Final Report on

2009 Drainage Study (Greendale)

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April 26, 2010

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City of Chilliwack
8550 Young Road
Chilliwack, BC V2P 8A4

Attention: Frank Van Nynatten
Supervisor, Roads and Drainage

RE: 2009 Drainage Study (Greendale) - Final Report (Revision 1)

We are pleased to submit the revised 2009 Drainage Study (Greendale) Final Report for your use. It outlines a strategic plan for significant infrastructure upgrades that will provide enhanced drainage service and flood protection to the community of Greendale over time.

We appreciate being given the opportunity to assist the City of Chilliwack, and the community of Greendale, in particular. Please do not hesitate to contact us if you have any questions.

Yours truly,

URBAN SYSTEMS LTD.

PROFESSIONAL ENGINEER G.D. SHKURHAN BRITISH COLUMBIA

April 26, 2010

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Senior Engineer, Principal

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EXECUTIVE SUMMARY

A comprehensive assessment of the Greendale community drainage system was asked to answer four fundamental questions, in response to the flooding event of January 2009:

1. What caused the flooding event?

Chilliwack has been impacted by large flood conditions in the past, most notable in 1894 and 1948 as a result of overtopping of the Fraser River. As demonstrated through past study, flood risk from the Fraser River, Vedder River and Vedder Canal still exists, and therefore, that type of event forms the basis of the City's floodplain bylaw.

In early January 2009, a unique sequence of climatic conditions occurred directly within the community itself, and was not attributed to the overtopping of an external watercourse. The event began with an extended period of freezing temperatures and accumulated snow pack, followed by rapid warming and heavy rainfall. While each individual component is not unique, the combination of them in rapid succession, at the intensities that occurred, is unique for this location. It is not a condition that current standards are based on.

2. How significant was the rainstorm?

The significance of the event is not easily quantified, but was certainly unique. While a reasonable amount of data is available to describe the event, there are still several uncertainties surrounding a number of complex variables. In addition, there are insufficient historic records of an event like this to compare to for this area. As such, it is very difficult to quantify with accuracy the significance of the specific climatic event that impacted Greendale. However, with consideration for the estimated precipitation and snow pack quantities alone, the event return period ranges from 1:50 to 1:200 years depending on the analysis approach.

Another approach taken was to estimate the significance of the observed flood effects rather than the climatic event. This approach involved estimating what magnitude of conventional storm would have been required to result in a similar effect to what was observed in January 2009. Taking this approach, it is estimated to have had a return period of approximately 1:200 years, meaning that such an event could be expected to occur once every 200 years, on average. Again, this is based on the cause being local precipitation, which cannot be compared to other potential risks such as the Fraser River or Vedder Canal overtopping.

3. What part of the drainage system failed to operate?

There is no single part of the system that failed; nor single cause to the problem. The Greendale drainage conveyance system (i.e. channels and culverts), controls (i.e. pump stations, gates, floodboxes, diversions) and topography (i.e. general grading of the land) are all critical components of a complex system. The community's elevation relative to the Vedder Canal and Fraser River is also a significant fact. In general, the internal drainage system and controls are generally sized to their established criteria, however during the January 2009 event they did not have capacity to prevent the floodplain from activating; in which numerous buildings and infrastructure are at risk.

4. What improvements are recommended to reduce the likelihood of a repeat system failure?

To start, it is less a question about recommendations to "reduce the likelihood of a repeat system failure", but more a question about recommendations to "increase the level of protection to vulnerable infrastructure". As noted above, Greendale is a floodplain, and its activation is going to occur from time to time. Having been a former lake, the community's relative elevation and topography is a challenge to flood proof. Unfortunately, the settlement of the community occurred prior to a quantified understanding of the hydraulic performance and risks. There is now greater awareness through the completion of various risk assessment studies and the development of the City's floodplain bylaw. Assessment conducted and presented herein has identified some deficiencies in the conveyance and pumping systems; however, in general the pumping and conveyance systems are providing a level of service consistent with their established criteria; criteria which is not established to prevent activation of the floodplain. Potential actions required to achieve this would be one, or a combination of:

- Major enlargement of most of the drainage network conveyance system combined with new pump stations;
- Internal dyking and pumping systems, to reduce the dependency of the floodplain from the trunk drainage system;
- Regrade land and reconstruct vulnerable buildings at a higher elevation.

Aside from technical challenges, all these approaches would be very disruptive to the community and would require extensive consultation and evaluation that goes beyond the scope of this current study. A more detailed study would also be required to prepare a comprehensive cost estimate and financial plan for any of these potential solutions; however an order of magnitude value would be tens of millions of dollars.

Within the scope of this current investigation, a comprehensive assessment has been completed of the current systems, identifying where the greatest risks / opportunities exist, and suggesting an approach to improving system performance in a way that is practically achievable; with the underlying philosophy of making best use of existing infrastructure in the near term.

By in large, infrastructure improvements investigated include increased pump station capacity, culvert replacements, flow diversions, and new channeling to supplement current ones. A large number of potential solutions were tested against both established criteria and beyond. The total capital value of all options investigated in depth range from about $10M to $19M. The minimum level of service achieved by all options reviewed would be consistent with currently established standards; namely the 10 year criteria for the channels, pipes and culverts, and the 25 year criteria for the pump stations, while some options strive to the 1:100 year level. However, activation of the floodplain and major flow paths will still occur to some extent under extreme events.

In formulating a near term recommendation, four core considerations were given:

1. The ability to meet the minimum performance standard currently established;
2. The cost of works relative to the change in hydraulic performance;
3. Potential risks from other sources (i.e. Fraser River flood)
4. The ability to achieve a gradual implementation strategy

Based on the above, the following core set of recommendations are made for consideration. A significant investment is required to achieve a measureable increase to the level of protection beyond currently established levels. It is recommended that the near term action be taken towards implementing Option 3B, as described below. Option 3B strives to maximize the potential performance, practically. Overall, this option offers improved hydraulic performance above baseline levels and lends itself to a gradual implementation program which City staff view as more financially viable than other options offering similar performance. It is recommended that the City lobby senior levels of government and perhaps consider other financial mechanisms such as a utility fee or a local service area charge with attempt to secure additional funding sources, which may influence future decisions.

Due to overall topographic challenges and channel conveyance limitations across the watersheds system, flooding cannot be practically eliminated. Rather, the recommended works maximize system potential to the 1:100 year level. The specific actions associated with Option 3B include the following. Please refer to Section 8.1 for further description of each action:

1. Upgrade the existing pump stations for a total estimated cost of $4.9M, less GST. (refer to Section 8.1.2)
2. Replace 71 culverts at various locations through the internal drainage system, for a total estimated cost of $8.8M, less GST. (refer to Section 8.1.3)
3. Construct a new 950 m long drainage channel west of Sumas Prairie Road to supplement the McGillivray Creek, for an estimated cost of $918,000, including land acquisition, less GST. (refer to Section 8.1.4)
4. Construct a new 1,670 m long drainage channel north of Keith Wilson Road to supplement the Lewis Slough, for an estimated cost of $1.7M, including land acquisition, less GST (refer to Section 8.1.4).
5. Conduct site specific reviews of select vulnerable homes to assess local drainage and determine appropriate flood protection actions, if any. (refer to Section 8.2)
6. Review and maintain a comprehensive operation and maintenance program. (refer to Section 8.3)

7. Review and update the Floodplain Bylaw accordingly to reflect the findings of this study. (refer to Section 8.4)
8. Implement a 5 year water level monitoring program to obtain additional base line information and to monitor performance after the pump station upgrades are complete, but prior to undertaking further actions such that uncertainties can be reduced and the capital plan confirmed. The total cost of the monitoring program is estimated at $84,000 to implement, $7,000 per year to operate and maintain ($35,000 over 5 years). The total estimated cost of the review and capital plan update is $50,000. (refer to Section 8.5)

There is sufficient information to allow some firm decisions to be enacted, while other decisions require the resolution of uncertainties that cannot be adequately addressed within the scope of this study. As such, some critical early steps are required in order to solidify decisions associated with some of the recommended works. These steps involve:

- Complete system monitoring to enhance the understanding of hydraulic performance in some key areas of the system.
- Conduct a comprehensive review of the existing pump stations.
- Conduct a pump station optimization review that will guide final conclusion on the degree to which to reinvest in the existing McGillivray pump station, to invest in a new supplemental station, or potentially a combination of the two.

With final conclusion made on the physical works, a gradual and systematic approach to implementation is required; starting with upgrading the Collinson pump station, followed by upgrading the McGillivray pump station. Next would be the two supplemental channels to McGillivray Creek and Lewis Slough, and culvert upgrades working in an upstream direction. At this time, no specific time frame has been identified to complete the works, as this is dependent on the City's cash flow position and its decision on municipal wide priorities. When this investigation was launched in spring 2009, provincial and federal grant programs with active, although heavily prescribed. Since that time, and with the economic downturn, suitable grant programs are no longer active and it is unknown if and when new ones will become active for the City to seek funding for this particular initiative.

Some level of liaison with MOE and DFO will be required for all recommended capital upgrade works. It is recommended that the City present this final strategic plan to these agencies and seek a Memorandum of Understanding covering long term operation and maintenance and capital upgrade works; the goal being to stream line approvals and prevent future conflict with respect to process and procedures. Please refer to Section 9 for further detail on the implementation program.

1.0 INTRODUCTION

In January 2009, a unique event caused localized flooding in many areas of the City, with the community of Greendale being one of the most significantly affected. As outlined in Figure 1 (inserted at the end of the report's main body), the Greendale watershed encompasses an area of approximately 30 km^2 bounded by the dykes of the Fraser River along the Trans Canada Highway to the north; Lickman Road and the community of South Sumas on the east; Vedder Canal dyke to the south/south-west, and Sumas River to the west.

Because of its topography and relative proximity, Greendale has been vulnerable to flooding in the past, with the most significant risk and cause being flooding from the Fraser River. The 1894 and 1948 floods were two events that caused widespread flood damage and prompted the development of dykes along the Fraser River in the Chilliwack area. As discussed in subsequent sections of this report, a separate study has been recently completed that looks further into the risks and vulnerabilities of the area from the Fraser River.

However, the January 2009 flood event is different from historic events, in that it was not caused by high water levels in surrounding watercourses (i.e. Fraser River or Vedder River), but rather a unique combination of climatic factors within the Greendale watershed itself. The event consisted of an extended period of freezing temperatures, an accumulation of snow on the ground, combined with a rapid temperature rise and heavy precipitation. The combination of these climatic factors is not common for the Fraser Valley, and therefore different from conventional criteria for the area.

Figure 2 provides a series of aerial photos demonstrating the extent of flooding during the January 2009 event. These photos are estimated to have been taken around the peak of the flood condition.

1.1 Study Objectives

The City of Chilliwack has retained Urban Systems to investigate the unique event of January 2009. While some level of impact was felt at various locations within the City, the scope of this investigation is limited to the Greendale community.

The primary objective of the investigation is to find the probable factors that caused the January flood event and potential remedies to mitigate future damages. More specifically, the City seeks answers to the following four questions:

- What caused the flooding event?
- How significant was the rainstorm?

- What part of the drainage system failed to operate?
- What improvements are recommended to reduce the likelihood of a repeat system failure?

1.2 Scope of Work

To answer the above questions, a multi staged work program was developed to make best use of available funds, consisting of the following tasks:

- Assess the climatic characteristics and magnitude of the January 2009 event relative to historic data;
- Review previous studies and flood protection regulations;
- Compile a comprehensive analytical model of the watershed systems;
- Assess the performance of Greendale's current drainage system against both established flood protection regulations and the January 2009 event;
- Determine system limitations and identify mitigation actions to be considered in improving performance;
- Quantify the benefit / effect that each improvement option may provide against both established criteria and the January 2009 event;
- Conduct an evaluation of the options; and
- Prepare recommendations of capital works and an associated implementation strategy.

1.3 Study Process

The assignment was conducted in 3 primary phases, as follows:

- Phase 1: Data Gathering Phase
- Phase 2: Analysis Phase
- Phase 3: Solution and Reporting Phase

The initial phase of the study involved gathering information on existing drainage infrastructures, identifying critical data that are missing but need to be obtained, and assessing the quality of the data. Coupled with analytical data, anecdotal information was also collected to enhance our understanding of the problem, which included interviews with City staff and seeking input from the general public through an open house process. The compiled information was then assessed and established the foundation for the analysis phase. The analytical phase primarily involved model development, calibration, verification, and an assessment of model simulation results. In Phase 3, evaluation criteria were developed to assess potential flood management alternatives, followed by the formation of recommendations and implementation strategies.

2.0 STUDY AREA OVERVIEW

2.1 Review of Previous studies

There have been several relevant studies prepared in recent years that warrant acknowledgement.

2.1.1 City of Chilliwack Master Drainage Plans

As part of City of Chilliwack's master drainage planning program that started back in 1999, three major drainage studies were completed, as follows:

1. Integrated Master Drainage Plan for the Chilliwack Creek Watershed (CH2MHILL, 2003)
2. Problem Identification Phase of the Hope River Master Drainage Plan (CH2MHILL, 2004)
3. Problem Identification Phase of the Master Drainage Plan for Chilliwack Western Areas (CH2MHILL, 2004)

The third study listed above is most relevant at this time, as it too studied the performance of the Greendale drainage system. The primary objective of that 2004 Master Drainage Plan was to assess overall performance against established criteria and then inform the development of the City's capital program. Its technical focus was on the conveyance capacity of the channel / culvert systems alone. It did not comprehensively investigate floodplain or pump station performance. In addition, assessment of the conveyance system was highly skeletonised; looking only at the performance of the trunk system. The general findings of that investigation did not identify significant performance deficiencies or system improvements; however, given the known limitations, this prior study did recommend that topographic and system data be improved for detail and accuracy as part of the capital planning and project implementation process. As such, this past study offered some baseline information, but was incomplete. While limited in scope and technical content, the analytical work of the 2004 study was conducted using the MOUSE (DHI) modeling software, and therefore was used as a foundation to create an updated and expanded model for this current study.

This current Greendale study overcomes the limitations of the previous study by looking into the Greendale drainage systems more comprehensively. This process has been significantly assisted by the acquisition of high resolution topographic information, which did not previously exist, as well as the application of modelling approaches that account for the complex drainage systems and hydrologic processes in this watershed.

2.1.2 2008 Fraser River Hydraulic Model Update (nhc, 2008)

The 2008 Fraser River Hydraulic Model update report (nhc, 2008) elaborates the process and the findings of the comprehensive Fraser River hydraulic model update project by the BC Ministry of Environment (MOE). The report presents the updated design flood levels (200 year return period) in the Fraser River between Laidlaw and the Strait of Georgia and includes a qualitative assessment of the existing dyke elevations with respect to the design 200 year flood level. In order to provide stakeholders with accurate real-time water levels, the model also has an excellent ability to forecast flood levels depending on the boundary conditions. The forecasting model is a dynamic model that is maintained through regular updates.

That report documents the estimated water levels within the Fraser River, which have been used as boundary conditions at the Wilson Slough, McGillivray PS, and Collinson PS outfalls for this current Greendale study. Another noteworthy aspect of the Fraser River study is that it provides a qualitative assessment of the dike elevation, indicating that the Vedder River right bank dike crest is below the design flood level, whereas the Vedder River Left Bank dike crest is above the design flood level in most cases (Ref: Table 7.2). This information is of general interest to be aware of all potential flood risks; however it does not influence this current Greendale study, which is focussed purely on internal performance and conditions.

2.1.3 Quantitative Flood Risk Analysis (BGC Engineering Inc., 2009)

Flood management in British Columbia has traditionally been based on the height of a design flood augmented by a standard freeboard. For the Fraser River, the design flood is the largest flood on record (1894), whose return period has not been defined with great certainty, though it is believed to be in the range of 200 to 1000 years (ref. BGC Engineering Inc, 2009 report). While this pragmatic approach has served communities reasonably well since the 1948 flood, greater consideration for changing risk and potential consequences of failure have motivated movement towards a risk-based flood management approach that systematically accounts for flood consequence.

The 2009 Quantitative Flood Risk Analysis study introduces the risk-based approach that systematically assesses potential consequences and vulnerabilities due to Fraser River flooding within the City of Chilliwack. This report provides a preliminary quantitative flood risk assessment, including the assessment of damage and loss for three potential dike breach scenarios that are regarded as the most likely flood defence failure locations, should a failure occur. This study does not claim to be a complete flood risk assessment, which would require additional work such as reliability analyses for dikes and their respective probabilities of failure and consideration of a wider spectrum of flood frequencies, losses of life, types of development, and quantification or estimates of environmental and social losses. As the study suggests, this study:

- Provide a systematic, transparent, repeatable and scientifically defensible method to quantify losses and vulnerabilities due to flooding of developed areas on the Fraser River floodplain;

- Provide a logical, science-based roadmap to prioritize mitigation efforts for areas of highest potential loss;
- Provide a comprehensive and updatable database of elements-at-risk on the Fraser floodplain for the District of Chilliwack;
- Provide an efficient method to allow future assessment of potential losses for different flood scenarios and flood mitigation alternatives such as setback dikes, flood ways, building covenants and land sterilization;
- Identify areas of higher human vulnerability to assist with emergency planning;
- Identify areas of flooding with respect to agricultural resources to assist with emergency planning; and
- Improve frequency-return period analysis used to refine dike elevations.

Of all the reports discussed above, the 2009 Quantitative Flood Risk Analysis study perhaps best compliments this Greendale study; being most influential in helping the City move forward in creating a comprehensive approach to flood risk management, setting broader priorities, and establishing a pragmatic approach to implementation.

2.2 Consultation Process Overview

A valued step of the study process was active engagement by local residents in the Data Gathering Phase of the investigation. Two open houses were arranged (May 21 and 28, 2009) at the Greendale community fire hall to both present and seek information from the local residents and business owners about '*what happened'* and '*how they were impacted*' during the January 2009 flood. Questionnaires were filled out, and in some cases residents submitted supplemental information in the form of photos and sketches. Although generally anecdotal in nature, this information helped supplement and validate the findings of the investigation. Further details of this consultation process are provided in Appendix C.

City operation and maintenance staff were also interviewed during the Data Gathering Phase to enhance understanding of how City's drainage system performed during the January flood, what kind of challenges they faced and what measures the City took during the event. In addition, they were consulted for confirmation of control structure settings and operation, tour of pump stations, and channel maintenance procedures. City staff was consulted on various occasions throughout the early phase of the study. Details of this consultation are also provided in Appendix C.

2.3 Watershed and Drainage System Overview

The study area outlined previously in Figure 1 encompasses 3,072 hectares. Land use is primarily agricultural, with small pockets of single family residential and neighbourhood commercial designations. Topography is a significant aspect of drainage and floodplain performance for this area. Grades are generally flat, gently sloping in a south-east to north-west direction towards the Vedder Canal and Fraser River. But at the same time, there are many local undulations with numerous isolated depressions that are prone to poor surface drainage and flooding. Further information is provided in Section 5 Model Development.

As portrayed in Figure 3, drainage is conveyed through a network of watercourses and road side ditches to three primary outfall structures prior to discharge into the Fraser River and Vedder River; namely the Wilson Slough outfall, McGillivray Pump Station, and Collinson Pump Station. The study area is divided into three primary basins referred to as the McGillivray basin, the Salwein-Barret Creek basin and the Wilson Slough basin. Brief descriptions of these basins are as follows:

2.3.1 McGillivray Basin

This 2,026 ha area is bounded by the Fraser River levee on the north; Lickman Road on the east; the BC Hydro railroad on the southeast; and the Vedder Canal/Sumas River levee on the south, southwest and west. On the southwest, the geodetic ground elevation is approximately 3 m and gently rises to 14 m on the east limit. Internal drainage is primarily provided by an extensive, interconnected network of drainage channels. Wilson Slough, McGillivray floodbox and pump station, and the Collinson pump stations provide external drainage to the system. The system is operated in a manner such that gravity discharge can be maximized and pumping operations can be minimized.

2.3.2 Salwein-Barret Creek Basin

This basin is located west of the South Sumas community, north of the Vedder Canal levee, and southeast of the BC Hydro Railroad. The general direction of drainage is east to west-southwest. Geodetic ground elevations vary between 10 m on the southwest corner of the basin and 27 m on the east in South Sumas, resulting in significantly more grade relative to the other basins. It is understood from City sources that Salwein Creek is considered to be a prime salmon habitat. Habitat enhancement projects have been implemented by the Department of Fisheries and Oceans of Canada (DFO) in the vicinity of Keith Wilson Road.

2.3.3 Wilson Slough Basin

Wilson Slough, Chadsey Ditch, and their tributaries provide drainage to the easterly 690 ha of the area, located east of Hopedale Road and south of the BC Hydro Railroad. The primary outlet of the system consists of two culverts under Highway 1. The outlet of the easterly culvert (1,600 mm diameter reinforced concrete pipe) is equipped with a manually operated sluice gate. The outlet of the westerly culvert (1,500 mm diameter CSP) is equipped with an automatic flap gate.

Outside of the freshet period, flow through these culverts occurs, however during freshet periods, the gates are closed, putting reliance on the drainage and pumping system in the McGillivray Basin. The transfer of water from the Wilson Slough Basin to the McGillivray Basin occurs at interconnects at Hopedale Road, as noted below.

- To Miller Slough just south of Hwy. No. 1. – controlled by a manually operated rotating gate.
- To Branch MC-2 of McGillivray Creek via a 1,500 mm CSP culvert. This connection is permanently open.
- To Tranmer Ditch via a 600 mm CSP culvert, controlled by a sluice gate used for irrigation purposes only and is closed for flood control.

Based on information provided by City staff, the system is operated according to water levels in the Fraser River. The objective is to maximize gravity outflow to the Fraser River and minimize discharges into the McGillivray Basin. There are two typical operating modes for this system:

Winter Operation

The water levels in the Fraser River are typically low and gravity outflow from the system is feasible. During winter, the diversion that diverts flow into the Miller slough is closed and all storm water runoff, except the diversion into Branch MC-2 of McGillivray Creek, is discharged into the Fraser River.

Spring Freshet Operation

In spring, the water levels in the Fraser River are typically high and gravity outflow from the system is not feasible. The gates in the Wilson Slough are closed to prevent water flowing back from the Fraser River. The flow diversion into the Miller Slough is open. All runoff is diverted into the McGillivray Basin through Miller Slough and Branch MC-2 of McGillivray Creek, and ultimately discharged through the McGillivray pump station. In between the winter and freshet periods there are several intermediate operational modes. These modes become operational according to the various combinations of the Fraser river water levels and flooding conditions inside the system. The described operating protocol has been used in the development and assessment of the model.

2.4 Overview of Soils

Soil mapping (Soils of the Langley Vancouver Map Area' by the Provincial Ministry of Environment) indicates that a significant portion of the study area (60-70%) has soils with high silt and/or clay content, which are imperfectly to poorly drained, moderately to slowly pervious, and cause moderate to slow surface runoff. Ponding takes place frequently during high storm events. A fluctuating water table is present which is usually high during the winter, during the freshet period of the Fraser River and after heavy, prolonged rains.

The other major soil type available in the Greendale area (30–40%) consists of well-decomposed organic material with underlying silty clay loam or silty clay soils. These soils are poorly to very poorly drained, and slowly pervious. Most of these soils are found at a higher elevation than the valley floor and provide better surface drainage. Because of higher elevation, the water table is less likely to be influenced by the Fraser River or Vedder Canal water levels.

There are few areas in Greendale (less than 1%) that have soils composed of sand with varying degree of sandy loam and silty loam. These soils are moderately well to well drained, moderately pervious and cause slow surface runoff. Temporary groundwater tables may develop in the subsoil during the freshet or during heavy prolonged rain.

3.0 REVIEW OF JANUARY 2009 FLOOD EVENT

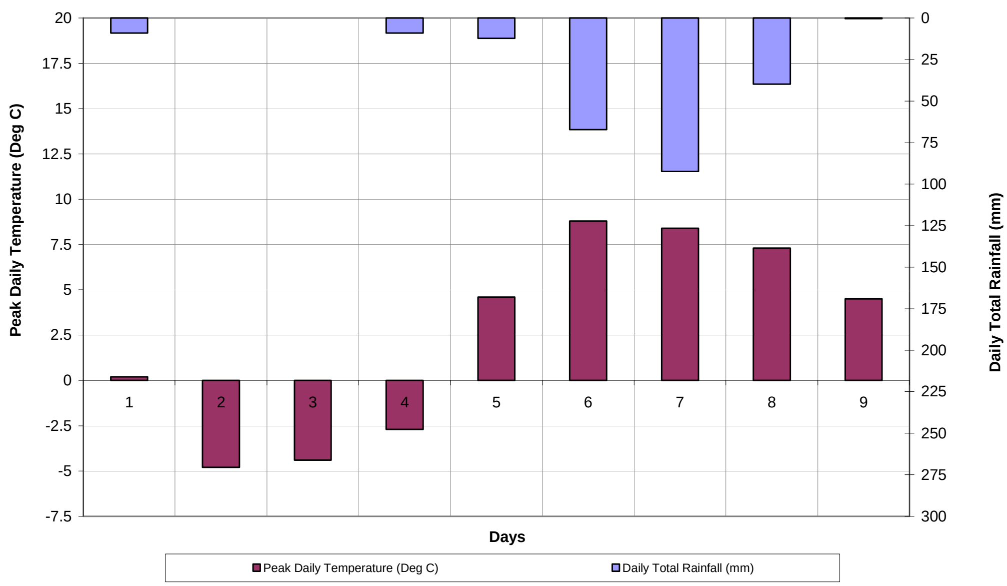
As noted in Section 1.1, one of the fundamental questions this study was asked to answer is "how significant was the rain storm?"

Temperature and precipitation data from local recording stations has been used to assess the uniqueness and significance of the January 2009 event. Extended freezing temperatures, accumulated snow pack, rapid warming and heavy rainfall combined for a unique event. Unfortunately there is no weather gauge located directly in the Greendale community, therefore, the weather gauge at Chilliwack Airport (City Gauge) and the Environment Canada gauge 'Chilliwack' were used. The 'Chilliwack Airport' gauge collects rainfall and temperature data at 5 minute intervals. Information is available since 2005, offering insufficient information to assess how the January 2009 event compares to historical data. The Environment Canada gauge 'Chilliwack' provides record between 1889 and 2007, and was therefore applied for assessing statistical relevancy of the January 2009 event.

Temperature and precipitation are two major climatologically variables that impact a flooding event. The significance of the combination is far different then when looked at as independent variables. However, the assessment approach has been to first look at their significance as independent variables, then in combination, recognizing its uniqueness for this area.

Figure 4 shows the relationship of temperature and rainfall over a nine day period. Between January 2 and January 4 the 'Chilliwack Airport' gauge recorded continuous subzero temperatures with daily highs between -2.5 and -4.8 ^{0}C. Unfortunately the local stations do not record snowfall, or equivalent rainfall during freezing conditions, therefore snow accumulation estimates are only available anecdotally from staff and residents, which suggests that there was in the order of 200-300 mm of snow on the ground at the onset of the rainfall event. It is also presumed that the ground was largely saturated due to earlier precipitation and freezing temperatures. As such, while no field data exists to quantify it, it is also presumed that infiltration into the ground was extremely limited at the start of the rainfall event. Combined with a rapid temperature rise, a peak 3-day (72 hour) rainfall of 161 mm fell at the Chilliwack Airport, and 200 mm at Promontory. The 72 hour peak period has been selected for the purposes of evaluating the statistical relevancy. The total precipitation between January 4th and 8th was approximately 220 mm.

Figure 4: Temperature and Rainfall Pattern between January 1st and January 9th , 2009



3.1 Climatic Assessment to Historical Data

Since 1894, the City of Chilliwack has experienced major floods due to the Fraser River flood flows; however, the January flood was not caused by surrounding watercourses, but rather the unique local climatic conditions. This section of the report provides an overview of the climatic assessment.

To characterize this flood event with statistical significance, independent frequency analysis of each climatic factors was first conducted; namely the daily total rainfall, daily maximum temperature, and daily minimum temperature. Daily data (1900-2006) from the Environment Canada weather station 'Chilliwack' was used for the analysis. There is no snowfall record available; therefore it is not possible to accurately assess the significance of this variable.

The bulk of the rainfall occurred over a 72 hour period; therefore frequency analysis was completed for this time duration. As stated in the previous section, while the total rainfall event occurred over a 5-day period, the significant portion of it fell in a 3-day (72 hour) period with a rainfall depth of 161 mm at the Chilliwack Airport and 200 mm at Promontory. This peak 72 hour period has been applied for the purposes of evaluating the statistical relevancy. Although discussed in later sections, analysis suggests that Greendale did experience a large volume of water, seemingly larger than what was recorded at the Chilliwack Airport and Promontory stations. Also due to relative proximity, greater focus was placed on the records from the Promontory station. Those available records suggest that the 3 day (72 hour) rainfall during the January 2009 event had a return period of between 1:25 years and 1:50 years depending on the regression analysis. With an estimated 30 cm snow pack also factored in, the total rainfall equivalent volume rises to approximately 230 mm and a return period between 50 years and 200 years, again depending on the regression approach. This represents the average interval that an event of this magnitude can be expected to occur.

When looked at independently, frequency analysis of the daily minimum and maximum temperature data indicates that the temperatures during the January 2009 event were common occurrences; expected to occur with a frequency of less than two years, on average.

3.2 Statistical Significance of the January Event

While it is fairly simple to evaluate the statistical significance of each independent variable, it is not simple to assess joint probability of all combined factors that contributed to the flooding for this unique event, due to a lack of information for both the January 2009 event, as well as the historic combination. Aside from the cause, climate change trends are also influencing statistically relevancy, adding additional uncertainty. As such, a different approach was also taken to estimate the statistical significance of this complex and unique event. The approach taken considers the statistical significance of the observed effects, rather than the specific climatic conditions.

The approach applies the calibrated hydrologic / hydraulic model (refer to Section 5) to compare the flooding impact from the January 2009 event against those that would be caused by conventional precipitation events for which statistical records are readily available. For this comparison, the McGillivray flood box is considered closed and the Wilson Slough flood box is considered open, as this is how the system operated during the January event.

A point of measure had to be selected for this comparison; in this case the peak water levels at the two pump stations. Using the calibrated model, a conventional, 24 hour synthetic design storm was scaled upwards until the peak water levels predicted at the pump stations match what was observed during the January 2009 event. This process suggests that the effects resulting from the complex and unique January event are similar to what would occur from a 1:200 year, conventional rainfall event which assumes no prior freezing or snow pack conditions; results that generally similar to looking directly at the precipitation records alone.

4.0 FRAMEWORK FOR FLOOD MANAGEMENT PLANNING

Evaluating the January 2009 event and considering actions that may be taken to protect against future events, it is necessary to consider established criteria.

4.1 City of Chilliwack's existing drainage and flood protection criteria

In May 2002, the City formulated the 'Policy and Design Criteria Manual for Surface Water Management in the City of Chilliwack,' which provides a comprehensive framework to develop and implement a sustainable, integrated storm water strategy at watershed, neighborhood and subdivision scales. According to the design manual, the storm water drainage system is normally sized to convey 10-year or 100-year peak flows, the pump station is sized for 25-year spring/summer peak flows and the flood boxes are sized for 100 year fall/winter peak flows. The flood management objective for urban areas is to ensure that the drainage system can convey runoff from extreme rainfall events, up to a 100-year storm, without posing a threat to property or public safety. The City's criteria document also discusses application of the ARDSA standard, which is intended to address agricultural production, which is not understood to be an issue, and not a focus of this investigation. The predominant focus of this investigation is around conveyance and flood protection, therefore application of events other than ARDSA winter and summer are expected to govern.

4.2 Floodplain Bylaw

The City adopted a floodplain bylaw 'Floodplain Regulation Bylaw 2004, No. 3080' that designates the Greendale area and other areas as floodplains and makes provisions in relation to flood control, flood hazard management and development of land subject to flooding or erosion. The floodplain bylaw includes a 'floodplain map (Schedule A) showing the flood construction levels (FCL) that were developed based on the Fraser River design flood. The bylaw specifies different setback and elevation requirements for areas not protected by standard dikes and also for areas protected by standard dikes. However, there are some general exemptions and local exemptions. Following are some of the key setback and elevation requirements:

4.2.1 Setback Requirements

"Unless specified elsewhere in this Bylaw, no landfill or structural support required to support a floor system or pad of a building or structure for which full flood proofing or partial flood proofing is required, shall be constructed, reconstructed, moved, extended or located:

(1) In an area not protected by a standard dike where the natural ground elevation is less than the applicable Flood Construction Level from the Fraser River, Chilliwack River, Vedder River, Vedder Canal;

(2) Within 30m of the Natural Boundary of the Fraser River, Chilliwack River, Vedder River, Vedder Canal or any Watercourse on an Alluvial Fan;

(3) Within 15m of the Natural Boundary of any other watercourse or the edge of a bluff, subject to erosion or 3x the height of the bluff (as measured vertically from the toe to top of bluff), whichever is greater; and

(4) Within 7.5m of the Natural Boundary of a lake, swamp, pond, drainage ditch or any structure for flood protection or seepage control or any dike right-of-way."

4.2.2 Elevation Requirements

FOR AREAS PROTECTED BY STANDARD DIKES

"Unless specified elsewhere in this Bylaw, no building, Manufactured Home or unit, shall be constructed, reconstructed, moved, extended or located with the underside of a wooden floor system or top of a concrete slab of any area used for habitation, institutional use, assembly use, tourist accommodation use, business, or storage of goods damageable by floodwaters, or in the case of a Manufactured Home or unit the Pad on which it is located, lower than:

(1) the FCL for the Fraser River, Chilliwack River, Vedder River or Vedder Canal as shown on the Schedule "A" Floodplain Map. "

FOR AREAS NOT PROTECTED BY STANDARD DIKES

"Unless specified elsewhere in this Bylaw, no building, Manufactured Home or unit, shall be constructed, reconstructed, moved, extended or located with the underside of a wooden floor system or top of a concrete slab of any area used for parking, basement, entrance foyer, habitation, institutional use, assembly use, tourist accommodation use, business, or storage of goods damageable by floodwaters or in the case of a Manufactured Home or unit the Pad on which it is located, no lower than:

(1) The FCL for the Fraser River, Chilliwack River, or Vedder River as shown on the attached Schedule "A" Floodplain Map;

(2) 1.5m above the natural boundary of any natural watercourse; and

(3) 0.6m above the top of bank of any drainage ditch."

This last section of the Bylaw, noted immediately above, is the most pertinent to this Greendale assessment. More discussion is provided in Section 7.8 and 8.4.

4.3 Develop Performance Assessment Criteria

In Section 1.1, it was mentioned the City seeks answers to four questions, three of which are answered above. The final question is '*What improvements are recommended to reduce the* *likelihood of a repeat system failure*?' This is a complex question that requires several considerations.

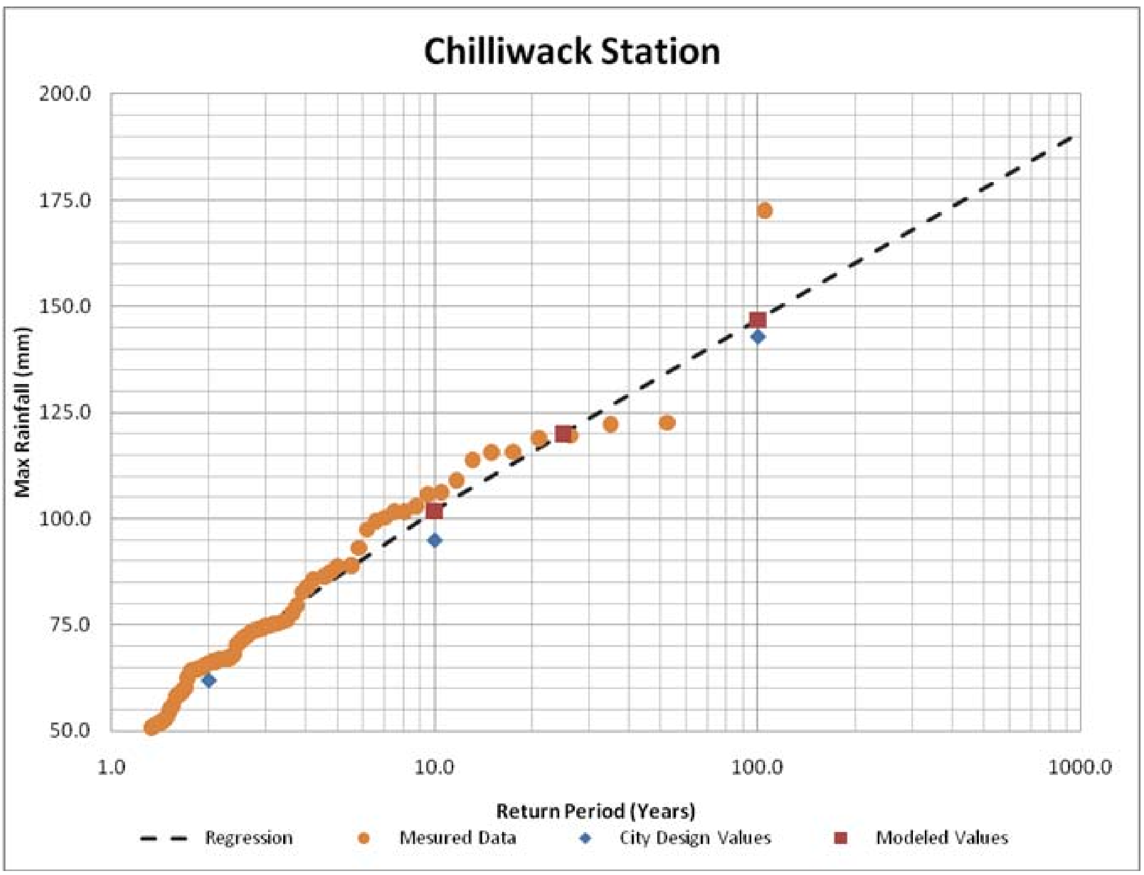
The City's existing drainage criteria, as documented in the 'Policy and design criteria manual for surface water management in the City of Chilliwack' (Final Draft May 2002), represent the most relevant design criteria, and used as the benchmarks to evaluate performance. Although not an established criteria, the unprecedented January 2009 event is also used as potential performance target. As previously discussed, there are a complex set of variables that influence performance for this area, with possible combinations being nearly endless. As such, a set of scenarios and boundary conditions needed to be established, as summarized in Table 1.

Table 1 – Summary of Performance Assessment Criteria

Scenario	Storm Event	Ground Condition	Fraser River Water Level	Vedder River Water Level	Performance Standard
1	10 year return period (24 hr event)	Ground water table close to surface (saturated condition)	High water level causing gates to be closed.	High water level causing gates to be closed.	Represent Freshet condition. Minor conveyance system should be adequate to convey flows from 10 year storm event without excessive surcharge or channel breach.
2	25 year return period (24 hr event)	Ground water table close to surface (saturated condition)	High water level causing gates to be closed.	High water level causing gates to be closed.	Represent Freshet condition. Pumps should be designed to convey flows from 25 year storm event without causing additional surcharging into the upstream conveyance system.
3	100 year return period (24 hr event)	Ground water table close to surface (saturated condition)	Winter period water level allowing gates to function.	High water level in the River forcing gates to be inoperable.	Represent winter condition. Breach of minor drainage system and controlled flooding to occur, provided habitable property, infrastructure, and public safety is protected.

Scenario	Storm Event	Ground Condition	Fraser River Water Level	Vedder River Water Level	Performance Standard
4	January 2009 event	Ground water table close to surface; snow on ground; frozen ground.	Winter period water level allowing gates to function	High water level in the River forcing gates to be inoperable.	No established criteria for this event.

For this purpose of this study, 24 hour SCS storm event distributions were applied to statistical design storms. This distribution generates higher, and therefore, more conservative flow rates than a longer, ARDSA type event with equal return period. The 1:10 year, 1:25 year, and 1:100 year design storms were created using regression analysis of historic daily rainfall data from the Environment Canada Chilliwack weather station, of which are graphed below. For comparison, the graph has also plotted the City's current 2, 10 and 100 year criteria values, which are generally consistent, but slightly lower than those computed by regression analysis.



5.0 HYDROLOGIC AND HYDRAULIC MODELLING

5.1 Model Development

The hydrologic and hydraulic models were developed using the Danish Hydraulic Institute (DHI) software MIKE SHE and MIKE 11 respectively. MIKE-SHE is a powerful, physically-based hydrologic model that accounts for the complex processes of overland flow and surface storage, infiltration and groundwater interactions, and evapotranspiration. MIKE SHE was coupled with the one dimensional hydro-dynamic MIKE 11 model to simulate the hydraulic conveyance system comprised of ditches, channels, culverts, pump stations and other key structures. The configuration of the conveyance system, as it is understood from available records, was shown in Figure 3.

LIDAR (Light Detection and Ranging) topographic data was supplied by the Fraser River Basin Council (FRBC), via the City of Chilliwack, providing flood plain and channel detail that did not previously exist for past studies. The resolution of the LIDAR information was sufficient to define the shape of the floodplain, and generate channel geometry for model development. This data set has been relied upon for accuracy and completeness, and has not undergone field verification specific to this study. The topographic surface generated from the LIDAR information is presented as Figure 5. While generally sloping from east to west, there are a number of undulations that form isolated depressions that will always be prone to flooding no matter what the capacity and performance of the conveyance system. A number of the more significant isolated depressions are also outlined on Figure 5. Although discussed further in the following sections, topography, and the relative elevation of buildings, is a significant influence on the performance and vulnerability of the Greendale community.

LIDAR data is reported with a 1 m x 1 m grid cell resolution. For an area the size of Greendale, and for the large number of scenarios that were assessed for this assignment, the 1 m x 1 m resolution data would have been excessively constraining to the analytical processing. As such, for the sole purpose of analysing the floodplain hydraulic performance, topographic information of the floodplain was converted into a 40 m x 40 m grid cell resolution.

Culvert and other hydraulic structure information, such as pump stations, flood boxes, and gates, were developed from records provided by the City.

5.2 Model Calibration and Verification

The hydrologic and hydraulic conditions and processes during the January 2009 event were complex. There is relatively limited data available for model calibration / verification compared to the extensive number of system variables. Two key pieces of information have been used:

Measured water levels at the pump stations were used for calibration; aerial photos taken during the flood event were used for validation.

Measured water levels at the pump stations were available from SCADA (supervisory control and data acquisition) records. For the Collinson pump station, pumping hours, pump bay water level, and pond level (outside the pump bay) were recorded at one hour interval between 2005 and 2009. At the McGillivray pump station, in addition to the above data, the Vedder River level is also recorded, because of the flood box at that location. There are five important variables for which there is insufficient information to complete comprehensive calibration and verification, therefore represent potential sources of error. These include:

- Operating efficiency of the pump stations and control gates
- Snow pack accumulation volume prior to rainfall event
- Groundwater levels and infiltration / exfiltration rates
- Precipitation rates directly within the Greendale community
- Roughness and efficiencies of channel and culverts (potential debris or barriers have not been accounted for)

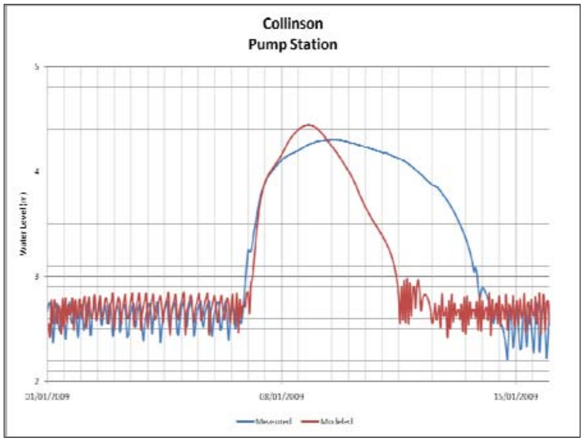
Many variables remained fixed through the calibration process; however the variables that were adjusted to achieve a best fit included:

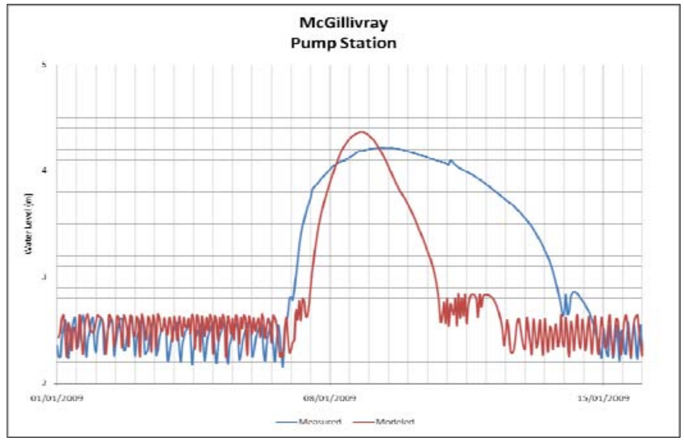
- pump station efficiency
- groundwater conditions
- channel roughness
- snow accumulation and melt rates

We understand that the City has not had the stations physically tested to determine true pumping performance relative to the original design set points. Anecdotally, we understand from City staff that there are no observed problems with the performance of the pumps, however, it is possible for efficiencies to drop by as much as 20% without being obvious. The ultimate pump efficiency used in the calibrated model is 80% of the set design point, which assisted with achieving a best fit between observed and measured water levels.

The best fit achieved by the calibration process is shown in Figure 6.

Figure 6 - Model Calibration Water Levels





The comparison reveals a very good fit for the base flow conditions prior to and following the flood event, leading limb of the flood up to the peak condition, and the peak level. However, the observed recession of the flooding extended far longer than the model is able to replicate. Through extensive sensitivity analysis, this recession limb could not be replicated by the model. A better fit would have required purely arbitrary changes to the data and parameters, which is not a sound analytical practice. There could be many causes for this discrepancy, but there is insufficient data to achieve a better fit or make conclusions on the cause. The most likely causes may be higher than predicted water volumes associated with precipitation, groundwater intrusion, and / or a less than expected pump station efficiency due to debris clogging the inlets to the pump stations. With that said, it is the rising limb and peak conditions that are most critical for decision making, therefore sufficient comfort exists in the results of the purposes of this study.

A threshold value needed to be selected for what constitutes "flooding", as opposed to surface runoff and minor depression storage that occurs within the roughness of the soils and ground cover, which is largely undetectable by high aerial observation. For the purposes of this study, a threshold depth of 0.05 meters has been selection. Throughout this study, all graphics and measurements reporting the aerial extent of flooding have applied this threshold value.

For model validation, the extent and distribution of flooding depicted by the model results were compared to aerial photos captured by the City on January 9th, 2009. The City estimates that the date and time of the photos represent the near peak flooding condition. Three representative samples of the validation comparison are shown in Figure 7, resulting is a comparatively good fit across the study area.

6.0 PERFORMANCE ASSESSMENT

Using the criteria described in Table 1, four baseline scenarios were analyzed to assess the performance of the existing drainage infrastructures. They are used as a point of comparison against which mitigative actions are assessed. The scenarios are:

Baseline 1:10 year event:
This scenario represents the existing drainage infrastructure with a 10-year storm event during a freshet period. A 10-year storm event (minor event) was used to assess the adequacy of the conveyance system (channels and culverts). The performance target under this criterion is that the conveyance system does not breach or be the direct cause of property flooding.

Analysis indicates that most of the channels and culverts are adequate to convey flows generated by a 10-year storm event, however there are definitively some culverts considered too small to optimize system potential; in the sense that their capacity is less that the conveyance capacity of the channel in which they are in. These culverts are highlighted on Figure 8.

As discussed in Section 2, aside from the conveyance capacity of the drainage system, topography has a big influence on the ability for land to drain (assuming no subsurface drainage system is present). An overall image of flooding extents is shown in Figure 9.

Baseline 25 year event:
This scenario represents the existing drainage infrastructure during a 25-year storm event in a freshet period. A 25-year storm event was primarily used to assess the performance of the pump stations, which is the City's currently stated design criterion for pump stations.

Particularly over time in their life cycle, it is not uncommon for pump stations to performance below their intended design point. It is possible for pump performance to drop upwards of 20% without being obvious to the operator. Design point information was obtained from the City and used as a start point for model calibration and performance assessment, however there is no performance testing information available, therefore there is a level of uncertainty on how close the stations are performing to their original design point. Station efficiency was ultimately a variable used in the model calibration process. A summary of the pump station evaluation for this criterion is as follows:

Pump Station	Original Design Capacity	Estimated Capacity by Calibration	Required Capacity to Satisfy 25 yr Criterion
Collinson	3.1 (m^3/s)	2.7 (m^3/s)	4.4 (m^3/s)
McGillivray	5.6 (m^3/s)	4.9 (m^3/s)	6.2 (m^3/s)

An overall image of flooding extents is shown in Figure 10. This is based on the "estimated capacity by calibration" values. Information regarding pump station upgrades is provided in Section 7 and 8.

The conveyance system was not investigated under this scenario, as the 25 year event is not currently an established criterion for capacity of the conveyance system.

Baseline 1:100 year event:

This represents the predicted performance during a 100 storm event in a winter condition, when flood box capacity is available and both the McGillivray and Wilson Slough locations. For this magnitude of event, particularly within a designated floodplain, the risk of flooding cannot be eliminated. The expectation under this criterion is that flooding will be managed; habitable property, critical infrastructure, and public safety will be protected. An overall image of flooding extents is shown in Figure 11.

Baseline January 2009 event:

This represents the calibrated model results of the January 2009 event. As discussed in earlier sections, the flood magnitude for this event is estimated to be equivalent to upward of a 1:200 year conventional storm. An overall image of predicted peak flooding extents is shown in Figure 12. A daily progression of the event as predicted by the model is shown in Figure 13. Based on the reported information through the public open house process and emergency claim information reported to the City by Greendale residents, Figure 12 also highlights the most vulnerable zones from the January 2009 event, which again are generally consistent with what is being predicted by the calibrated model. The urban central core of the community is the one area which is known to have been more heavily impacted than the predicted model is suggesting. Through further review and model calibration, the observed flooding could not be more accurately matched in the community core area. Flow through McGillivray Creek is a critical aspect to the flood risk to the uban core. The difference between predicted and observed flooding in this area cannot be resolved at this time due to too many unknown variables. Possible blockages may occur from time to time due to circumstance beyond reasonable maintenance control. Commentary with respect to operation and maintenance of the system is provided in sequent sections of this report.

Overall, the analytical assessment of this event suggests that neither the conveyance system nor the pumping system had sufficient capacity. The system as a whole was overwhelmed by the rate and quantity of runoff generated. However, as with the other scenarios, general topography also has a strong influence on the performance and flood vulnerability for this area. Performance is expected to have been impacted by the frozen ground, which early in the event likely prevented infiltration. Later in the event, considerable groundwater intrusion from external areas may have occurred as lands thawed and drained through subsurface pathways into the Greendale conveyance system.

During the consultation process of this study, there were questions raised by the public with respect to the efficiency and operation of pump stations and control gates. These are valid questions and may also be likely causes to justify the anomalies between the observed and modeling outcomes, but without extensive record and field measurement, there is no way to quantify these aspects at this stage. With that said, analysis does indicate that regardless of these operational questions, the existing systems were simply under capacity for this unique event.

There are no local criteria or precedence for this unique event, but it has been used as a point of comparison to assist with decisions about future standards and potential mitigation actions.

Summary of Baseline Performance

To supplement the preceding figures, the following table provides a measure of the estimated effects, or impacts under each scenario.

Baseline Performance	10 year	25 year	100 year	January 2009
Estimated vulnerable homes	1	5	12	27
Estimated vulnerable “out buildings”	28	80	113	150
Estimated Vulnerable roads flooded (m2)*	0	17	475	909
Estimated maximum flooded area (Ha)	312	621	783	871
Estimated maximum flood volume (Ha.m)	56	121	165	240

* - represents pavement area

The above are estimates based on best available information, and do not necessarily represent absolute impacts. This information is communicated as a point or reference for comparing mitigation actions, and to bring a level of awareness to areas that are expected to be most vulnerable. The same measures are used in subsequent sections of this report to compare the benefits of mitigation actions. All values reported are qualified as follows:

- Building classification – as accurately as possible, all buildings were identified from the orthophoto and categorized as either of “home” (i.e. dwelling unit) or “out building” (i.e. all other buildings).

- Point of measure – the vulnerability of all buildings is measured by comparing the modeled maximum water elevation adjacent to the building to the estimated lowest ground elevation around the perimeter of the building. The estimated ground elevation **does not** represent the floor elevation (MBE) of the building; it is interpreted using GIS software processing the LIDAR topographic data. A similar approach is taken for roadways, however in this case the modeled maximum water elevation is compared to the surface elevation of the roadways.

Further discussion is provided in subsequent sections of this report for those homes and roads which may remain highly vulnerable even with the application of mitigation actions.

6.1 Sensitivity Assessment of Potential Actions

Given the complexity of the Greendale systems, and the high number of variables that effect performance, before developing mitigation solutions, model sensitivity analysis was conducted to better identify what actions warranted consideration.

Many individual, but co-dependent, components affect the performance of the Greendale system. Added complexity is that different components are currently guided by different criteria. Identifying what actions should be taken requires a systematic process. The start point was to divide the system into its key components; in this case:

- pump stations and floodboxes
- channels, ditches and culverts
- topography and flood plain storages

The second step in the process was to isolate each key component and conduct sensitivity analysis to better understand their independent significance to overall performance. Operational settings and boundary conditions at the pump stations were varied to first identify critical conditions (e.g. winter versus freshet). Building on baseline conditions, a number of independent physical actions were considered, including:

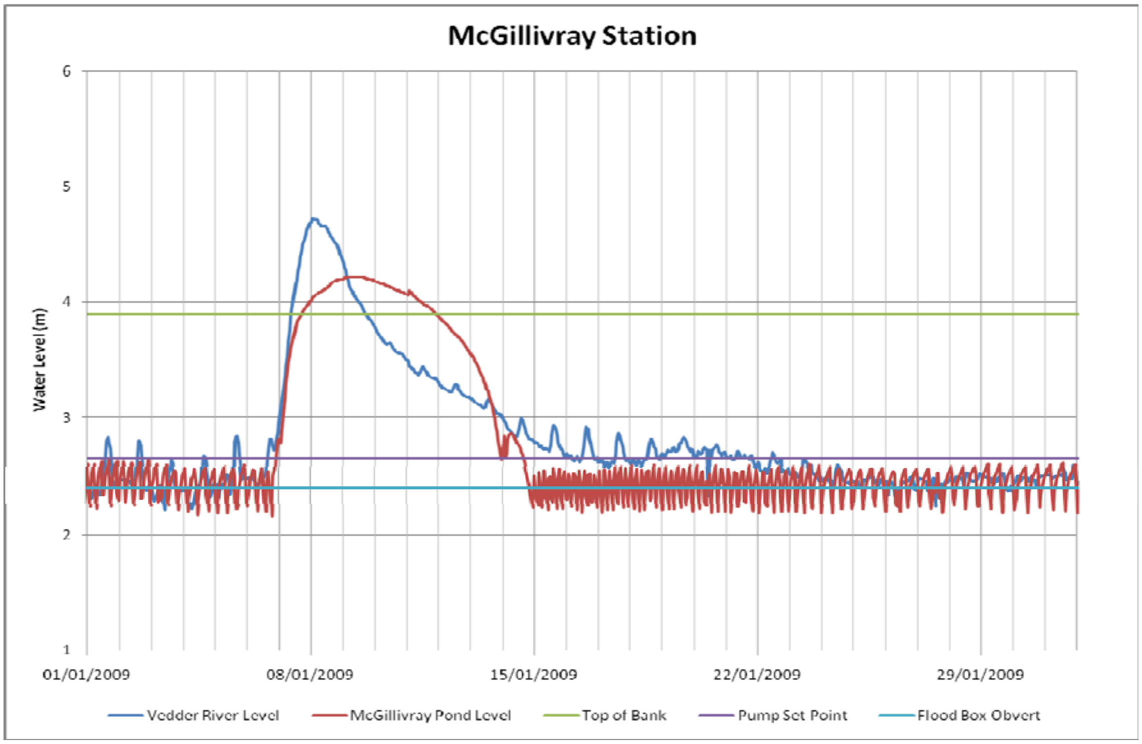
- McGillivray flood box position
- channel cleaning
- increasing channel capacity (widening or deepening)
- increasing culvert capacity
- increase capacity of the pump stations and reviewing set points
- isolation of catchments to optimize available gravity discharges

McGillivray Flood Box Position

During the January 2009 storm event, we understand that the McGillivray flood box was manually closed, and is therefore represented as such in the baseline model calibration for that event. The question has been asked as to whether this box could have been open and whether it would have made a difference to the performance and impacts. As shown in Figure 14A, SCADA information for the McGillivray pump station shows that for the first 3 days of the event, from January 7th to 9th, water levels in the Vedder River were higher than those at the McGillivray pump station. The

Figure 14A

January 2009 Event - Water Levels at McGillivray Station

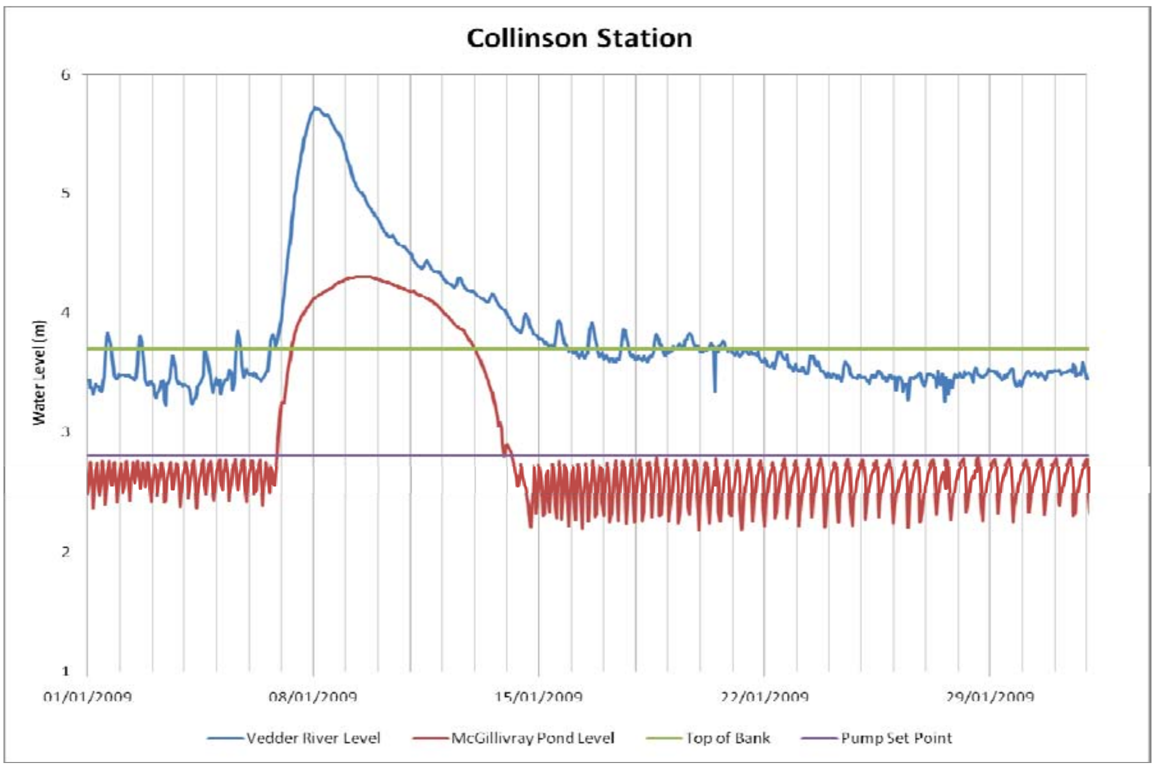


data also indicates that from the 7th to the 10th of January the water levels in the Vedder River were higher than the internal field level (top of bank). As such, a manually closed flood box did not influence the rising limb or peak of the flooding event; however it likely would have assisted in draining the community faster during the recession of the event. The SCADA data was also reviewed to see what the typical water levels are for the Vedder River at the McGillivray pump station during the winter. In reviewing 3 years of winter water levels for the Vedder River (excluding the freshet period), there were 9 separate events where the Vedder River water levels at the McGillivray station rose above the internal field level. Only two of these events resulted in water levels on the pump station inlet side rising above field level. Moreover, it is observed that the pattern indicated by Figure 14A is common; in that water levels in the Vedder River respond similarly, and generally higher, than the internal water levels. As such, in response to local precipitation, the demand on the Vedder River rises consistent with the demand on the Greendale system. There is therefore a high chance of the floodbox capacity being heavily reduced or fully inoperable during a winter storm. Based on this likely occurrence, design analysis in the in the following sections continues to assume this box is closed.

For sensitivity comparison, however, a model comparison was completed for a 100 year winter condition, one with the box open and one with the box closed. The results of this comparison indicated a modest reduction of peak water levels in the vicinity of the station (drop of 0.25 m), but did not significantly lower water levels further upstream in the system (drop of 0.02 m in McGillivray channel at Highway 1 crossing), suggesting that the limiting factor is more associated with conveyance capacity of the channel system rather than the pump station or floodbox.

The Collinson pump station does not currently have a floodbox, nor does the SCADA system record water levels in the Vedder Canal at that location, however by using the SCADA data recorded at the McGillivray pump station and projecting consistent with the slope of the Vedder Canal, an elevation relationship is also estimated for the Collinson pump station, as shown in Figure 14B. The purpose of this comparison was to reaffirm whether there may be any advantage to having a floodbox at the Collinson pump station.

Figure 14B
January 2009 Event – Water Levels at Collinson Station



As shown in the above figure, the estimated water level in the Vedder Canal at the location of the Collinson pump station is significantly higher than on the land side of the dyke. If fact, the estimated base flow water level in the Vedder Canal is generally equal to the field level. As such, there is no benefit under any circumstance to having a floodbox at the Collinson location.

Channel Cleaning

During the consultation process, it was learned that the City has an ongoing annual ditch cleaning program, dating back as far as 1948 when the dykes were built. It is understood that the current program involves the trunk system and tributaries. The process includes excavating silt from the channel bottom and clearing of vegetation from the bottom and side of the channel. In last three years, a total of 41,130 m of ditches was cleaned and another 21,201 m of ditches is scheduled to be cleaned in 2009. In order to understand how the ditch cleaning may influence the overall drainage system performance, independent scenarios were modeled using varied channel roughness values. It should be noted, however, that this assessment only considered channel bank roughness and did not investigate acute barriers or blockages within the system. Such blockages and barriers may certainly be significant, however cannot be practically analyzed herein, as the possibilities are endless. It should go without saying that the system needs to be fee of barriers and blockages to optimize performance. It is also assumed that sediment removal returns the channel to its original geometry, rather than over-excavation. Testing the significance of making the channel larger was done and discussed separately.

Assuming they are reasonably clean, the simulations reveal that the geometry of existing channels generally have capacity to convey the 1:10 year event flows without breaching, consistent with their established performance criteria. As such, the roughness, or cleanliness of the channel system, is very significant for events up to the 1:10 year level; however have diminishing significance for events beyond that level. Cleanliness will affect how frequently the channels breach their banks, but will not significantly change floodplain risk once the magnitude of the event exceeds the geometric capacity of the channel.

Increasing Channel Capacity

As expected, model simulations of wider and deeper channels showed larger benefit for all storm events than ditch cleaning alone. Both approaches (widening / deepening) can provide similar results, as carrying capacity is directly related to the cross sectional area of the channel. In order to prevent over steepened banks, deepening of channels can generally not be done without widening, whereas widening can be done without deepening. However, the advantage of deepening is that it can offer increased capacity with a smaller channel footprint than widening alone. With that said, while there is evidently an opportunity to increase conveyance capacity through channel enlargement, in order to provide substantive improvement for large storm events, channel works would need to be extensive and widespread through the system. As stated earlier, the general findings are that the current channel system capacity is reasonably consistent at the 1:10 year level, and satisfies base criterion. The magnitude of the 1:10 year event is approximately 50% of the 1:100 year event, and 30% of the January 2009 event. In order to significantly increase channel capacity and reduce the dependency on the floodplain, channel cross sections would need to be increased by the reciprocal amount; 2 times larger for the 1:100 year event, and 3.5 times larger for the January 2009. Should this approach be taken, all culverts and pump stations would also need to be upgraded by a comparable amount, which would also require the station structures to be replaced. For the stations alone, if a conventional configuration was chosen, the ball park cost would be in the order of $3.5M to $4M per station, whereas Archimedes screw pump stations would be in the $5M to $6M range per station. Order of magnitude, the total cost for this approach would be in the tens of millions of dollars when also considering the channel and culvert implications. As such, from both a cost and logistic perspective, this overall approach would be very extensive and generally not considered practical, therefore not pursued further at this time. While not technically complicated, its significant implication to land and environment would require a comprehensive consultation process.

Increasing Culvert Capacity

Ideally, culvert capacity will be equivalent to the capacity of channel. Analysis has identified a number of culverts that do have capacities below, some significantly below, that of the channel, and are therefore candidates for upgrading. However, overall sensitivity analysis on culverts showed generally local effects and modest improvement overall, with diminishing improvement for large events. This is because at a certain point channel capacity governs.

Increasing Capacity of Pump Stations

It is determined that the existing pump stations have adequate structural and outfall capacity to accommodate upgrading to match increased duty points, therefore upgrading of the existing stations is one possible approach. For this assessment, a common set of pump station upgrade works has been indentified to make best use of the existing station structures, however duty points will vary depending on incoming flow and head conditions. It was found that water levels in the lower reaches of the system near the stations could be dramatically lowered, but the mid and upper reaches of the system were much more dependent on the channel capacity and, therefore, their ability to deliver water. As discussed above, with the channel having a capacity approximately equal to the 1:10 year level, increased pump station capacity cannot prevent activation and dependency of the floodplain, but may assist in reducing the magnitude of the flood, particularly in the lower zones of the floodplain.

Catchment Isolation

The Wilson Slough system was described in Section 3. Flows from the areas east of Hopedale Road mostly discharges into the Fraser River during the winter period, however is not fully isolated from pumped systems in its current configuration. Initial testing considered complete separation of the area east of Hopedale road in an attempt to maximize the utilization of the gravity outfall to the Fraser River and reduce the area draining to the pump stations during not freshet condition. Model results show that in general, separation of the east catchment offered potential benefit in reducing the severity of flooding to the west. It is recognized that freshet conditions could not remove dependency on the pumped systems all together, for this approach would need to consider seasonal differences.

Topography

Topography and the relative elevation of building and infrastructure has perhaps the most significant influence on risk and consequence once the floodplain activates. At this time, no particularly sensitivity analysis has been conducted on the floodplain shape itself. A review of the LIDAR topographic data set indicates that the floodplain vulnerabilities are widespread; therefore there is no small number of locations where topographic regrading could benefit the broad community. Similarly, there are no obvious locations that could be designated as a flood storage zone. Laser grading of agricultural fields is common practice done by individual farm owners to improve local drainage; done to improve crop production. This work is often done in combination with installation of tile drains and ditching. While the City should support property owners completing such work, it is not considered a practical approach to overall floodplain management, particularly for more extreme design storm events. Re-contouring of farmland may or may not be of interest to DFO depending on the proximity of the proposed re-contouring to fish-bearing streams as well as the possibility of alienating the floodplain.

Comprehensive dyking of the internal trunk drainage network, such as done is the Nicomekl River and Serpentine River floodplains in the City of Surrey, is a possibility, however not pursued at this time. Developing a strategy around internal dyking and pumping is technically complex, and would require an extensive, comprehensive study and consultative process in itself to properly evaluate this scenario. Dyking along the internal watercourse network will be a more difficult option from the agency perspective. There are various agencies with an interest in dyking, including the Ministry of Environment through the Dike Maintenance Act, the Ministry of Environment, and Environmental Stewardship Division, and DFO. Through past discussions with DFO, they have concerns with the alienation of the floodplain. In addition, the land requirements, cost, and approval implications of an internal dyking / pumping approach would be very extensive, as would the long term operation and maintenance implications. If ultimately deemed feasible at all, the anticipated order of magnitude cost would be tens of millions of dollars.

7.0 FLOOD MANAGEMENT OPTIONS

Based on the findings of the initial sensitivity analysis, a series of options, each representing a collection of infrastructure works, has been developed for consideration. As noted in the previous section, the costs and logistics of comprehensive internal dyking systems and / or increasing channel capacity to eliminate dependency on the floodplain are very extensive, and would require independent study and consultation; therefore, have not been considered further at this time. The philosophy taken for this study has been to optimize and make best use of existing infrastructure in the near term, provided a solution can be found to satisfy established criteria and standards. The approach taken in each of the following options is similar; generally consisting of pump station and culvert upgrades, but in some cases also looking at channel upgrades and diversions.

A level of judgment had to be used in identifying culvert upgrades, recognizing that overall conveyance performance is also highly governed by the channels themselves. Ideally the culverts are no more restrictive than the channel; however this will realistically not be achievable in all cases. The general approach of the assessment was to first set a threshold limit of 0.2 meters of head loss through culverts. Using this value as an initial target, new culvert sizes were determined. For culverts where the recommended size was less than 20% larger than the existing size, some sensitivity testing was done to better assess whether these minor upgrades had a significant influence on overall performance or not. In general, they did not, therefore the final approach was to suggest culvert upgrades only if they require a cross sectional area more than 20% larger than the current size.

For the pumps stations, the existing structures have potential for increased duty points by changing pumps, motors, impellers, and electronics. A common set of equipment has been identified for all options, providing a constant potential capacity. However, because actual flow rates are dependent on incoming flows and head levels, performance will vary under different conditions. The specifics of the recommended upgrades are discussed in subsequent sections and the appendix, while the anticipated duty points for the stations are reported below for each option.

To begin, each option is introduced and discussed independently. A comparative summary is then provided at the end of this section, along with budgetary costs.

Evaluating the performance of each option has been looked at in two ways, recognizing the important influence the flat topography plays in this watershed. One way has been to look at performance horizontally by reporting extent of area potentially flooded, along with the associated vulnerabilities or impacts, such as homes, outbuilding, and roads. The second way is to look at performance vertically by reporting the predicted water levels directly within the conveyance system, along with the estimated flood volume or depth changes. For each of the options reviewed, a comprehensive set of output has been compiled to offer a look at performance both ways.

Performance has been assessed under all four design events, again being the 10 and 25 year storms under a freshet condition, and the 100 year and January 2009 events during a winter condition when flood gate capacity is available. Summary tables reporting estimated impacts, along with a complimentary graphic that shows the extent and location in graphic form. On that same graphic, physical infrastructure upgrades for the option are also highlighted. To review the hydraulic performance of the conveyance system directly, a series of tables are provided in Appendix D that compare the maximum water elevation at the upstream end of all culverts and pump stations to the corresponding baseline condition. A key plan has been provided as Figure 15 to identify the location of the Culvert ID and Channel Reach listed in the tables.

The magnitude of flooding is without question influenced by the hydraulic grade line in the conveyance system; however analysis demonstrates that the generally flat topography has a more significant role in the ultimate extent of area affected. In other words, the extent of the area flooded is relatively similar regardless of the flooding depth. As discussed in Section 5.1 Model Development, the estimated area flooded is based on processing the topographic data in 40 meter x 40 meter grid cells, which does not significantly affect the hydraulic performance results (i.e. predicted water levels and conveyance), but is a partial contributor to a margin of error in the computed extent of flooded area. The computational processing of the LIDAR topographic data at a much higher resolution is excessively constraining.

Similar to the baseline scenarios discussed in Section 6, flooding is defined by a water depth threshold of 0.05 meters and deeper.

7.1 Option 1

Synopsis: Culvert upgrades to satisfy their current 10 year conveyance criterion, and pump stations operating at their current 25 year criterion.

- Number of culverts upgraded: 48
- Pump station duty points (total station):
    - Collinson pump station: 4.0 m3/s
    - McGillivray pump station: 5.6 m3/s

A performance summary is presented in Section 7.10, Figure 16, and water elevations reported in Appendix D.

7.2 Option 1B

Synopsis: Option 1 plus new channel to supplement McGillivray Creek and Lewis Slough.

- Number of culverts upgraded:48
- New channel ROW area: 4.39 hectares
- Pump station duty points (total station):
  - Collinson pump station: 4.0 m3/s
  - McGillivray pump station: 5.6 m3/s

McGillivray Creek is perhaps the most significant watercourse within the system, servicing the majority of the study area, which includes the most densely populated areas in the general vicinity of South Sumas Road and Sumas Prairie Road. From Sumas Prairie Road to Chadsey Road, McGillivray Creek significantly meanders, dramatically increasing the flow path length to the pump station and, therefore, decreasing overall performance. A new channel to supplement the capacity of McGillivray Creek in a "short circuit" fashion has been tested by this sub-option "B". The model was tested using channel bottom widths of 2, 3 and 4 meters. Predicted water levels through the area dropped significantly with the addition of the 2 meter wide base channel; however performance did not significantly gain with the wider channels. As such, only the 2 meter wide base channel is brought forward for consideration. The suggested alignment, profile and typical cross section of this channel is shown in Figure 17A. The suggesting aligned is preliminary at this stage and would be subject to negotiations with property owners and details field reconnaissance and design. In addition, the intended function of this channel is to supplement capacity of the existing system and not take away base flow from McGillivray Creek. Specific profiles and flow controls will need to be reviewed in concert with discussions with DFO to agree on operational and habitat restoration requirements of the new channel. At this point in time, the suggested alignment straddles property lines. Based on the available LIDAR topographic information, the estimated channel extent (top of bank to top of bank area) is shown and land area computed. In addition, a suggested easement setback of 1 meter on the north side, and a 4 meters setback on the south side is shown; with the 4 meter setback being for long term maintenance access. Again, these are subject to negotiation. A new crossing with 2-1500 mm diameter (or equivalent area) culvert crossing of Sumas Prairie Road would also be required.

Similarly, a supplemental channel is also assessed for Lewis Slough as defined in Figure 17B. Very similar principles, as described above for the McGillivray channel, are applied. It too is subject to detailed site review and negotiations with property owners and approving authorities (ie. DFO).

A performance summary is presented in Section 7.10, Figure 18, and water elevations reported in Appendix D.

7.3 Option 2

Synopsis: Culvert upgrades to maximize convey performance for a 100 year storm event and pumps stations operating at their current 25 year criterion.

- Number of culverts upgraded: 71
- Pump station duty points (total station):
  - Collinson pump station: 4.4 m3/s
  - McGillivray pump station: 6.2 m3/s

The premise of this option was in further increase the conveyance of the culverts system, with focus on the 100 year criteria, which also ensure the culverts are not a constraint beyond the channels themselves. However, the pump stations are still only sized to the 25 year criteria. This option was tested to see if water could be moved through the conveyance system at a faster rate to improve performance in the upper reaches without significantly increasing risk in the vicinity of the pump stations.

A performance summary is presented in Section 7.10, Figure 19, and water elevations reported in Appendix D.

7.4 Option 3

Synopsis: Culvert and pump station upgrade to maximize conveyance for a 100 year event.

- Number of culverts upgraded: 71
- Pump station duty points (total station):
  - Collinson pump station: 5.2 m3/s
  - McGillivray pump station: 6.4 m3/s

The premise of this option was to further increase size of the pumps stations and culverts beyond the general capacity of the channels. Pump station sizing is based on the 100 year flow that can be delivered by the conveyance system, not the 100 year flow should all conveyance restrictions be eliminated.

A performance summary is presented in Section 7.10, Figure 20, and water elevations reported in Appendix D.

7.5 Option 3B

Synopsis: Option 3 plus new channel to supplement McGillivray Creek and Lewis Slough.

- Number of culverts upgraded: 71
- New channel ROW area: 4.39 hectares
- Pump station duty points (total station):
  - Collinson pump station: 5.2 m3/s
  - McGillivray pump station: 6.4 m3/s

Similar to Option 1B, this sub-option includes the new channel to supplement the capacity of McGillivray Creek west of Sumas Prairie Road, and Lewis Slough north of Keith Wilson Road. Refer to Figure 17A and 17B for additional details. All other aspects of this sub-option remain the same as described for Option 3.

A performance summary is presented in Section 7.10, Figure 21, and water elevations reported in Appendix D.

7.6 Option 4

Synopsis: Catchment separation east of Hopedale Road, channel upgrade on the east side of Hopedale Road, various culvert improvements throughout watershed to the 100 year criterion, and pump stations operating to their current 25 year criterion.

- Number of culverts upgraded: 59
- Increased channel ROW area: Unknown (subject to investigation to resolve alignment discrepancies between the cadastral and topographic data)
- Pump station duty points (total station):
  - Collinson pump station: 3.8 m3/s
  - McGillivray pump station: 5.8 m3/s

For this option, the premise is to force as much water as possible north through the Wilson Slough during the more significant winter storm conditions, but when the floodbox capacity is available. Similar to how it works today, Hopedale Road would act as the barrier with all drainage interconnects crossing Hopedale Road and at Miller Slough being closed during the major winter storms. The would have to be open during the freshet period when the Wilson floodbox is closed. The hydraulic assessment indicates that this would impact the conveyance performance of not only some culverts, but the channel on the east side of Hopedale Road. As such, channel upgrades have been conceptually identified for this Option, as shown in Figure 22 and 23. Similar to the channel described for Option 1B, the estimated top of bank location and land area is based on the available LIDAR topographic data, and would be subject to property negotiations and detailed design. Also, there are alignment discrepancies between the cadastral base and topographic data sets that will require further investigation to resolve. As such property impacts and land acquisition areas cannot be determined at this stage. Further investigation will be required to confirm viability and costs of these channels. However, because the channel is accessible from the road, only a 1 meter permanent easement is suggested on the east side of the channel.

Special consideration was given to the highway and rail crossings, given their significance. As indicated by the hydraulic performance tables, the upstream water levels at these crossing are expected to rise marginally, in the order of 0.2 m. However, at this level of assessment, it does not appear to result in additional impacts to warrant upgrade of these crossings. Therefore, at this stage, upgrade crossings to the highway and rail are not suggested.

The 10 year and 25 year events are believed to have a reasonable chance of occurrence during a freshet period, therefore for these two events, it is assumed that water from the Wilson Slough basin will still need to convey west to the McGillivray system. Results are presented accordingly, and hence why the pattern of comparative water levels in the accompanying tables is not consistent across all four events.

A performance summary is presented in Section 7.10, Figure 24, and water elevations reported in Appendix D.

7.7 Option 5

Synopsis: Catchment separation east of Hopedale Road, channel upgrade on the east side of Hopedale Road, various culvert improvements throughout watershed to the 100 year criterion, and pump upgrades to the 100 year criterion.

- Number of culverts upgraded: 71
- Increased channel ROW area: Unknown (subject to investigation to resolve alignment discrepancies between the cadastral and topographic data)
- Pump station duty points (total station):
    - Collinson pump station: 5.2 m3/s
    - McGillivray pump station: 6.4 m3/s

This option is similar to option 4; however culvert upgrades are more extensive, focusing on the 100 year criteria. Pump station sizing is based on the 100 year flow that can be delivered by the conveyance system, not the 100 year flow should all conveyance restrictions be eliminated.

A performance summary is presented in Section 7.10, Figure 25, and water elevations reported in Appendix D.

7.8 Option 6

Synopsis: This option was conceived following a review of the analytical results of Options 1 through 5 based on the observation that the previous five options did not have the ability to offer performance improvement in the vicinity of Chadsey Road and South Sumas Road because of the excessive flat and long flow path to either of the existing pump stations. As such, Option 6 represents all supplemental channel and culvert improvements as described in Option 1B, however rather than upgrade the two existing pump stations, a single new Archimedes screw pump station would supplement the two existing stations. For this option, no upgrades have been applied to the existing stations. In this case the pump station capacity was determined through sensitivity analysis to optimize performance under the 1:100 year and January 2009 events. As such, criteria for the new pump station and supplemental channels can be considered 1:100 year.

- Number of culverts upgraded: 54
- Increased channel ROW area: 6.31 ha
- Pump station duty points (single new station):
  - New pump station: 8 m3/s

The suggested location of the pump station and the alignment of the connecting channel is shown in Figure 26. A performance summary is presented in Section 7.10, Figure 27, and water elevations reported in Appendix D.

7.9 Option 6B

Synopsis: Same set of works as described for Option 6, however culverts were upsized based on 100 year criteria, similar to the approach described for Option 3B.

- Number of culverts upgraded: 71
- Increased channel ROW area: 6.31 ha
- Pump station duty points (single new station):
  - New pump station: 8 m3/s

A performance summary is presented in Section 7.10, Figure 28, and water elevations reported in Appendix D.

7.10 Evaluation of Options

A evaluation summary tables is provided below. Details of the pump station upgrades and cost estimates for all works are provided in Appendix C. Planning level costs include contingencies, engineering, and construction administration, but exclude GST.

	Options Evaluation Summary								
	1	1B	2	3	3B	4	5	6	6B
Pump Station Cost ($M)	4.9	4.9	4.9	4.9	4.9	4.9	4.9	7.9*	7.9*
Design criteria	1:25	1:25	1:25	1:100	1:100	1:25	1:100	1:100	1:100
Culvert Cost ($M)	5.5	5.5	8.8	8.8	8.8	8	8.8	6.5	8.8
Design criteria	1:10	1:10	1:100	1:100	1:100	1:100	1:100	1:10	1:100
Channel / Land Cost ($M)	0	2.6	0	0	2.6	1.8*	1.8*	2.6	2.6
Total Cost ($M)	**10.4**	**13**	**13.7**	**13.7**	**16.3**	**14.7***	**15.5***	**17**	**19.3**
Total pump station duty point (Collinson / McGillivray) (m3/s)								[new station]	
1:10 Year Event	4.0/5.6	4.0/5.6	4.4/6.2	5.2/6.4	5.2/6.4	3.8/5.8	5.2/6.4	8	8
1:25 Year Event	4.0/5.6	4.0/5.6	4.4/6.2	5.2/6.4	5.2/6.4	3.8/5.8	5.2/6.4	8	8
1:100 Year Event	4.0/5.6	4.0/5.6	4.4/6.2	5.2/6.4	5.2/6.4	3.8/5.8	5.2/6.4	8	8
January 2009 Event	4.0/5.6	4.0/5.6	4.4/6.2	5.2/6.4	5.2/6.4	3.8/5.8	5.2/6.4	8	8
Total number of culverts upgraded									
1:10 Year Event	48	48	71	71	71	59	71	54	71
1:25 Year Event	48	48	71	71	71	59	71	54	71
1:100 Year Event	48	48	71	71	71	59	71	54	71
January 2009 Event	48	48	71	71	71	59	71	54	71
Area of new channel ROW (m2)									
1:10 Year Event	0	43,900	0	0	43,900	Unknown	Unknown	63,100	63,100
1:25 Year Event	0	43,900	0	0	43,900	Unknown	Unknown	63,100	63,100
1:100 Year Event	0	43,900	0	0	43,900	Unknown	Unknown	63,100	63,100
January 2009 Event	0	43,900	0	0	43,900	Unknown	Unknown	63,100	63,100
Estimated vulnerable homes									
1:10 Year Event	1	1	1	1	1	1	1	1	1
1:25 Year Event	5	5	5	5	5	5	5	5	5
1:100 Year Event	12	12	12	12	12	8	8	8	8
January 2009 Event	24	24	23	18	19	19	14	15	15
Estimated vulnerable out buildings									
1:10 Year Event	27	28	28	28	28	26	26	28	28
1:25 Year Event	77	75	77	77	75	75	75	77	77
1:100 Year Event	111	108	110	107	105	108	105	104	103
January 2009 Event	133	128	128	121	115	122	110	109	108
Estimated vulnerable roads (m2)									
1:10 Year Event	0	0	0	0	0	0	0	0	0
1:25 Year Event	14	14	14	14	14	14	14	14	14
1:100 Year Event	107	107	107	107	107	165	162	88	88
January 2009 Event	530	434	477	354	355	398	385	374	398
Estimated max. flood area (ha)									
1:10 Year Event	307	311	307	307	308	304	304	306	304
1:25 Year Event	610	606	607	604	603	597	595	594	591
1:100 Year Event	771	765	764	757	753	755	745	744	739
January 2009 Event	832	830	816	791	791	801	769	767	759
Estimated max. flood volume (ha.m)									
1:10 Year Event	53	54	54	54	54	53	53	53	53
1:25 Year Event	117	116	116	115	116	114	113	111	110
1:100 Year Event	158	156	154	151	151	154	151	147	145
January 2009 Event	223	222	215	201	202	207	195	186	184

* Note that costs for Option 4 and 5 does not include land acquisition costs or potential building impact as they are immeasurable at this time without more comprehensive study. They are expected to be significant. Also, the pump station cost of $7.9M for Option 6 and 6B includes the required connecting channel to McGillivray Creek.

The results demonstrate and verify that local grading and general topography play a very significant role in the overall ability of land to drain, and therefore, is a significant contributor to building vulnerability. To varying degrees, the detailed tables in Appendix D, which report water levels for each option, demonstrate a general drop in the conveyance system water levels for all options, however, many of the options do not demonstrate a significant drop in flooded area or a reduction in vulnerable homes and out building when reviewed against the baseline performance results presented on page 25. This highlights the criticality of overall topography and grading of the watershed. However, by dropping the water levels in the conveyance system, opportunity increases and additional benefits can be gained by improving site grading and local drainages around vulnerable buildings. Based on the results, it is estimated that at least 5 homes and 75 out buildings are vulnerable primarily or solely based on local grading and site conditions alone. This is the number of buildings vulnerable during more frequent (< 1:25 year) conditions, and no mitigation option has the ability to reduce these numbers even with lowered water levels in the conveyance system. For the more extreme events (1:100 year and January 2009) a reduction of impacts from baseline conditions becomes more apparent, albeit more for some options than others.

Before conclusion can be reached on the vulnerability of buildings, site specific survey will be required to confirm the building floor elevation relative to the predicted flood levels. Should it be confirmed that these building are in fact at risk, a practical approach would be to protect them with localized dyking or to raise the buildings; the costs of which would need to be investigated individually for each building.

With respect to pump stations, Options 1 through 5 involve significant reinvestment in the existing two stations, whereas Option 6 and 6B involve investment in a new station, while maintaining the two existing stations. There is expressed concern for the impact to fish posed by the current propeller axial pumps in the two existing stations. The most fish friendly approach would be to convert the stations fully to an Archimedes screw type, however, as discussed in previous sections, the cost for full replacement of these stations to Archimedes screw type will be dramatically higher than finding a solution to modify the existing structures. Therefore, there is a desire to select an appropriate design that takes into consideration both the environmental benefits and cost effectiveness. If consideration is given to upgrading the existing pump stations, one alternate approach is to explore screw axial flow pumps. These pumps employ a screw in place of the propeller and are commonly employed at fish farms to transfer fish and are also used in high solids applications at sewage treatment plants. They are more expensive than the propeller axial flow pumps but can be employed in a similarly designed structure.

The risk to fish is related to a number of variables including: frequency of the pump operation, flow passed, season and fish concentration during pumping. For instance, if most routine storms could be handled by a fish friendly jockey pump (Archimedes Screw or Screw Axial Flow Pump), the higher return period more intense storms could be pumped with a non fish friendly, but lower cost propeller pump without a screen. An arrangement such as this could be considered a hybrid type of pump station. This could have the effect of substantially reducing the frequency of fish encounter with a propeller pump when compared to a propeller only station. With a hybrid station it may be possible to ascertain the reduction in encounter frequency with the propeller pump. For additional consideration, comments received from the City's Public Works Department indicate that the existing screens have been problematic and may be too small to accommodate increased pumping capacity. A review indicates that the intakes are sufficient from a hydraulic capacity perspective, however can be problematic from a debris removal perspective. Increased pumping rates may increase the movement of debris and, therefore, maintenance of the intakes to ensure optimum performance. Maintenance associated with changing the intake screens to meet fish protection demands would likely be unmanageable. Lastly, upgrading the existing stations would require new back up gen-set power supply and external kiosks.

Fundamentally, this study finds the opportunity to upgrade the existing stations and offers preliminary information and budget costs for consideration. However, identifying the specific configuration and equipment to optimize all issues requires more extensive review beyond the scope of this study. Should upgrades to the existing pump stations be selected, it is recommended that the City conduct a dedicated pre-design study of the stations through the engagement of equipment suppliers. As part of this, and as discussed in Section 8.3, liaison and a Memorandum of Understanding with DFO is viewed as critical, because meeting environmental needs is likely a significant driver on the final pump station design and costs associated with upgrading the existing stations.

For options involving reinvestment in the existing stations (Options 1 through 5), Option 3B offers the greatest long term potential benefit and is most practically implemented. While Option 4 and 5 appear to offer similar performance, there would be significant impacts to some properties along Hopedale Road in constructing the improved channel. The land acquisition and property impact issues make these less favourable options to Option 3B.

Investing in a new pump station, as represented by Option 6 and 6B, offers the best all around performance of the system, and better meets environmental objectives. However, the challenge with these options is that a very significant investment is required ($7.9 Million) in land acquisition, the pump station, and the connecting channel to McGillivray Creek before any benefit is realized. While some phasing may be possible, a commitment to the new station does not offer the same financial flexibility for gradual implementation that is offered by the other options. Short of an external funding source, near term implementation of these options may not be possible. Except for a few isolated locations, there is no marked different in performance between Option 6 and 6B. As such, if a new station option was selected, there does not appear to be value in fully adopting Option 6B, leaving Option 6 as the favourable choice.

All things considered, if commitment is to raise the level of service beyond base levels, Option 6 and Option 3B are the suggested short list contenders. Using the 1:100 year and January 2009 events as measures, the impact reductions from baseline conditions for the two options are as follows:

	1:100 year		January 2009	
	Option 6	Option 3B	Option 6	Option 3B
Reduction in vulnerable homes	33%	0%	45%	30%
Reduction in vulnerable out buildings	8%	7%	30%	23%
Reduction of flood area	5%	4%	12%	9%
Reduction of flood volume	11%	10%	22%	16%

In addition to the above table, Figure 29 presents a hydraulic grade line profile comparison of these two options. This figure highlights the topographic and channel conveyance challenge of the McGillivray Creek system. As shown, the lower portion of McGillivray Creek has a channel segment approximately six kilometers in length with insufficient topographic fall. The channel invert over this long length varies by only 0.4 meters, ranging between 1.6 and 2.0 meters in elevation. The drop that does exist occurs in a localized spot, separating two long flat sections of channel.

General commentary on the effect each option has on system water levels under varying design events is as follows:

1:10 year Event – Both options are virtually the same and essentially mimic the baseline condition. This verifies that the existing system is generally sufficient for the 1:10 year design event.

1:100 year Event - In many areas the performance of the two options is considered equal, and in other areas not. Particularly in the Lewis Slough system and the upper reaches of the McGillivray Creek system, Option 3B offers slightly lower water levels than Option 6, but both offer significant improvement with water levels dropping between approximately 0.4 m to 1.5 m in many areas. In the mid and lower reaches of the McGillivray system, Option 6 offers superior performance. This is because the positioning of the new pump station provides an alternate outlet and diminishes the load on the critical flat section of McGillivray Creek. It is highlighted that under the 1:100 year event, Option 3B shows the potential for a slight increase in peak water levels through the mid reaches of the McGillivray Creek system. This is due to the fact that water is being more rapidly drained from the upper reaches of the system, thereby requiring the lower critical portion of McGillivray Creek to surcharge. The supplemental channel identified in Figure 17A does offer a significant benefit and helps minimize potential negative impact, but cannot fully resolve the capacity challenge of the existing creek. Additional pumping at the existing McGillivray pump station will also not overcome this constraint because the limitation is the ability of the channel to deliver water to the pump station. Altering the channel profile by dredging would likely offer a hydraulic benefit, however there would be significant environmental approval barriers, as well as significant additional cost. If at all possible, this potential action lends itself better to Option 6 than Option 3B because the dredge length for Option 6 is 35% shorter and would not require replacement of the Highway 1 crossing, however would still impact 3.5 km of creek channel and would alter the low flow patterns of the system, which would be a significant fisheries habitat issue. This specific limitation, and how it influences the implementation process, is discussed further in Section 9 – Implementation.

January 2009 Event – Both options offer an improvement, and in many areas a very significant improvement, over the baseline condition. The majority of the systems could expect to see peak water levels in the order of 0.5 m to 1.5 m lower under this event, should it happen again. Again, while a benefit is realized, it is modest through the mid reaches of the McGillivray Creek system because of the topographic limitations described above.

Another aspect to consider is the current state and condition of the existing pump stations. It is recognized that they are aging and will need reinvestment at some time. As suggested previously, a more comprehensive review of current equipment performance and condition is warranted. Even with a commitment to a new pump station through Option 6, it will be important that the two existing pump stations be operated and maintained long term. As such, reinvestment at some time will be necessary regardless.

The last aspect under consideration in formulating a recommendation is other potential sources of flood risk. As discussed in Section 3.2, analysis conducted for the January 2009 event has estimated the impact from that event to be equivalent to a conventional storm upwards of a 1:200 year return period. As discussed in Section 4.2, the current floodplain bylaw is based on a 1:200 year flood risk from external sources, including the Fraser River, Chilliwack River, Vedder River and Vedder Canal. The flood construction level (FCL) stated in Schedule A of the Floodplain bylaw ranges from a low of 10.9 meters at the Vedder Canal to about 11.1 meters towards the east limit of the Greendale community. As shown in Figure 5, the ground elevation for the vast majority of the Greendale community ranges from about 3.2 meters to 11 meters.

For this same area, and by also applying a freeboard height of 0.6 meters, flood construction levels (FCL) for the January 2009 would range from about 4.3 to 11.6 meters elevation.

8.0 RECOMMENDATIONS

The relative elevation and topography of Greendale, and profile of McGillivray Creek in particular, makes it a very difficult area to fully flood proof. Significant investment is required to significantly raise the level of protection above base levels. We understand from the City that the financial cost is a very significant factor, therefore achieving a practical, gradual implementation program is critical to success. Based on this, and the preceding sections, it is recommended that the near term action be taken towards Option 3B, with longer term view towards implementing a new supplemental pump station should financial resources become available. Option 3B provides a more gradual implementation program which is more financially viable at this time. It is recommended that the City lobby senior levels of government and perhaps consider other financial mechanisms such as a utility fee or a local service area charge with attempt to secure additional funding sources. Should that be successful, Option 6 would be advantageous to the mid and lower portions of the McGillivray Creek system. Further discussion is provided in Section 9 – Implementation.

In addition, the capital works need to be supported by a proactive O&M program to ensure that performance is consistently optimized. Further exploration is recommended to address the select number of highly vulnerable homes, and the City should encourage land owners who wish to improve grading and drainage on their own property. Lastly, all future building and infrastructure should be installed at a minimum of 0.6 meters above the water elevations documented herein for the January 2009 event (refer to Option 3B in Appendix D), or those specified by the Floodplain Bylaw.

Further details on the recommendations are as follows. A table describing the full suite of recommended works is provided at the front of Appendix C.

8.1 Capital Infrastructure Plan

8.1.1 Design Criterion

In consideration of costs, benefits, and established criterion, key infrastructure has been ultimately sized to maximize performance potential to the following criteria:

- Upgrades to existing pump stations 1:100 year
- New culverts 1:100 year
- New McGillivray and Lewis Slough channels 1:100 year (activate only in higher flows)

8.1.2 Pump Station

It is understood that there has been no efficiency testing done on the existing pump stations to determine if they are truly performing to their original design set points. Model calibration suggests that the stations may not currently perform at their optimum, or intended duty point. While testing of the current equipment could be done, given that both stations are some 30 years old without having major work done, and they are likely nearing the end of their life cycle. As such, recognizing the need to increase capacity, no particular recommendation is made with respect to the current pumps or motors. Detailed information describing the pump station upgrades and costs is provided in the appendix, with a synopsis of the recommendation as follows.

The existing station structures due offer the opportunity for increased capacity. Recommended pump geometry and design flow rates conform to limits set out by the Hydraulic Institute (HI) Standard for Pump Intake Design, therefore no structural upgrades are required to the existing stations, however, a wet well partitioning wall will be required. The stations have not been investigated for structural condition at this time. As such, no allowance for seismic or structural upgrades has been included herein. It is also assumed that the existing roof access hatches can accommodate the increased pump and motor size.

It is determined that the existing pump stations have adequate structural and outfall capacity to accommodate upgrading to match increased duty points. The mechanical upgrades proposed for the station include replacing the existing pumps and related internal piping, motors, controls, back-up power supply, and electrical systems; details of which are specified in the appendix. The recommended changes will significantly boost the potential capacity of the station, however recognizing that performance is dependent on incoming flows and inlet / outlet head conditions, performance will vary. The table below presents the maximum potential of the station, and the anticipated duty points for the implementation of Option 3B. VFD's are to be used to optimize performance for varying inflow / head conditions.

	McGillivray		**Collinson**	
	Total Station Flow (m^3/s)	TDH (m)	Total Station Flow (m^3/s)	TDH (m)
Total potential capacity (freshet / winter)	6.2 / 7.7	10 / 6	5.3 / 5.8	12 / 9.9
Anticipated operating point for Option 3B (freshet / winter)	6.4*	10.1 /5.4	5.2*	11.7 / 8.7

* - sustained pump rate by use of VFD.

No upgrade to the outlet structure or discharge piping through the dike are required for hydraulic reasons, however, their condition should be reviewed early in the design stage to determine if structural upgrades are required.

The proposed motors present a significant increase in horsepower over the existing motors. Upgrades to the existing electrical service at both stations are required to accommodate the increased motor size. The electrical upgrades include:

- Upgrading the existing 480 Volt 3 Phase electrical services to 600 Volt 3 Phase service with 1000kVA transformer
- The installation of VFD's and associated switchgear
- Construction of an electrical kiosk to accommodate the additional electrical infrastructure, including new backup gen-sets at each station.

It is recommended that a comprehensive pre-design review be conducted, during which screw axial flow pumps should be considered and explored with equipment suppliers, along with a review of the inlet screens and their debris management systems to ensure sufficient inlet capacity.

The cost estimate assumes that the electrical service upgrade can be supplied from the same connection point to the grid and that the electrical kiosk can be constructed on the City land adjacent to the pump station without any significant earthworks. The estimated cost of upgrading the stations pumps, mechanical and electrical works, is $1.6M for the McGillivray and $1.7M for the Collinson, excluding contingency, engineering, and GST. The total estimated cost for both stations, including engineering and contingency, but excluding GST, is $4.9M. Refer to the appendix for cost details. This cost does not include any environmental or structural modification that may be discovered through the completion of the comprehensive pre-design review.

8.1.3 Upgrade Culverts

A total of 71 culverts have been identified for upgrade or replacement. Specifics on the implementation process are provided in Section 9 - Implementation. At this time, recommended culvert upgrades are selected based on the conveyance needs of the crossing. Depending on the specific condition at each crossing, DFO may require an open bottom or oversized culvert to offer improved fish habitat. The total estimated cost to replace the noted culverts is $8.8M, including engineering and contingencies, but excluding GST. This cost estimate includes a reasonable contingency to account for potential site specific needs, such as those mentioned above, or inlet improvements (refer to the appendix for cost details), however, the specific configuration and cost at each crossing will require confirmation through a pre-design exercise and through consultation with DFO. A complete list of the recommended culvert upgrades is provided at the front of Appendix C.

Projecting type culvert inlets should be avoided to reduce the risk of debris buildup. Whether a formal headwall is applied or not, the inlet end of the culvert should be flush with the banks.

Several of the culverts are in public right of ways, while many others are on private lands. The estimated distribution is as follows.

Summary of Culvert Upgrade Locations

Private Lands Beyond Roadway	Public ROW on Private Lands	Private Driveway Crossings	Public Road Crossing	Total Number of Culverts
27	5	19	20	71

8.1.4 Construct New Supplemental Channels

Analysis does indicate that new channels, as shown in Figure 17A and 17B, to supplement McGillivray Creek and Lewis Slough do provide a meaningful improvement. While they will not eliminate all risks during an extreme event, it does improve the overall performance, particularly through the core, most densely populated area of the community, thereby decreasing the chance of occurrence and consequences. It is therefore recommended that the City initiate early discussions with property owners to secure an agreement in principle prior to detailed design. At the same time, it also recommended that a similar agreement in principle be reached with DFO / MOE with respect to the profile and channel restoration requirements. Once all are received, more detailed field work will be required to support the design and construction process. The total estimated cost of the two channels with engineering, contingencies, and land acquisition is estimated at $2.6M, excluding GST. This cost estimate accounts for only basic channel restoration and not elaborate fish habitat features. As discussed in Section 7.2, pre-design review and consultation with DFO will be required to address operational and channel restoration issues. Refer to the Appendix C for cost details.

8.2 Review Vulnerable Homes

To verify the specific vulnerability of homes, it is recommended that a survey of the floor elevation of homes reported to have been impacted during the January 2009 event be conducted, along with a site specific review of grading and drainage. These site details are to be reviewed against the predicted flood levels to assist the City in making conclusions on the appropriate course of actions for each home.

8.3 Review and Maintain a Comprehensive O&M Plan

The City has an established channel cleaning program for the Greendale area. The scope of this investigation has not included a comprehensive review to assess how well this program is working, however commentary is offered below to emphasize the importance and continuation of the program.

As discussed in earlier sections of this study, while a lack of maintenance was not the underlying cause of the flooding that occurred in January 2009, it does emphasize that an ongoing inspection and maintenance program is an important aspect of optimizing the capacity of the infrastructure in place, and contributes significantly to managing risk.

The scope of this investigation does not include a comprehensive review or development of O&M programs; however the City is encouraged to review its current program and budgets and make adjustment as necessary to ensure that it is both proactive and optimized for efficiency and effectiveness. It may go without saying, however the three fundamental requirements of the O&M program are to:

- Remove the buildup of excessive sediment in deposition zones;
- Cut bank vegetation and remove debris to maximize conveyance capacity and limit the chance of blockage;
- Inspect control structures, pumps stations, and culvert inlets frequently to ensure optimum performance.

Recognizing that there may be external influences, such as property access on private lands or approvals by MOE / DFO, the City is encouraged to engage stakeholders as required to ensure clear understanding or agreements are in place. Further suggestions on this fact are offered in the implementation section below.

8.4 Floodplain Bylaw

The recommended works for the Greendale community will provide performance in accordance with established criteria for the internal systems; however, risk cannot be eliminated, and the area will remain vulnerable. As discussed in Section 7.8, flood construction levels (FCL) required by the current floodplain bylaw are far more stringent than what is required by the January 2009 event. The City may wish to amend the floodplain bylaw and establish a designated zone within the Greendale area that specifies minimum FCL elevations based on water levels described herein for the January 2009 event. Given this unique circumstance, it is recommended that the City seek legal counsel with respect to amending the bylaw to recognize two FCL levels, and how enforcement of one over the other influences public versus private limitations and liabilities.

8.5 Monitoring Program

An estimated $16M is required to implement the recommended works, and it is anticipated that several years will be required to achieve it. As discussed in early sections of this report, while SCADA data at the pump stations has been a significant set of data to support this study, the Greendale drainage system is highly complex and some level of uncertainty remains as to how hydraulic performance and risk is distributed across the watershed. As described in preceding sections, the performance of the mid and lower reaches of McGillivray Creek is critical. Given this, it is recommended that a monitoring program be implemented prior to any infrastructure upgrades, such that additional base line information can be collected, and performance changes can be monitoring after the completion of the pump station upgrades. The intent of the program is to collect sufficient data such that the predictive hydraulic model can be better calibrated, with the ultimate goal of validating the needs and priorities for recommended works. This is viewed as an important part of the implementation process as discussed in Section 9.

Also highlighted on Figure 30, the recommended monitoring program would include the following:

- tipping bucket rain gauge installed central to the watershed; suggested location could be the City's Greendale fire station.

- two water level sensors and data loggers on McGillivray Creek; one in the vicinity of Sumas Prairie Road, one in the vicinity of Chadsey Road.

- one water level sensor and data logger on Lewis Slough where it crosses Keith Wilson Road west of Sumas Prairie Road.

- one water level sensor and data logger on Dixon Ditch at Hopedale Road.

- one water level sensor and data logger at the diversion structure at the intersection of Adams Road and Hopedale Road.

- one water level sensor and data logger at the intersection of Miller Slough and Wilson Slough.

All loggers should record in 5 minute increments and be hard wired to a power source. As such, it is ideal to locate the water level sensors in the vicinity of a power pole / source. In addition, it is recommended that all data loggers be housed in a safety kiosk and include a remote wireless module for data transfer. All level sensors must be surveyed and calibrated to a geodetic datum.

An installed budget estimate for each station is approximately $12,000, for a total estimate cost of $84,000 for 7 stations. This is based on the City purchasing all equipment outright (as opposed to rental). The final cost will need to be determined by confirming site locations and conditions, as well as the City's desire to include signal relays for remote downloading. The operation and maintenance (O&M) of the stations may be added to the City's contract for current monitoring stations elsewhere in the City, at a suggested budgetary cost of $1,000 per station per year.

It is suggested that the City commit a minimum of 5 years to the program. It may be possible to shorten the program if weather provides a large range of events, however this may not be successful, therefore a 5 year period is suggested. As such, for budgetary purposes, the estimated total cost of the monitoring program is $84,000 for installation and $35,000 for O&M.

At the end of the 5 year period, or once sufficient data is available, the data would be used to recalibrate the model and update the overall capital program. The suggested budget for this process is $50,000.

In terms of value, the total cost of the monitoring and capital plan update program is $169,000; equivalent to the typical cost of constructing only one, perhaps two, culverts across a roadway.

8.6 Community Education and Engagement

Many components of the community wide drainage system are located on private property with no public right-of-ways or protective covenants established. Particularly for the trunk system, unauthorized and inappropriate actions by land owners can have significant consequences on the performance of the system. Public Work staff have reported that it is not uncommon for dams, barriers and unauthorized culverts to be discovered during their routine inspection and maintenance activities. During the data acquisition phase of this study, a significant number of culverts were discovered on private lands that were not recorded in the City's inventory. Along with the responsibility of City staff, optimizing performance and managing risk to the community also requires its residents to be good stewards of the system. It is recommended that the City develop educational material and communicate the importance of this to its residents. It would also outline the "do's and don'ts" and authorization processes for activities they propose on the drainage system. However, this somewhat passive approach may have short lasting effects; therefore, for system components having significance to the greater community, the City may wish to consider putting a program in place to secure formal rights of ways or covenants that will run in perpetuity with the properties.

9.0 IMPLEMENTATION STRATEGY

There is sufficient information to allow some firm decisions, while other decisions require the resolution of uncertainties that cannot be adequately addressed within the scope of this study. As such, aside from a strategy to implement the physical works, the following program is recommended to make final decisions on what all physical works should entail. This is not an exhaustive list, but one of critical early steps to reduce technical uncertainty, manage risk, and optimize decisions.

Step 1 - Implement the recommended monitoring program and revisit the O&M Program.

Step 2 - Conduct the more comprehensive review of the existing pump stations.

Step 3 – Initiate dialogue with MOE and DFO (see Section 9.2 below).

Step 4 - Subject to Step 2 and 3, implement Collinson pump station upgrades.

Step 5 - Once sufficient data is collected from the monitoring program, conduct an updated assessment of the systems hydraulic function and predicted performance with the recommended works. More clearly understanding the hydraulic performance of the mid and lower portions of McGillivray Creek is critical to managing risk, and is important prior to completing culvert upgrades in this system.

Step 6 – Dependant on step 5, conduct a pump station optimization review that will guide final conclusion on the degree to which to reinvest in the existing McGillivray pump station, to invest in a new station, or potentially a combination of the two. In the near term, continue towards a reinvestment in the McGillivray station, and at a later date look to a new supplemental station if the updated assessment (step 5) reaffirms the findings and money becomes available.

Step 7 – Proceed with implementing all physical works in a systematic fashion, as discussed below.

9.1 Priorities and Phasing of Physical Improvements

With respect to implementing the recommended infrastructure improvements, a strategic approach is required. This strategy has been developed based on a three dimensional matrix as follows:

Dimension 1 - Start downstream, progress upstream. In general, if this principal is not followed, relieving constraints in the upper reaches can increase risk to the downstream reaches. The system has been divided into three equal zones; lower (priority 1), mid (priority 2) and upper (priority 3), as shown in Figure 31.

Dimension 2 - Start with the trunk system, end with the local system. As shown in Figure 31, each reach of the conveyance system has been assigned a ranking order; 1st order (priority 1), 2nd order (priority 2), and 3rd order (priority 3).

Dimension 3 - Start with the most significantly undersized units. The premise of this dimension is to relieve the biggest constraints first. As shown in Figure 31, all culverts have been divided into roughly three equal categories based on the ratio of the proposed size to the existing size; major deficiency (priority 1), mid deficiency (priority 2), and minor deficiency (priority 3). Pump stations and channel upgrades are assigned priority 1.

All priority 1 items are assigned a score of 3, priority 2 items are assigned a score of 2, and priority 3 items are assigned a score of 1. Using GIS technology, a composite score is computed for each item. Based on their score, the items are finally grouped into three phases, the results of which are presented in Figure 32. The number one recommended priority is to start implementing the pump station upgrades, starting with a pre-design study which will involve a detailed status review of the existing stations, confirm optimum equipment upgrades, and prepare a financially viable phasing program. First priority should be put to the Collinson pump station, followed by the McGillivray pump station. Supplemental channels then culvert upgrades would follow completion of the pumping improvements.

While the implementation strategy outlines the sequence of works, it does not suggested a time frame schedule, as that is dependent on the City's cash flow position and priorities relative to other needs of the City.

9.2 Environmental Approvals

For the most part, the recommended program involves working in and about a watercourse which will require liaison with MOE and DFO during design and construction phases. For the purposes of this study, the SHIM (Sensitivity Habitat Inventory Mapping) has been predominantly used as a guide in formulating recommendations. Attempts were made on numerous occasions to seek early feedback from ministry representatives, but were unsuccessful prior to completion of this report.

Given the nature of the proposed works, and the fact that most of the watercourses are classified as having fish presence, it is anticipated that Provincial Water Act, Section 9 Notifications will be required. As stated in preceding sections, there are many aspects of potential (eg. dredging of McGillivray Creek) and recommended works that require environmental considerations and discussion with ministry representations, with solutions being tailored to suit the specific site condition. It is recommended that the City present this strategic plan to both MOE and DFO representatives, followed by a meeting(s) to establish a clear Memorandum of Understanding with respect to the fundamental capital work recommendations and the long term O&M program. The goal is to reach clear understanding between all parties with respect to practices and procedures in order to streamline the approval process and avoid future conflict. It is recommended that this process be taken near term and prior to the City adopting formal implementation schedules and budgets.

9.3 Seeking Grant Funding

The City had a desire to receive input on pursuing grant funding for the implementation of works in the Greendale area. When this study was initiated in the spring of 2009, provincial and federal grant programs were active, but heavily prescribed. The recent economic downturn has been detrimental to programs and available funding. At this current time, most of the programs have gone idle and we are not aware of any active program that will be applicable to implementation funding, however we offer the following commentary to assist the City with lobbying senior levels of governments and being prepared for programs when they become available once again.

In the past year, the most relevant grant programs that may have been considered as potential sources to assist with the Greendale area included:

- **Building Canada Fund - Communities Component** – The Building Canada Fund was the primary source of infrastructure funding in Canada. The Province delivers its $100 million, 10-year Flood Protection Program through the Building Canada Fund (i.e., for flood protection projects, the Province contributes its share from the Flood Protection Program). Flood protection projects are specifically identified as eligible projects under the Building Canada Fund. Recent communications with the Ministry of Community and Rural Development indicate that there may not be an uptake for the Building Canada Fund in 2010.

- **Gas Tax Funding** – The other significant source of infrastructure funding is provided through the Gas Tax Agreement between the government of Canada, British Columbia, and the Union of BC Municipalities. The Agreement provides grant funding for initiatives that reduce greenhouse gas emissions, provide cleaner air, or provide cleaner water. Gas tax funding can be used for capital investments as well as capacity building initiatives. While the Agreement specifically identifies "developing or upgrading wastewater and storm water systems to improve water quality and improve aquatic habitat" as an objective, it is unclear whether flood protection measures would be eligible under this program.

- **Infrastructure Planning Grants** – This is the one program that does still exist, however is consistently in high demand and heavily overprescribed. The Province's Infrastructure

Planning Grants provide up to $10,000 to local governments for long-term infrastructure planning, including feasibility assessments for municipal infrastructure projects. The City would be eligible to apply for multiple infrastructure planning grants for flood protection infrastructure.

Notably, none of these grant programs specifically fund land acquisition. While grants can be of significant assistance, communities must realize that: 1) funding programs are routinely oversubscribed, thereby lowering chances of success; and, 2) most funding programs require local governments to make a contribution (typically 1/3rd of the total cost) – it is rare to receive 100% grant funding. Therefore, local governments must carefully consider the role that grants play in their longer term financial plans.

Local governments compete most effectively for grants if their applications:

- **Reflect broader community policies and priorities** – Projects that reflect broader community policies and priorities are generally more successful at obtaining grant funding than projects that do not. Therefore, for grant applications related to this study, the City should clearly outline the link between each project and policies included in documents such as:

  Official Community Plan – The City's Official Community Plan includes the following policy statements related to flood prevention:

  - *To develop a flood protection management plan for valley lands.*
  - *Initiate and manage municipal flood protection measures for the Fraser River.*
  - *Identify flood protection measures with Provincial agencies that are appropriate for the Fraser Valley.*
  - *To provide flood protection and minimize effects upon aquatic environments.*
  - *To review and revise municipal stormwater management.*

  Choices for Our Future – Regional Growth Strategy for the Fraser Valley Regional District – The Regional Growth Strategy supports protecting rural and agricultural areas, and includes these specific action items related to flood protection:

  - *Develop, in collaboration with federal, provincial, local government and First Nations, a floodplain and flood proofing management plan that addresses issues identified in the BC Ministry of Water, Land and Air Protection's "Flood Hazard Management Program Review".*
  - *Promote and facilitate the coordination and financing of federal, provincial, regional, and local efforts with respect to flood control and dyke management.*

- *Manage the removal or movement of gravel using best management practices to assist in flood management and prevention.*

Goals and Objectives – 2009 – The City establishes annual goals and objectives, one of which for 2009 was: *Good stewardship of municipal infrastructure - Protect the community from flood risk*.

- **Align with Provincial/Federal priorities** – Provincial and Federal governments design their grant programs to reflect their respective priorities. Therefore, the greater the congruence between a given project and Provincial/Federal priorities, the more likely the project will be funded. Grant applications related to this study will need to show how each project reflects the following policies:

Living Water Smart – Living Water Smart outlines policies for managing BC's water resources sustainably. The following policies relate to flooding:

- *By 2012, new approaches to water management will address the impacts from a changing water cycle, increased drought risk, and other impacts on water caused by climate change.*
- *Adapting to climate change and reducing our impact on the environment will be a condition for receiving provincial infrastructure funding.*
- *Where new development on flood plains is unavoidable, it will be flood-proofed to high provincial standards.*
- *Government will provide $100 million for flood protection over 10 years to help communities manage flood losses.*

Climate Action Plan – the Climate Action Plan outlines a plan towards meeting the goal of reducing greenhouse gas emissions. Relevant policies are:

- *By 2020, B.C. will reduce its greenhouse gas emissions by 33 per cent, compared to 2007 levels. In addition, legally binding targets will be set this year for 2012 and 2016.*
- *By 2050, GHG emissions in the Province will be reduced by at least 80 per cent below 2007 levels.*
- *By 2010, the B.C. public sector will be carbon neutral. In other words, the government is setting an example and keeping its own carbon footprint as small as possible.*

BC Agriculture Plan – The Province's Agriculture Plan identifies the need to meet environmental and climate challenges, and specifically mentions "*reducing or mitigating the risk posed by the negative impacts of climate change*". Flood protection measures in Chilliwack could align with this objective

- **Build an effective business case** – Obtaining grant funding is fundamentally about building a solid business case. Local governments must demonstrate how and why a given project should be a funding priority. As discussed above, part of building a strong business case is to demonstrate linkages to broader community objectives and Provincial/Federal priorities. Building an effective business case also entails demonstrating, wherever possible, cost savings (typically for the Provincial/Federal government). In this case, it may mean comparing PEP claims to infrastructure costs over time. Comparing costs and benefits will require a broad assessment of impacts (e.g., financial impacts, social impacts, environmental impacts).

- **Clearly demonstrate the community's ability to fund its portion** – Since most grants require a local contribution, it is critically important that Chilliwack be able to demonstrate its ability to fund its portion. Funding agencies are not interested in allocating resources to communities that do not have a plan in place for funding its portion of the one-time infrastructure costs or for funding on-going operations and maintenance.

- **Leverage other opportunities** – Wherever possible, projects should be coordinated with other works to realize cost savings. Coordinating works may also give the City access to funding sources that may not otherwise apply to a flood protection project. For example, the City may be able to secure funding for active transportation projects (e.g., bikepaths on a dike) that could also be used to address flood protection deficiencies (e.g., dike deficiencies).

- **Show how the community plans to manage its assets** – For grant programs that fund capital infrastructure, communities may find they have a better chance of receiving funding if they can demonstrate that they will be good stewards of the infrastructure over time. Most funders are interested in funding infrastructure only once; therefore, demonstrating an ability and plan to reinvest in infrastructure proactively can help a grant application.