ (832,303)	$ 127,765	$ 41,765	$ 110,765	$ 345,765	$ 345,765	$ 345,765	$ 329,765	$ (273,235)	$ (278,235)	$ (278,235)	$ (263,235)	$ (263,235)
Reserve Balance		$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 689,769	$ 832,303	$ -	$ 129,043	$ 172,516	$ 286,114	$ 638,198	$ 993,803	$ 1,352,964	$ 1,699,557	$ 1,440,585	$ 1,173,974	$ 904,696	$ 647,876	$ 388,488
Proposed Capital and New O&M Schedule					Excluded WTP - Loan Calculated Separately																
Expenditures - NOT INFLATED - From Capital and O&M Plan - Excludes 2018 WTP Costs as Covered by U		$ 70,000	$ 336,000	$ 165,000	$ 55,000	$ 55,000	$ 563,000	$ 478,000	$ 1,678,000	$ 463,000	$ 479,000	$ 410,000	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 794,000	$ 799,000	$ 799,000	$ 784,000	$ 784,000
Input Assumed Annual Inflation Rate			0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%
Used only to simplify edits (year it's assumed inflation rate starts to be consistent)								1	2	3	4	5	6	7	8	9	10	11	12	13	14
Cumulative Inflation			0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Expenditures - INFLATED		$ 70,000	$ 336,000	$ 165,000	$ 55,000	$ 55,000	$ 563,000	$ 478,000	$ 1,678,000	$ 463,000	$ 479,000	$ 410,000	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 794,000	$ 799,000	$ 799,000	$ 784,000	$ 784,000
Assumed Senior Government Grants (as a Percentage of Inflated Expenditures)	0%	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Total Capital and New O&M Requirement (Subtracted Grants and No WTP Loan)		**$ 70,000**	**$ 336,000**	**$ 165,000**	**$ 55,000**	**$ 55,000**	**$ 563,000**	**$ 478,000**	**$ 1,678,000**	**$ 463,000**	**$ 479,000**	**$ 410,000**	**$ 175,000**	**$ 175,000**	**$ 175,000**	**$ 191,000**	**$ 794,000**	**$ 799,000**	**$ 799,000**	**$ 784,000**	**$ 784,000**
Total Revenue from Current Year Revenue for that Year (after rate increase)=	2014 $ 413,000	**$ 413,000**	**$ 474,950**	**$ 546,193**	**$ 628,121**	**$ 722,340**	**$ 830,691**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**	**$ 955,294**
Contribution from Non-Conditional Gas Tax Fund (Assume Program Continues)		$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000											
Annual Tax Increase for Current Year (i.e. What is in the Newspaper at Beginning of that Year)		0.00%	15.00%	15.00%	15.00%	15.00%	15.00%	15.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Revenue Increave Over Current Year for Capital and New O&M Investment		**$ 70,000**	**$ 131,950**	**$ 203,193**	**$ 285,121**	**$ 379,340**	**$ 487,691**	**$ 612,294**	**$ 612,294**	**$ 612,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**	**$ 542,294**
Service Existing Debt First																					
Existing Annual Debt Payment (Before Additional Debt Added From this Year's Work)			$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529
Current Year's Funds Remaining After Existing Debt Payment is Made		$ 70,000	$ 131,950	$ 203,193	$ 285,121	$ 379,340	$ 487,691	$ 612,294	$ 612,294	$ 590,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765	$ 520,765
Funding Shortfall for Current Year's Work Before Drawing From Reserves		$ -	$ 204,050	$ -	$ -	$ -	$ 75,309	$ -	$ 1,065,706	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 273,235	$ 278,235	$ 278,235	$ 263,235	$ 263,235
Draw From Reserves (If Reserves Are Available)		$ -	$ (204,050)	$ -	$ -	$ -	$ (75,309)	$ -	$ (832,303)	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ (273,235)	$ (278,235)	$ (278,235)	$ (263,235)	$ (263,235)
Funding Shortfall After Drawing From Reserves (i.e. New Debt)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 233,402	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Required New Debt This Year =(Funding Shortfall + 1st yr's interest +a hint more)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 257,579	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt with Annual Interest =	5%	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 21,529	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
20 yr Debt Servicing Factor (Interest + S/F Factor Related as per MFA 20 Yr Debt)	0.08358																				
Loan Number (Maximum in Spreadsheet is 25 Different Debts - More Will Result in Math Errors)									1												
Reinvestment Reserve Funds																					
Reserve Balance at Start of Year (None from reserve but $351k from Community Works Fund reserve)		**$ 351,000**	$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 689,769	$ 832,303	$ -	$ 129,043	$ 172,516	$ 286,114	$ 638,198	$ 993,803	$ 1,352,964	$ 1,699,557	$ 1,440,585	$ 1,173,974	$ 904,696	$ 647,876
Additions (if money left over after capital from current year and debt repayment)		$ -	$ -	$ 38,192	$ 230,121	$ 324,340	$ -	$ 134,294	$ -	$ 127,765	$ 41,765	$ 110,765	$ 345,765	$ 345,765	$ 345,765	$ 329,765	$ -	$ -	$ -	$ -	$ -
Expenditures From Reserve (Assume End of Year)		$ -	$ (204,050)	$ -	$ -	$ -	$ (75,309)	$ -	$ (832,303)	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ (273,235)	$ (278,235)	$ (278,235)	$ (263,235)	$ (263,235)
Assumed Interest Earned On Invested Funds	1%	$ 3,510	$ 1,505	$ 1,902	$ 4,222	$ 7,507	$ 6,829	$ 8,241	$ -	$ 1,278	$ 1,708	$ 2,833	$ 6,319	$ 9,840	$ 13,396	$ 16,827	$ 14,263	$ 11,624	$ 8,957	$ 6,415	$ 3,846
Total Reserve Funds - Year End		$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 689,769	$ 832,303	$ -	$ 129,043	$ 172,516	$ 286,114	$ 638,198	$ 993,803	$ 1,352,964	$ 1,699,557	$ 1,440,585	$ 1,173,974	$ 904,696	$ 647,876	$ 388,488
Warning(s)																					
Warning - Annual Payment is Greater than Available Funding in that Year - Occurs at least Once																					
Debt Schedule																					
Annual Payment for New Capital Debt For Loan Number	1	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529	$ 21,529
Annual Payment for New Capital Debt For Loan Number	2		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	3		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	4		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	5		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	6		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	7		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	8		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	9		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	10		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	11		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	12		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	13		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	14		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	15		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	16		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	17		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	18		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	19		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	20		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	21		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	22		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	23		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	24		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	25		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -

Village of Ashcroft Cash Flow Analysis - Water Fund
Excludes Water Treatment Plant Loan



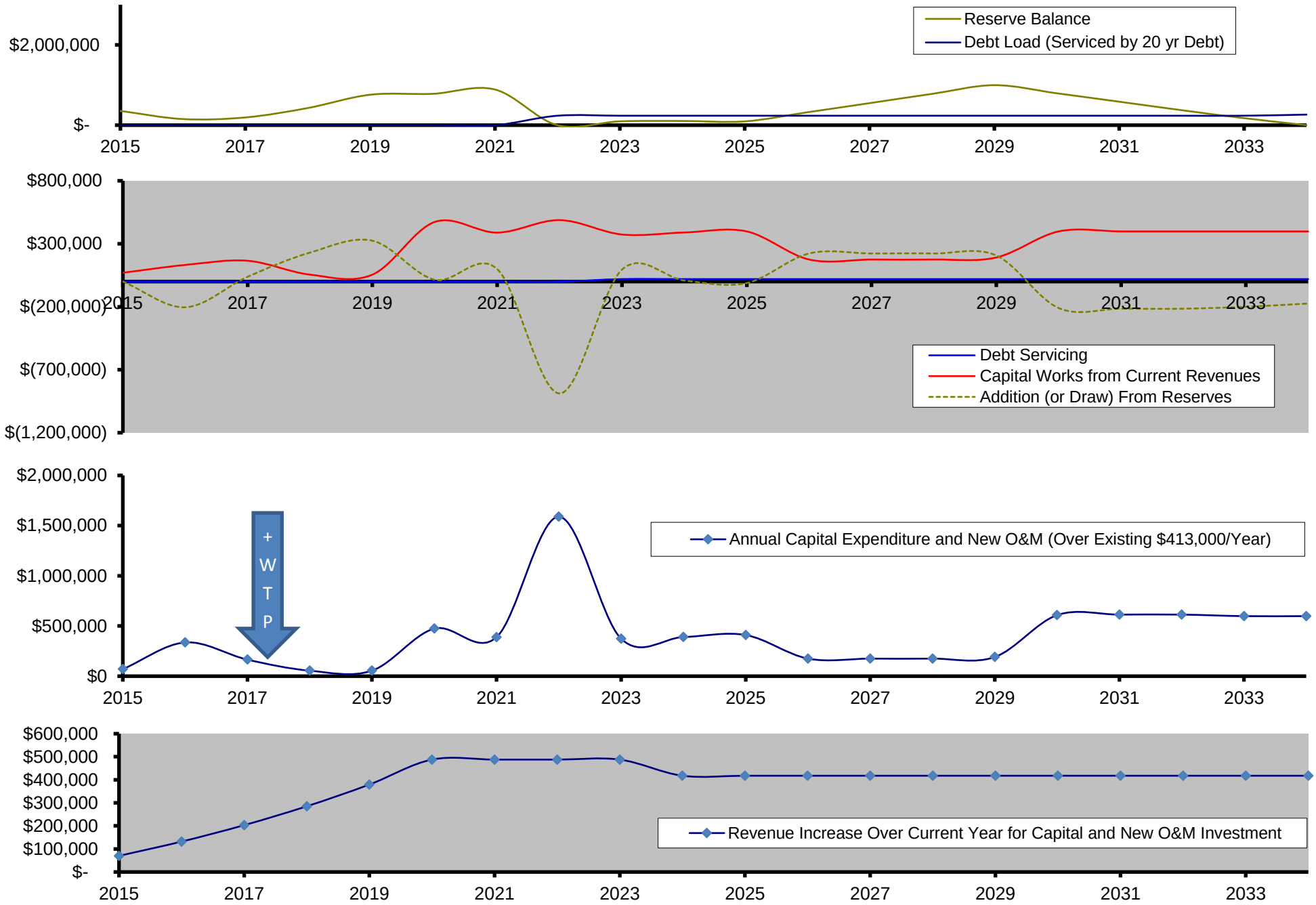
Water Capital Plan - 20 Year Budgetary Cash Flow Projection - Capital and O&M - REDUCE COST OF PIPE REPLACEMENT BY 1/3

DOES NOT INCLUDE WATER TREATMENT PLANT LOAN AS THAT IS CALCULATED SEPARATELY

	Year	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034
FINANCIAL SUMMARY																					
Debt Servicing		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644
Debt Load (Serviced by 20 yr Debt)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 235,024	$ 263,700
Capital Works from Current Revenues		$ 70,000	$ 131,950	$ 165,000	$ 55,000	$ 55,000	$ 473,600	$ 388,600	$ 487,691	$ 373,600	$ 389,600	$ 398,047	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047
Addition (or Draw) From Reserves		$ -	$ (204,050)	$ 38,192	$ 230,121	$ 324,340	$ 14,091	$ 99,091	$ (887,945)	$ 94,447	$ 8,447	$ (11,953)	$ 223,047	$ 223,047	$ 223,047	$ 207,047	$ (210,253)	$ (215,253)	$ (215,253)	$ (200,253)	$ (174,268)
Reserve Balance		$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 780,063	$ 887,945	$ -	$ 95,391	$ 104,876	$ 93,853	$ 320,068	$ 548,546	$ 779,309	$ 996,219	$ 793,826	$ 584,358	$ 372,796	$ 174,268	$ -
Proposed Capital and New O&M Schedule	**Excluded WTP - Loan Calculated Separately**				**Pipe Discount =**	**0.3**	**Pipe Discount relates to reducing cost of water main replacements, as a sensitivity analysis, to consider the impact of reducing the amount of work and/or reducing the costs (e.g. less road repairs). Using the discount does not refl**														
Expenditures - NOT INFLATED - From Capital and O&M Plan - Excludes 2018 WTP Costs as Covered by L		$ 70,000	$ 336,000	$ 165,000	$ 55,000	$ 55,000	$ 473,600	$ 388,600	$ 1,588,600	$ 373,600	$ 389,600	$ 410,000	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 608,300	$ 613,300	$ 613,300	$ 598,300	$ 598,300
Input Assumed Annual Inflation Rate			0%	0%	0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Used only to simplify edits (year it's assumed inflation rate starts to be consistent)								1	2	3	4	5	6	7	8	9	10	11	12	13	14
Cumulative Inflation			0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
Expenditures - INFLATED		$ 70,000	$ 336,000	$ 165,000	$ 55,000	$ 55,000	$ 473,600	$ 388,600	$ 1,588,600	$ 373,600	$ 389,600	$ 410,000	$ 175,000	$ 175,000	$ 175,000	$ 191,000	$ 608,300	$ 613,300	$ 613,300	$ 598,300	$ 598,300
Assumed Senior Government Grants (as a Percentage of Inflated Expenditures)	0%	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Total Capital and New O&M Requirement (Subtracted Grants and No WTP Loan)		**$ 70,000**	**$ 336,000**	**$ 165,000**	**$ 55,000**	**$ 55,000**	**$ 473,600**	**$ 388,600**	**$ 1,588,600**	**$ 373,600**	**$ 389,600**	**$ 410,000**	**$ 175,000**	**$ 175,000**	**$ 175,000**	**$ 191,000**	**$ 608,300**	**$ 613,300**	**$ 613,300**	**$ 598,300**	**$ 598,300**
Total Revenue from Current Year Revenue for that Year (after rate increase)=	2014 $ 413,000	**$ 413,000**	**$ 474,950**	**$ 546,193**	**$ 628,121**	**$ 722,340**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**	**$ 830,691**
Contribution from Non-Conditional Gas Tax Fund (Assume Program Continues)		$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000											
Annual Tax Increase for Current Year (i.e. What is in the Newspaper at Beginning of that Year)			15.00%	15.00%	15.00%	15.00%	15.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Revenue Increase Over Current Year for Capital and New O&M Investment		**$ 70,000**	**$ 131,950**	**$ 203,193**	**$ 285,121**	**$ 379,340**	**$ 487,691**	**$ 487,691**	**$ 487,691**	**$ 487,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**	**$ 417,691**
Service Existing Debt First																					
Existing Annual Debt Payment (Before Additional Debt Added From this Year's Work)			$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644
Current Year's Funds Remaining After Existing Debt Payment is Made		$ 70,000	$ 131,950	$ 203,193	$ 285,121	$ 379,340	$ 487,691	$ 487,691	$ 487,691	$ 468,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047	$ 398,047
Funding Shortfall for Current Year's Work Before Drawing From Reserves		$ -	$ 204,050	$ -	$ -	$ -	$ -	$ -	$ 1,100,909	$ -	$ -	$ 11,953	$ -	$ -	$ -	$ -	$ 210,253	$ 215,253	$ 215,253	$ 200,253	$ 200,253
Draw From Reserves (If Reserves Are Available)		$ -	$ (204,050)	$ -	$ -	$ -	$ -	$ -	$ (887,945)	$ -	$ -	$ (11,953)	$ -	$ -	$ -	$ -	$ (210,253)	$ (215,253)	$ (215,253)	$ (200,253)	$ (174,268)
Funding Shortfall After Drawing From Reserves (i.e. New Debt)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 212,965	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 25,985
Required New Debt This Year =(Funding Shortfall + 1st yr's interest +a hint more)		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 235,024	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,676
Annual Payment for New Capital Debt with Annual Interest =	5%	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 19,644	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,397
20 yr Debt Servicing Factor (Interest + S/F Factor Related as per MFA 20 Yr Debt)	0.08358																				
Loan Number (Maximum in Spreadsheet is 25 Different Debts - More Will Result in Math Errors)									1												2
Reinvestment Reserve Funds																					
Reserve Balance at Start of Year (None from reserve but $351k from Community Works Fund reserve)		**$ 351,000**	$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 780,063	$ 887,945	$ -	$ 95,391	$ 104,876	$ 93,853	$ 320,068	$ 548,546	$ 779,309	$ 996,219	$ 793,826	$ 584,358	$ 372,796	$ 174,268
Additions (if money left over after capital from current year and debt repayment)		$ -	$ -	$ 38,192	$ 230,121	$ 324,340	$ 14,091	$ 99,091	$ -	$ 94,447	$ 8,447	$ -	$ 223,047	$ 223,047	$ 223,047	$ 207,047	$ -	$ -	$ -	$ -	$ -
Expenditures From Reserve (Assume End of Year)		$ -	$ (204,050)	$ -	$ -	$ -	$ -	$ -	$ (887,945)	$ -	$ -	$ (11,953)	$ -	$ -	$ -	$ -	$ (210,253)	$ (215,253)	$ (215,253)	$ (200,253)	$ (174,268)
Assumed Interest Earned On Invested Funds	1%	$ 3,510	$ 1,505	$ 1,902	$ 4,222	$ 7,507	$ 7,723	$ 8,792	$ -	$ 944	$ 1,038	$ 929	$ 3,169	$ 5,431	$ 7,716	$ 9,864	$ 7,860	$ 5,786	$ 3,691	$ 1,725	$ -
Total Reserve Funds - Year End		$ 354,510	$ 151,965	$ 192,059	$ 426,402	$ 758,249	$ 780,063	$ 887,945	$ -	$ 95,391	$ 104,876	$ 93,853	$ 320,068	$ 548,546	$ 779,309	$ 996,219	$ 793,826	$ 584,358	$ 372,796	$ 174,268	$ -
Warning(s)																					
Warning - Annual Payment is Greater than Available Funding in that Year - Occurs at least Once																					
Debt Schedule																					
Annual Payment for New Capital Debt For Loan Number	1	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644	$ 19,644
Annual Payment for New Capital Debt For Loan Number	2		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 2,397
Annual Payment for New Capital Debt For Loan Number	3		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	4		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	5		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	6		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	7		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	8		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	9		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	10		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	11		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	12		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	13		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	14		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	15		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	16		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	17		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	18		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	19		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	20		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	21		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	22		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	23		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	24		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Annual Payment for New Capital Debt For Loan Number	25		$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -

Village of Ashcroft Cash Flow Analysis - Water Fund - Apply 1/3 of Pipe Replacement Costs

Excludes Water Treatment Plant Loan & Pipe Cost Reduced to Reflect More Risk by Assuming Pipe Costs will be Less



20 Year Term

Cost for Treatment Plant Loan - 1/3 of Capital Cost

Principal: 2,610,300.00 Interest Rate: **5.00%**

S/F Factor: *0.03358175*

	Principal Pymnt	Interest Pymnt	Total Pymnt	Actuarial	Reducing Balance
					2,610,300.00
Yr 1 Semi Annual		65,257.50	65,257.50		2,610,300.00
Yr 1 Annual	87,658.44	65,257.50	152,915.94		2,522,641.56
Yr 2 Semi Annual		65,257.50	65,257.50		2,522,641.56
Yr 2 Annual	87,658.44	65,257.50	152,915.94	3,506.34	2,431,476.78
Yr 3 Semi Annual		65,257.50	65,257.50		2,431,476.78
Yr 3 Annual	87,658.44	65,257.50	152,915.94	7,152.93	2,336,665.40
Yr 4 Semi Annual		65,257.50	65,257.50		2,336,665.40
Yr 4 Annual	87,658.44	65,257.50	152,915.94	10,945.38	2,238,061.58
Yr 5 Semi Annual		65,257.50	65,257.50		2,238,061.58
Yr 5 Annual	87,658.44	65,257.50	152,915.94	14,889.54	2,135,513.60
Yr 6 Semi Annual		65,257.50	65,257.50		2,135,513.60
Yr 6 Annual	87,658.44	65,257.50	152,915.94	18,991.46	2,028,863.70
Yr 7 Semi Annual		65,257.50	65,257.50		2,028,863.70
Yr 7 Annual	87,658.44	65,257.50	152,915.94	23,257.45	1,917,947.80
Yr 8 Semi Annual		65,257.50	65,257.50		1,917,947.80
Yr 8 Annual	87,658.44	65,257.50	152,915.94	27,694.09	1,802,595.27
Yr 9 Semi Annual		65,257.50	65,257.50		1,802,595.27
Yr 9 Annual	87,658.44	65,257.50	152,915.94	32,308.19	1,682,628.64
Yr 10 Semi Annual		65,257.50	65,257.50		1,682,628.64
Yr 10 Annual	87,658.44	65,257.50	152,915.94	37,106.85	1,557,863.34
Yr 11 Semi Annual		65,257.50	65,257.50		1,557,863.34
Yr 11 Annual	87,658.44	65,257.50	152,915.94	42,097.47	1,428,107.44
Yr 12 Semi Annual		65,257.50	65,257.50		1,428,107.44
Yr 12 Annual	87,658.44	65,257.50	152,915.94	47,287.70	1,293,161.29
Yr 13 Semi Annual		65,257.50	65,257.50		1,293,161.29
Yr 13 Annual	87,658.44	65,257.50	152,915.94	52,685.55	1,152,817.30
Yr 14 Semi Annual		65,257.50	65,257.50		1,152,817.30
Yr 14 Annual	87,658.44	65,257.50	152,915.94	58,299.31	1,006,859.55
Yr 15 Semi Annual		65,257.50	65,257.50		1,006,859.55
Yr 15 Annual	87,658.44	65,257.50	152,915.94	64,137.62	855,063.49
Yr 16 Semi Annual		65,257.50	65,257.50		855,063.49
Yr 16 Annual	87,658.44	65,257.50	152,915.94	70,209.46	697,195.58
Yr 17 Semi Annual		65,257.50	65,257.50		697,195.58
Yr 17 Annual	87,658.44	65,257.50	152,915.94	76,524.18	533,012.96
Yr 18 Semi Annual		65,257.50	65,257.50		533,012.96
Yr 18 Annual	87,658.44	65,257.50	152,915.94	83,091.48	362,263.04
Yr 19 Semi Annual		65,257.50	65,257.50		362,263.04
Yr 19 Annual	87,658.44	65,257.50	152,915.94	89,921.48	184,683.12
Yr 20 Semi Annual		65,257.50	65,257.50		184,683.12
Yr 20 Annual	87,658.44	65,257.50	152,915.94	97,024.68	-0.00
TOTALS:	1,753,168.86	2,610,300.00	4,363,468.86	857,131.14	

20 Year Term

Cost for Treatment Plant Loan - Full Capital Cost

Principal: 7,910,000.00 Interest Rate: **5.00%**

S/F Factor:
0.03358175

	Principal Pymnt	Interest Pymnt	Total Pymnt	Actuarial	Reducing Balance
					7,910,000.00
Yr 1 Semi Annual		197,750.00	197,750.00		7,910,000.00
Yr 1 Annual	265,631.65	197,750.00	463,381.65		7,644,368.35
Yr 2 Semi Annual		197,750.00	197,750.00		7,644,368.35
Yr 2 Annual	265,631.65	197,750.00	463,381.65	10,625.27	7,368,111.44
Yr 3 Semi Annual		197,750.00	197,750.00		7,368,111.44
Yr 3 Annual	265,631.65	197,750.00	463,381.65	21,675.54	7,080,804.26
Yr 4 Semi Annual		197,750.00	197,750.00		7,080,804.26
Yr 4 Annual	265,631.65	197,750.00	463,381.65	33,167.83	6,782,004.78
Yr 5 Semi Annual		197,750.00	197,750.00		6,782,004.78
Yr 5 Annual	265,631.65	197,750.00	463,381.65	45,119.81	6,471,253.33
Yr 6 Semi Annual		197,750.00	197,750.00		6,471,253.33
Yr 6 Annual	265,631.65	197,750.00	463,381.65	57,549.87	6,148,071.82
Yr 7 Semi Annual		197,750.00	197,750.00		6,148,071.82
Yr 7 Annual	265,631.65	197,750.00	463,381.65	70,477.13	5,811,963.04
Yr 8 Semi Annual		197,750.00	197,750.00		5,811,963.04
Yr 8 Annual	265,631.65	197,750.00	463,381.65	83,921.48	5,462,409.92
Yr 9 Semi Annual		197,750.00	197,750.00		5,462,409.92
Yr 9 Annual	265,631.65	197,750.00	463,381.65	97,903.60	5,098,874.67
Yr 10 Semi Annual		197,750.00	197,750.00		5,098,874.67
Yr 10 Annual	265,631.65	197,750.00	463,381.65	112,445.01	4,720,798.01
Yr 11 Semi Annual		197,750.00	197,750.00		4,720,798.01
Yr 11 Annual	265,631.65	197,750.00	463,381.65	127,568.08	4,327,598.29
Yr 12 Semi Annual		197,750.00	197,750.00		4,327,598.29
Yr 12 Annual	265,631.65	197,750.00	463,381.65	143,296.07	3,918,670.58
Yr 13 Semi Annual		197,750.00	197,750.00		3,918,670.58
Yr 13 Annual	265,631.65	197,750.00	463,381.65	159,653.18	3,493,385.75
Yr 14 Semi Annual		197,750.00	197,750.00		3,493,385.75
Yr 14 Annual	265,631.65	197,750.00	463,381.65	176,664.57	3,051,089.54
Yr 15 Semi Annual		197,750.00	197,750.00		3,051,089.54
Yr 15 Annual	265,631.65	197,750.00	463,381.65	194,356.42	2,591,101.48
Yr 16 Semi Annual		197,750.00	197,750.00		2,591,101.48
Yr 16 Annual	265,631.65	197,750.00	463,381.65	212,755.94	2,112,713.89
Yr 17 Semi Annual		197,750.00	197,750.00		2,112,713.89
Yr 17 Annual	265,631.65	197,750.00	463,381.65	231,891.44	1,615,190.80
Yr 18 Semi Annual		197,750.00	197,750.00		1,615,190.80
Yr 18 Annual	265,631.65	197,750.00	463,381.65	251,792.37	1,097,766.79
Yr 19 Semi Annual		197,750.00	197,750.00		1,097,766.79
Yr 19 Annual	265,631.65	197,750.00	463,381.65	272,489.33	559,645.81
Yr 20 Semi Annual		197,750.00	197,750.00		559,645.81
Yr 20 Annual	265,631.65	197,750.00	463,381.65	294,014.17	-0.00
TOTALS:	5,312,632.90	7,910,000.00	13,222,632.90	2,597,367.10	