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1.0 Introduction

1.1 Background

The last Wastewater Master Plan update was completed in 1995. Since then the City of Grande Prairie has become the servicing hub of northern Alberta and experienced tremendous growth. In order to accommodate new developments, an assessment of the existing wastewater system is warranted to assist Aquatera in the approval of new developments. In 2003, Aquatera Utilities Inc. commissioned Infrastructure Systems Ltd. To update the 1995 wastewater collection system master plan. The new master plan will become a planning tool for Aquatera.

1.2 Objective

The purpose of this study is to develop a plan for a cost-effective and efficient wastewater collection system within the City of Grande Prairie, and to improve the deficiencies in the existing system and accommodate future developments.

1.3 Scope of Work

The scope of work for this assignment is described as follows:

- Software evaluation and selection
- Update the existing wastewater system model, including model calibration
- Determine deficiencies in the existing system and improvement requirements
- Use the wastewater system model and projected future growth to determine system upgrading and expansion requirements
- Incorporate recent and proposed system acquisitions into the Master Plan including the Airport and Brochu Industrial
- Estimate timing and costs for upgrading and expansion of the wastewater system
- Review design guidelines for water and the wastewater system
- Recommend funding mechanisms for upgrading and expansion of the wastewater system
- Develop 5-year and 10-year capital plans

- Develop a financial plan to fund capital infrastructure, including benefiting area calculations for cost recovery
- Provide training to Aquatera staff on the operation of the wastewater system model
- Review best management practices for maintaining asset value of the existing infrastructure
- Conduct public consultations
- Prepare draft and final reports.

2.0 Literature Review

2.1 Previous Report

“City of Grande Prairie – Wastewater Collection System Master Plan.” Infrastructure Systems Ltd., April 1995.

ISL completed a Wastewater Master Plan for the City of Grande Prairie in 1995. The Master Plan was prepared by developing a computer model of the existing wastewater collection system. The model was calibrated using flow monitoring information accumulated in the spring and summer of 1994. The model was then used to determine if excess capacity was available in the system to accommodate the anticipated growth. Some of the key findings in the report included:

- The existing wastewater collection system has adequate capacity to accommodate dry weather flows.
- A significant wet weather inflow problem exists within the wastewater collection system.
- Large inflow is having a significant impact on the collection and treatment systems.

Some of the key recommendations provided in this report are summarized as follows:

- City of Grande Prairie should intensify their inflow reduction program.
- Inflow reduction program should include smoke testing, and manhole and flat topped roof inspections (program should last two years).
- The flow monitoring program should be continued for at least five years to further refine the model and to obtain a more accurate measure of inflow.
- A moratorium on development north of 116 Avenue until the inflow monitoring program is completed.
- Construct the East Trunk Sewer from 100 Street to Resources Road to service new development in the southeast and east side of the City.
- Construct a relief interconnect at 100 Avenue and 92 Street.

2.2 Other Reports

“City of Grande Prairie – Utility Management Study Volume II (Inflow/Infiltration).” GCD Dillon Consulting Limited, May 1992.

This study undertaken by GCD Dillon in 1992 was performed to estimate the magnitude of the inflow and infiltration (I/I) into the sanitary system and assess the impact that this I/I was having on the system. Some of the findings included:

- Peak daily wet weather flows can be four times more than the average daily dry weather flow
- Rain induced infiltration (flow from weeping tiles to the sewers and direct infiltration into the sanitary sewer pipes due to rain) represents approximately 25% of the wet weather I/I.
- Inflow represents approximately 75% of wet weather I/I.

Some of the key recommendations in the report include:

- Revise the City’s Development Standards to preclude any further weeping tile connections to the sewers.
- Continue the current program of CCTV sewer inspections and remedial works.
- Seal three of every four manhole vent holes and place loose bolts in every fourth vent hole.
- Carry out field trials to determine the effectiveness of sealing lid/frame gaps and frame/ground gaps of all manholes in grassed areas.
- Consider establishing a weeping tile charge on property owners who have weeping tile systems connected to the sanitary sewers.
- Establish a continuing program to identify and remove storm connections to the sewers.
- Repair or replace the influent flow meter at the WWTP.
- Establish a continuing program of sanitary flow monitoring.

“City of Grande Prairie – 116 Avenue North Inflow Assessment.” Infrastructure Systems Ltd., January 1996.

This study was undertaken by ISL in 1996 to complete an inflow reduction assessment for the area north of 116 Avenue. This included a manhole inspection program, a flat-

topped roof inspection program, selective smoke testing and flow monitoring on the two collector sewers servicing the area. Some of the key findings of the study included:

- The 1995 flow monitoring program revealed an extremely severe inflow problem in the residential area north of 116 Avenue.
- During a 1-year event experienced on July 14, 1995 the pipe was flowing at 1.3 times its full flow capacity.
- The industrial area north of 116 Avenue also experienced significant wet weather inflow problems, they were deemed not as critical as there are no basements within the service area.

Based on these findings some of the key recommendations included:

- All bolts in manhole cover vent holes should be replaced immediately in the spring with watertight P.V.C plugs.
- Manhole covers which are below surrounding grade should be raised, re-landscaped; and the grade rings, frame and cover seat be sealed with mastic compound.
- The owners of the three houses south of 116 Avenue that are known to have rooftop leaders connected to the sanitary sewer be approached and a cooperative effort be made to disconnect them.
- A flow monitoring program should be conducted in 1996 at MH's 200082, 110039 and 140001 to assist in evaluating the effectiveness of the inflow reduction program.

**“City of Grande Prairie – 1995 Flow Monitoring Summary Report.”
Infrastructure Systems Ltd., January 1996.**

This report conducted by ISL in January, 1996 was conducted to summarize the flow monitoring data collected in the summer of 1995. Some of the key findings included:

- Flow monitoring on the 96 Avenue collector sewer servicing Richmond Industrial Park yielded a wet weather peaking factor of 4 for the 1:1 year rain event.
- The monitored wet weather flows on the collector sewer servicing the residential area north of 116 Avenue exceed the pipe capacity by approximately 40% for the 1:1 year rain event.
- The critical storm event of approximately 1:10 year severity is still required to obtain a more accurate assessment of wet weather inflow.

Some of the key recommendations included:

- No changes should be made, at this time, to the Assessment Event utilized in the 1994 Wastewater Collection System Master Plan.
- Additional flow monitoring is required to obtain wet weather flows for rain events in the 1 to 10 year severity range.
- An aggressive inflow reduction program should be completed for the area of the city north of 116 Avenue.

Various Flow Monitoring Summary Reports, Rocky Mountain Instruments (RMI), 1994 – 2000.

ISL also reviewed the various flow monitoring reports done by RMI over the last 6 years. These reports provided a summary of the flow monitoring program for that year, as well as graphical presentation of the results. They did not produce any findings or make any recommendations.

“West Annexation Lands Sanitary Study – City of Grande Prairie.” Stewart, Weir and Company Limited, 2003.

This report was conducted to address the necessary feasibility and cost sharing required to provide services for the airport area and the existing Brochu industrial subdivision.

The conclusions reached in this study included:

- The main trunk would be constructed by the developer of the Centre West Business Park during Phase II and the size of the trunk would be increased in size to 600mm.
- An additional trunk would be extended north to Highway 43 from 99 Avenue.
- The trunk running from the existing airport lift station into the Brochu quarter would be at 375mm and run at minimum grade to keep the elevations of all of the downstream systems from being adversely impacted.
- Each developer of the adjoining lands would either construct their storage requirements at the time of development, or provide a levy or bond to the City of Grande Prairie for construction when required.

“Hidden Valley Area Structure Plan.” UMA Engineering Ltd., January 2000

The Hidden Valley Area Structure Plan was prepared to describe the proposed land uses, population density, transportation routes, infrastructure and staging for an area northwest of Grande Prairie. It was intended to provide a conceptual framework for

development at a neighbourhood scale, rather than providing site specific guidance.

The ASP area was bounded by the following features:

- to the east, 108 Street and 106 Street
- to the south, the Highway 43 Bypass and the Gateway Power Centre
- to the west, the west limit of NE27-71-6-6, SW34-71-6-6 and Bear Creek within NW34-71-6-6
- to the north, 132 Avenue which defines the north limit of the City of Grande Prairie.

The report covers the topography in the area, the geotechnical conditions, the flooding potential and the future land use. Future roadway alignments as well as a conceptual stormwater management plan and a water distribution plan, were produced.

The conclusions reached in the report in relation to the sanitary collection system included:

- sanitary sewer servicing for the lands west of Bear Creek will be provided in accordance with the 1995 Wastewater Collection System Master Plan
- the West Trunk has the depth and capacity to service both the College lands and the lands directly north (SE34)
- the future 250mm line will run west along 107 Avenue, then north through the College lands, to ultimately service the lands directly to the north (SE34)
- east of Bear Creek, the Wastewater Collection System Master Plan shows the area being serviced by the trunk system extending along the Highway 2 Bypass
- servicing the Hidden Valley Area east of the creek will require the extension of the sanitary sewer from the existing 450mm stub along Hidden Valley Drive to the south portion of the ASP
- the north-eastern corner of Hidden Valley is a distinct drainage basin and will require a lift station for sanitary sewer drainage.

“Proposed Pinnacle Ridge Development – SE ¼ Section 15-71-W6M – Overall Sanitary Sewer Servicing Concept.” GPEC Consulting, March 1999.

The purpose of this report was to consider the conceptual design for Sanitary Sewer Servicing for the Proposed Pinnacle Ridge Development, located in the SE ¼ Section

15-71-6-W6M. The proposed location, grades, inverts. Etc. of the sanitary trunks are conceptual only.

2.3 List of Additional References

- Proposed Flying Shot Lake Development – NW ¼ Section 15-71-6-W6M – Overall Sanitary Sewer Servicing Concept – GPEC Consulting Ltd., 2000
- Sanitary Sewer Design Report – HAZCO Industrial – Focus Intec, 2002
- Proposed Northridge Commercial Development, Proposed Royal Oaks Residential Development – SW ¼ Section 35-71-6-W6M – Overall Sanitary Sewer Servicing Concept – GPEC Consulting Ltd., 1998
- Corral Group Limited – Sanitary Sewer Calculations – Focus Urban Consulting, 1999
- Design Report for Sanitary Sewer System – Westgate Business Park – UMA Engineering Ltd., 1999
- Proposed Development – Part of NW ¼ Section 23-71-6-W6M – Overall Sanitary Sewer Servicing Concept – GPEC Consulting Ltd., 2001
- City of Grande Prairie – Annexation Servicing Assessment – ISL, 1995
- Proposed O'Brien Lake Development – NE ¼ Section 10-71-6-W6M – Overall Sanitary Sewer Servicing Concept – GPEC Consulting Ltd., 2003
- City of Grande Prairie – Northeast Area Structure Plan – ISL, 2003
- City of Grande Prairie – Southwest Area Structure Plan – ISL, 2002
- Utilities Corporation Review Update December 2002
- Aquatera Utilities Inc. 2003/2004 Business Plans/Capital & Operating Budgets
- Transportation Master Plan
- AEP Standards and Guidelines
- County, City, Sexsmith Regional Water and Wastewater Systems
- Letter of Intent – Town of Sexsmith
- Letter of Intent – County of GP
- Annexation Agreement
- Intermunicipal Development Plan
- Clairmont Servicing and Planning Area Study
- 1991 Sanitary Sewer Flow Monitoring Report, GCG Dillon
- Overall Sewer Plan Showing Flow Monitoring Locations 1990-2000

3.0 Population and Land Use Projections

3.1 Existing Population

According to the 2003 Municipal Census, the population of the City of Grande Prairie is 40,226. Over the past ten years, the City has experienced an average growth rate of approximately 3.5% annually.

Based on this past trend, it is assumed that the City's population growth rate will be sustained for the next two years, and then moderate to 2.5% per year thereafter.

3.2 Existing Land Use

The City has a total land base of approximately 6,140 ha within its current limits, of which approximately 3,100 ha (50.5%) is developed. This includes the airport, landfill and sewage treatment plant, all of which were annexed into the City in 2001.

As indicated on Figure 3.1, the majority of the City's residential development is located in the central, northeast, and southerly areas of the City. New residential development is occurring in all directions, however, most recently in the northwest, southwest and southeast quadrants.

Commercial development is concentrated in the downtown area, along 100 Street to the north City limits, along 100 Avenue west of 108 Street (Gateway/Westgate), and along the Highway 43 Bypass. Future commercial development is expected to continue to occur in these areas, with additional arterial commercial development occurring in the City's west end.

Current industrial development is concentrated on the west side of the City in the Richmond Industrial Park, with smaller pockets located on the east side near the rail yards, and in the City's north end. Future industrial development is expected to continue towards the west in proximity to the airport, along the east side of the rail line, and in newly annexed lands to the north of 132 Avenue.

Figure 3.1 also shows the locations of the various area structure plans that are in effect in the City's expansion areas.

3.3 Population Projections

The growth projection rationale is described as follow.

The City's population growth for the last ten years has averaged 3.5% per year. It is assumed that this rate will continue for the next three years , but moderate to 2.5% from 2007 through 2013.

The ultimate City population is estimated to be ~94,600 based on all approved and pending Area Structure Plans. The last annexation study and Transportation Master Plan estimated this figure to be 75,000. The main reason for the discrepancy is the recent trend toward higher densities, small lot sizes, and generally more compact urban form.

3.4 Growth Projection Rationale

The population projection on an area and zone basis is shown on the Population Table in Appendix A. The location of the areas and zones are shown on Figures 3.2 and 3.4 respectively.

Commercial and industrial growth is indexed to population. At present, commercial development in the City is ~5 ha/1,000 population. This ratio has been maintained for the three horizons (5 year/10 year/ultimate). The actual figure for the ultimate is 4.6 ha/1,000 population, based on land use allocations in all ASPs. Industrial development is approximately 11 ha/1,000 population for all horizons.

The commercial and industrial growth projection is shown in the Commercial and Industrial Table in Appendix A. The location areas and zones are shown on Figures 3.3 and 3.4 respectively.

4.0 Wastewater Collection System Design Criteria

4.1 Introduction

One of the tasks of the Wastewater Master Plan was to review the current wastewater collection system design criteria and establish new design criteria as required. The task consisted of examining the flow monitoring data at numerous manholes located throughout the City of Grande Prairie during the years 1999, 2000, and 2001, and comparing these findings with the design criteria of other municipalities. Based on this evaluation, appropriate wastewater collection system design criteria were established and were recommended for implementation.

The current design criteria for sewage collection as stated in Section 19 of the “*Grande Prairie Design Standards, Revised May 2002*” is tabulated and shown in Table 4.1.

The wastewater collection system design criteria for each type of land use is discussed in the following section.

4.2 Residential Development

4.2.1 Current Residential Wastewater Flow

Flow monitoring data was reviewed at MH 300045 located in the northeast corner of the City of Grande Prairie. Almost all of the wastewater generated by the upstream development that would flow to this manhole is from single family residential developments. This location also provided us with the best opportunity to provide an accurate assessment on the population in the year 2000, which was at the time when the flow monitoring data was collected. The population was based on an assumed average of 32 people/ha over a residential area of approximately 80 ha. The average per capita flow rate was calculated to be between 206 – 250 L/capita/day with the average flow being approximately 233 L/capita/day. Table 4.2 shows the data used in the analysis and the results generated.

4.2.2 Proposed Design Criteria

Wastewater flow design criteria for residential developments for other municipalities, shown on Table 4.1, were reviewed. Table 4.1 shows that the residential wastewater generation flow rates ranged from 300 to 395 L/capita/day.

Based on the analysis of the flow monitoring data, it is recommended that City of Grande Prairie continue to use their current residential design standard of 275 L/capita/day. The 275 L/capita/day is approximately 15% higher than the average flow generated based on the flow monitoring data.

4.3 Residential Development Peaking Factors

A review of the residential peaking factor for similar municipalities showed that the minimum peaking value ranged from no minimum (St. Albert) to a minimum of 3.0 (Strathcona County). All municipalities, with the exception of the City of Edmonton, utilized Harmon's formula to calculate their peaking factor.

The flow monitoring data at MH 300045, MH 300036, MH 340009, MH140001, and MH130006 was used to evaluate the peaking factor for residential development. The flow monitoring data showed that the maximum peak factor varied between 2.2 – 2.8, based on flow monitoring data. Based on this flow monitoring data, Harmon's equation for peaking factors, and the City of Edmonton's equation for peaking factors, were compared for the upstream areas to each of the flow monitors. The results of this analysis can be found in Table 4.3.

Based on this data, the City of Edmonton's formula produces peaking factors closer to those actually observed in Grande Prairie. It would be far too conservative to continue to use Harmon's formula based on these results. Hence, it is recommended that the City of Grande Prairie revise its standards from using the Harmon formula to the City of Edmonton formula.

4.4 Commercial Development

4.4.1 Current Commercial Wastewater Flow

Using the flow monitoring information to develop the standards for commercial developments was not possible. None of the flow monitoring data collected in the last

five years was dedicated to areas that were strictly commercial developments. There were a few areas where a flow monitoring location provided flow data for commercial and industrial development but this data proved highly unreliable.

The results of the analysis of water billing records for 25 lots indicated that the water consumption for commercial developments is approximately 13,200 L/ha/day.

4.4.2 Proposed Design Criteria

A review of the standards for commercial development for other similar municipalities showed a range of 17,280 to 40,000 L/ha/day, with an average value of approximately 28,000 L/ha/day. Table 4.1 shows that six municipalities out of 13 have a wastewater flow for commercial development range from 20,000 to 26,000 L/ha/day with an average of 23,000 L/ha/day. Since highway commercial generally has higher wastewater flow than other commercial developments, the following design criteria for commercial developments are recommended:

- Highway commercial developments – 26,000 L/ha/day (the high end of the range).
- Other commercial developments – 20,000 L/ha/day (the low end of the range).

4.5 Industrial Development

4.5.1 Current Industrial Wastewater Flow

In the data provided by the City of Grande Prairie to ISL, there was only one manhole (MH 200003) that monitored strictly industrial wastewater. The rest of the flow monitoring data provided was either for residential or a mix of development types.

The data from MH 200003 was analyzed and an average flow rate of approximately 6,100 L/day was calculated. The area serviced by this sewer is approximately 39 ha in size. This means that the average industrial flow rate for this area would be approximately 160 L/ha/day. Based on a review of the water billing information, it is likely that the flow data at this location is unreliable.

In consultation with the water billing records provided, the water consumption for industrial areas is approximately 8,000 L/ha/day.

4.5.2 Proposed Design Criteria

A review of the design standards for other industrial development in other municipalities showed a range of 16,875 – 30,000 L/ha/day. Unlike the City of Grande Prairie, the majority of the municipalities reviewed do not provide different rates for light and heavy industrial developments. Grande Prairie's standard for industrial sewage generation is similar to that of numerous other municipalities.

Since the current design criteria were determined on the basis of actual billing records, and the fact that adequate records were not available for this assignment, it is recommended that the City of Grande Prairie continue to use the wastewater design criteria for the industrial developments as follows:

- Light industrial – 10,000 L/ha/day.
- Heavy industrial – 20,000 L/ha/day.

4.6 Institutional Development

4.6.1 Existing Institutional Wastewater Flow

Flow monitoring data was not available for the analysis of wastewater flow from institutional developments. Based on water billing records, the water demand for hospital and school is 72,250 L/ha/day and 13,000 L/ha/day respectively.

4.6.2 Proposed Design Criteria

Based on insufficient flow monitoring data and the results from the water billing analysis it is recommended that the City of Grande Prairie use the same water demand standards for wastewater flow standards as follows.

- School: 20,000 L/ha/day
- Hospital: 30,000 L/ha/day

4.7 Non-residential Development Peaking Factor

The peaking factor used for non-residential developments in the current standard is the lowest among all municipalities shown on Table 4.1. Both Alberta Environment and the City of Edmonton have introduced similar peaking factors for non-residential developments. Generally, the peaking factor decreases when the average flow or

development area increases. This peaking factor is more realistic because generally, the amount of fluctuation in wastewater flow is lower for larger developments.

It is therefore recommended that peaking factor recommended by Alberta Environment be used by the City of Grande Prairie.

4.8 Inflow / Infiltration (I/I) Allowance

4.8.1 Monitored I/I in New Development Areas

The current rate of I/I was estimated based on the available flow monitoring data in new development areas where the servicing practices were assumed to be similar to future developments. These development practices include the use of PVC pipe for sanitary sewers, providing adequate lot drainage away from foundation drains, and connecting foundation drain sump pumps to the ground surface or storm sewers, not sanitary sewers.

This analysis estimated the I/I from new development areas by subtracting the monitored flows during wet periods from the monitored flows during dry periods to develop I/I hydrographs. This was conducted at three locations during the year 2000: MH 300036, MH 169901 and MH 340009. The results of this analysis are found in Table 4.4.

It should be noted that the September 2000 rainfall event represented less than a 1:2 year event. Thus higher I/I rates can be expected during more extreme wet weather events.

4.8.2 Proposed Design Criteria

A review of other municipality's infiltration rates showed a range of 0.069 – 0.50 L/s/ha with the majority of the municipalities infiltration rates falling between 0.20 – 0.28 L/s/ha. The most commonly used rate is 0.28 L/s/ha, which is used by five municipalities and is recommended in the provincial guidelines.

The monitored I/I rates in Grande Prairie's newly developed areas during the relatively small storm event of September 2000 (less than 1:2 year) are all larger than the City's current design standard of 0.10 L/s/ha. An increase in the I/I allowance is therefore required to provide adequate capacity in the sanitary sewers during wet weather flow conditions. It is recommended that the I/I allowance be increased to 0.28 L/s/ha to

accommodate peak wet weather flows from more extreme events. This criterion would be consistent with that of several Alberta municipalities and the provincial guidelines.

The 0.28 L/s/ha criterion could be reviewed in the future, if justified, based on flow monitoring data from new development areas during an infrequent rainfall event (say 1:10 year or less frequent).

It is recognized by many municipalities that the sag manhole is a source of inflow into the wastewater system. The majority of municipalities specify that the inflow amount at a sag manhole be 0.4 L/s. It is recommended that the same amount be used for the City of Grande Prairie.

4.9 Pipe Design Flow

At present, the City of Grande Prairie calculates design flows at pipe full conditions. As a factor of safety, it is common to divide peak pipe flows by 0.86 to determine design flows. It is recommended that the City of Grande Prairie make this change to its standards. This will be useful in protecting against the rare occasion where the peak flow in a pipe might exceed the recommended peaking factor.

4.10 Summary

In summary, the changes recommended to the Grande Prairie wastewater standards are as follows:

- Residential peaking factor changed to the City of Edmonton formula from the Harmon formula
- Industrial/Commercial peaking factor changed from 2 to the formula presently recommended by provincial standards
- Inflow/Infiltration allowance changed to 0.28 L/s/ha from 0.10 L/s/ha
- Pipe design flow increased by a factor of 1/0.86.

It is necessary to consider the ramifications of the changes to the standards as specified. Table 4.5 considers a typical residential/commercial quarter section with the old and new standards. As shown, the new standards increase the design flow and will provide a greater factor of safety in the design of any new areas.

5.0 Review of Wastewater Model Software

5.1 Introduction

The City currently utilizes XP-SWMM Version 6.1 for the evaluation of the sanitary sewer system. One of the tasks of the Wastewater Master Plan is to select the most suitable computer model for the City. The new modeling program must be widely used, user friendly and produce accurate and easily readable results.

An investigation and evaluation of the software available for the modeling of wastewater collection systems was undertaken. The results of the investigation and evaluation are documented below.

5.2 Selection Process

Initially, a literature review and internet search was conducted to determine the modeling programs that would best suit Grande Prairie's modeling needs. Once this was accomplished, the advantages and disadvantages of each model were determined based on cost, database compatibility, ease of use, model capabilities, results produced, access to results, visual aids, and ease of updating. The advantage of having one single model to analyze both the sanitary and storm systems to reduce the inconsistencies between models and the training of personnel, was also considered as a factor in the evaluation.

5.3 Models Evaluated

There are many different sanitary sewer models available on the market in North America. The models chosen for this part of the study were models that were the most widely available, had the most on-line help, and had sufficient after purchase support from the retailers. Special attention was paid to those models that are widely used in Canada, and especially in Alberta. To run the models, the computer system should at

least have a 1.5 GHz processor and 512 MB of RAM in order to efficiently calculate the results. The following models were chosen for evaluation:

- SWMM 4.3 (US EPA)
- MOUSE 2002
- XP-SWMM 2000 Version 8.52
- VISUAL OTTHYMO Version 2.0

Other models such as HYDRA, STORM, and SAM (System Analysis Model) were considered but not chosen for further evaluation, due to poor capabilities, poor user friendliness, and lack of popular use in Canada.

5.4 Model Evaluation

5.4.1 SWMM Version 4.3 (US EPA)

Advantages

SWMM Version 4.3 is provided free of charge from the United States Environmental Protection Agency (US EPA). The program is available for download on their website. The program contains the majority of the functions available in the other commercially available modeling programs. It is capable of reading flow monitoring input files, analyzing sanitary sewers, generating hydrographs and hydraulic grades in pipes. It is also capable of modeling large and relatively complex systems (including systems with pumps, weirs, orifices and storage) without any problems.

Disadvantages

The main disadvantage associated with this model is it is not user friendly. The model is DOS based and is programmed using FORTRAN. This requires the user to have strong programming skills in order to be able to input data and properly analyze the output files. The model would require extensive training in order to have a sufficient understanding of its capabilities and functions. The model is not compatible with AutoCAD or with GIS, and does not produce any on screen output graphics. This makes the model extremely difficult to use for anyone who is not an expert at computer programming and advanced hydraulics. The model is also not widely used by consultants and municipalities in Canada.

5.4.2 MOUSE 2002

Advantages

MOUSE is a user-friendly modeling program available from the Danish Hydraulic Institute (DHI). The model is capable of modeling both sanitary and stormwater sewer networks. It easily and efficiently performs all the functions of other well known commercially available software. It has no problem with any network size, and can easily generate hydrographs and hydraulic grade lines. It is also capable of analyzing real or design storms, surcharge levels, system hydraulics and other hydraulic structures (i.e. weirs, pumps, orifices, etc.). It is compatible with the City's database system and can easily be interfaced to work with AutoCAD and with GIS. The model is also capable of modeling systems with real time control. The software has the ability to produce on screen output graphics that are easy to understand. The training required to operate MOUSE would not be that substantial. The model is Windows based and utilizes a graphical interface to make input easier. The programming code of the model is hidden from the user behind the graphical interface but can be accessed by more experienced users. The program also has a very good support base, with good vendor support, along with a wide variety of on-line support groups available.

Disadvantages

The main disadvantage associated with MOUSE is the very high initial cost. The cost of purchasing the program can be upwards of \$25,000, which makes MOUSE relatively unaffordable to smaller municipalities and to many consultants. This cost does not include training and many of their extra modules. Therefore, MOUSE does not have widespread popularity in Canada and North America. The program is relatively easy to use but still takes a very strong understanding of hydraulic principles along with some knowledge of computer programming.

5.4.3 XP-SWMM 2000 Version 8.52

Advantages

XP-SWMM is an extremely user-friendly sanitary/stormwater modeling program available from XP Software Canada. The program utilizes a very friendly Windows-based graphic interface for data input. The interface allows the user to see the system as it is built, and

to be able to click the links to easily retrieve or modify pipe/node data. XP-SWMM readily generates inflow and outflow hydrographs, hydraulic grade lines and stage-storage curves. It is capable of analyzing real and design storms, surcharging in the system, and all system hydraulics and control alternatives (including pumps, weirs, storage, orifices, etc.). The model is completely compatible with the City's database and it easily interfaces with both AutoCAD and GIS. The model is also capable of modeling very large and complex systems. The training required to run XP-SWMM would be minimal compared to some of the other commercially available products. A basic understanding of hydraulic principles is required as well as some skills specific to XP-SWMM that can be acquired by attending one of their many training seminars. The model comes with excellent vendor technical support in the form of yearly contracts that provide unlimited upgrades and support. The model is also very popular in North America and there are a wide variety of support groups available on the Internet. Many different municipalities and consultants utilize XP-SWMM as their main modeling program.

Disadvantages

The main disadvantage in utilizing XP-SWMM as the City's main stormwater/sanitary model is the cost. XP-SWMM is relatively expensive to buy when compared to some of the other programs but is cheaper than MOUSE. Initial start up costs are around \$10,000. Upgrade and technical support costs are about \$1000 - \$2000 per year after that. XP-SWMM also requires the user to be somewhat skilled in hydraulics and in the model itself.

5.4.4 Visual OTT-HYMO Version 2.0

Advantages

Visual OTT-HYMO version 2.0 is available from Greenland Software. The model is Windows based with a graphic interface, which makes the model very easy to use. It has no problem utilizing monitored data and can easily generate a variety of hydrographs. It also has no problem using design or real storm events as input. The model is not very complex and would require very little training to have a good understanding of how it works. The model is also quite cheap when compared to the other commercially available programs. Start-up costs for the program are in the range

of \$4000 - \$6000. The model purchase also comes with very good technical support from the vendor. The model is relatively popular and there are a few support groups available on the Internet. The program also has no problem modeling very large systems.

Disadvantages

Visual OTT-HYMO Version 2.0 is not capable of generating either hydraulic grade lines or analyzing surcharge levels in the system. It also has trouble analyzing system hydraulics and control structures (i.e. pumps, weirs, orifices, storage, etc.). The model does not readily interface with GIS or with AutoCAD and is not compatible with the City's database. The model also does not produce any on-screen output graphics for the user. All results come in the form of a text file.

5.5 Assessment Results

The above evaluation is based on meeting the needs of the City of Grande Prairie, as well as making full use of the databases that the City has already developed. Some of the models discussed above are relatively strong in hydraulic features (like SWMM version 4.3) but lack any user friendliness and take a great deal of skill to use. Other models, such as Visual OTT-HYMO Version 2.0, are very easy to use but are weak in hydraulic computations. XP-SWMM 2000 Version 8.52 and MOUSE 2000 both have relatively easy to understand graphical interfaces, strong hydraulic computations and are compatible with the City's database and with AutoCAD and GIS. MOUSE is a very expensive model to purchase and does not offer any significant features that aren't available in XP-SWMM. The City of Grande Prairie also currently utilizes a lesser version of XP-SWMM, which will make the cost of upgrading much less than purchasing the software new. Below is a table showing the rating of the software based on the aforementioned criteria, with a rating of 1 being the least favourable and a rating of 5 being the most favourable.

	SWMM 4.3	MOUSE	XP- SWMM	Visual OTT- HYMO
Ease of Use	2	4	5	4
Software support	3	4	5	3
Compatibility with the City's GIS System	2	5	4	2
Purchase Price	5	1	2	3
Graphical Capabilities	2	4	4	3
Total	14	18	20	15

5.6 Recommendation

Based on this evaluation, it is recommended that XP-SWMM 2000 Version 8.52 be used to model Grande Prairie's sanitary sewer system. This model will provide the City with the required functions and an easy to use graphical interface for both inputting data and for viewing output. The software can also be used to model stormwater and sanitary sewers.

6.0 Wastewater Model Development

6.1 Existing Wastewater Model

The Wastewater System Model for Grande Prairie was developed by extracting any available and applicable data from the City's existing wastewater model. Unfortunately, the existing model was not functional as several problems presented themselves:

- 1) The hydrologic model contained no data. Any analysis conducted with the existing model would not have reflected any wet weather inflow/infiltration.
- 2) The sanitary layer contained some data which developed sanitary flows for various locations throughout the city. The data, however, was not present throughout the entire city and flows generated were unrealistically small, often less than 0.1L/s.
- 3) The hydraulic model was missing random pieces of data. The end result of this was that any time an attempt was made to run the hydraulics of the system, the entire model would not run.

As a result, only the network connectivity data generally was able to be extracted from the existing system model. Some pieces of data were missing in the hydraulics layer, and an extensive review led to the location of the missing items, which were repaired or replaced as necessary. It was determined that a complete reconstruction of the hydrologic model and sewage inflow hydrographs would be necessary.

6.2 Model Updating

Prior to utilizing the hydraulic model data, the pipe and node data were reviewed and compared to the wastewater system network from the City's database. Several inconsistencies were observed between the existing system model and the City's database which had to be corrected. In a number of instances, the City had made hydraulic modifications to the wastewater collection system since 1995. These were corrected by reviewing the record drawings, consulting with City / Aquatera personnel, and conducting field checks at select manholes. Changes to the hydraulic system, since 1995 have included:

- 1) At MH 200092 at 100 Street and 116 Avenue, the system was adjusted to force flow to the west except under wet weather flow conditions where flows could continue south.
- 2) At MH 130005 at 93 Street and 100 Avenue, the system was adjusted force flow to proceed west except during wet weather flow conditions when flows could also continue east.
- 3) At MH 200073 at 104 Street and 108 Avenue, the system was adjusted to force flows south except under wet weather flow conditions when flows could also continue east.
- 4) At MH 101111 at 102 Street and 104 Avenue, the system was adjusted to force flows east except under wet weather flow conditions when flows could also continue south.
- 5) At MH 110024 at 97 Street and 100 Avenue, the system was adjusted to only allow flow to continue east under any conditions.
- 6) At MH 120017 at 96 Street and 94 Avenue, the system was adjusted to only allow flow to continue west under any conditions.
- 7) At MH 120003 at 99 Street and 91 Avenue, the system was adjusted to force flows south except under wet weather flow conditions when flows could also continue west.

Other revisions were also necessary in the hydraulic model. There were over 80 storage nodes in the model that were not part of the City of Grande Prairie's sanitary sewer system and were removed. Additionally, a substantial portion of the pipe data, particularly pipe slopes, in the model was in disagreement with the data extracted from the City's database. Record drawings were obtained in the necessary areas to confirm that the model accurately represents the current sanitary sewer system.

The hydraulic model was also updated to reflect new pipe construction in growth areas within the City. These updates were based on the City's database and record drawings from the new development areas.

6.3 Hydrologic Model Set Up

As previously noted, the hydrologic component of the previous model could not be used. It was therefore necessary to set up the hydrologic model from the existing land

use and population data. Sanitary catchment areas were established based on the existing Wastewater Network (2003). Additional catchment areas were added for the future Development System Models (2008, 2013 and ultimate).

For each catchment area, the following design parameters were established for each model level of development:

- area for each type of land use (residential, commercial, industrial)
- population
- percentage of area contributing inflow/infiltration to the wastewater system.

The model parameters are summarized in Table 6.1.

6.4 Review of Rainfall and Flow Monitoring Data

The City of Grande Prairie has been monitoring flows in its wastewater collection system since 1993, with an average of about five sites being monitored at any time. The City sub-contracted ISCO, and more recently, Rocky Mountain Instruments (RMI), to supply and install the monitoring sites. City personnel have been responsible for downloading these sites. The locations of the flow monitoring sites are shown in Figure 6.1, with the monitored dates listed in Table 6.2.

The City has also been monitoring rainfall data since 1993, with rain gauges at three locations:

- Rain Gauge #1 – Country Club Estates – 60 Avenue, 92 Street
- Rain Gauge #2 – Service Center – 96 Avenue, 112 Street
- Rain Gauge #3 – Pumphouse #2 (Towers Site) – 113 Avenue, 94 Street.

The rain gauge locations are also shown in Figure 6.1. Data available for the various locations is summarized in Table 6.3.

The flow monitoring data was divided into dry weather periods and wet weather periods for data analysis and assessment. The dry weather flow data was used to calibrate the Dry Weather Flow (DWF) component of the wastewater model and to review the design standards for new developments. The wet weather flow data was used to calibrate the

hydrologic model (inflow / infiltration) and to review the I/I allowance in the design standards. Unfortunately, despite the quantity of rainfall and flow monitoring data that has been collected, the amount of high quality data for model calibration was quite limited, as outlined in the next section.

6.5 Wastewater Model Calibration

The first step in calibrating the wastewater model was to calibrate the dry weather flow component. This included establishing the dry weather flow hydrograph from each catchment area, as well as reviewing the system hydraulics under DWF conditions. Dry weather flow unit hydrographs were developed for the residential areas based on the DWF flow monitoring data. The diurnal factors used to develop the residential hydrographs are given in Table 6.4. Residential flow hydrographs were then developed in an Excel™ spreadsheet, by multiplying the estimated population in each catchment area by the unit hydrograph.

For commercial, industrial and institutional areas, a synthetic unit hydrograph was developed based on an assumed constant flow between 8:00am and 8:00pm. The diurnal factors for these hydrographs are also given in Table 6.4. Hydrographs for each non-residential area were developed by multiplying the unit hydrograph by the wastewater flow generation values and the applicable commercial, industrial or institutional areas.

Initially, hydrographs were developed for the population base corresponding to the flow monitoring dates (e.g. 1999, 2000). Similar hydrographs were then developed for the various modeling scenarios (2003, 2008, 2013 and ultimate).

The resulting DWF model was compared to the monitored DWF at several nodes to confirm that the hydraulic model was correctly routing the flows under DWF conditions. At this point a number of anomalies were noted between the modelled and monitored flows. Several of the anomalies were corrected by incorporating recent changes in the system as noted in Section 6.2. In some cases, the anomalies could not be corrected, and indicated probable errors in the monitoring data. For example, the monitored flow in the downstream end of the 108 Street Trunk was lower than the monitored flow further

upstream. In these cases, the flow monitoring data was set aside and not used for model calibration purposes.

Wet weather flow calibration was conducted using the available flow and rainfall data. Unfortunately, there was a limited amount of flow monitoring data with corresponding rainfall data, which limited the storm events and monitoring locations that could be used for Wet Weather Flow (WWF) model calibration. Three suitable wet weather events were found within the monitoring data. The first was on July 2, 2000. This storm produced 12mm of rain over a total period of 5 hours. The intensity of this storm rates slightly less than that of one with a 2 year return period. On July 5, 2000, a second storm followed which generated 10.8mm of rainfall over approximately 15 hours. At its peak, 7.8mm of rain fell over 5 hours. This storm was less intense than its predecessor, and did not come close to two year return period intensity. The third storm occurred on September 1, 2000 and produced 20.2mm of rain over approximately 7 hours at its peak. Additional precipitation occurred before and after the peak, for a total of approximately 45mm over 3 days. This storm also equates to slightly less than a 2 year return period storm at its peak. Based on the follow monitoring sites with available data, and these wet weather events, the following flow monitors were therefore used for model calibration:

- MH 300036 – July 2-5, 2000 and August 30 – September 4, 2000
- MH 110006 – July 2-5, 2000
- MH 169901 – August 31 – September 4, 2000
- MH 340009 – August 31 – September 4, 2000

The monitored and modelled flows for the above sites are included in Appendix B.

Unfortunately, most of the suitable WWF model calibration data is from flow monitors within new development areas. It was therefore necessary to use the daily flow monitoring data at the Grande Prairie Wastewater Treatment Plant (WWTP) in order to establish WWF model parameters for the older parts of the City. Daily monitored versus modelled flows at the WWTP for the calibrated model is shown in Table 6.5. This table includes a typical dry weather week (June 18-24, 2000), along with the July, August, and September wet weather weeks.

6.6 Calibrated Wastewater Model

Following model calibration, the wastewater model was extrapolated to include the existing (2003) level of development. The 2003 version of the model was used for the existing system assessment in Section 8. The model was also extrapolated to the 2008, 2013, and ultimate levels of development for system planning purposes.

It should be noted that the accuracy of the model is limited by the quality of the available flow and rainfall monitoring data used for model calibration. Since most of the flow monitoring sites are in new development areas, the level of confidence in the model is highest for new development areas. Also, since the model was calibrated using relatively frequent storm events (less than 1:2 year frequency), the level of confidence will be higher for more frequent storm events.

Due to these limitations, it is recommended that flow / rainfall monitoring be carried out on a regular basis to allow for subsequent model re-calibration. The recommended flow monitoring locations are shown in Figure 6.1. The rationale for the selection of these monitoring sites is outlined in Sections 8 and 9. To ensure model accuracy for the older areas of the city and for larger storms, the subsequent flow monitoring must produce quality data for the downtown areas and preferably include data under a higher intensity rainfall event than the ones present for this analysis.

7.0 Existing System Assessment and Upgrading Requirements

7.1 Existing Wastewater Collection System

The existing Wastewater Collection System in the City of Grande Prairie can be seen in Figure 7.1. This shows the existing sanitary sewer system as of 2003. There are four lift stations present in the City of Grande Prairie of which two are shown in this figure. Plan 1 shows the model of the sewer system with catchment boundaries as determined from system assessment

7.2 Selection of Design Rainfall Event

To assess the hydraulic capacity of the existing Sanitary Collection System, it was necessary to select a suitable wet weather event to represent the design rainfall conditions. Since there were no large monitored rainfall events, a synthetic rainfall event was utilized. Design rainfall events were developed using the City of Grande Prairie's rainfall Intensity-Duration-Frequency (IDF) relationship. Return frequencies of 1:2, 1:5 and 1:10 years were used with a four hour storm duration. Rainfall hyetographs were developed using the Huff rainfall distribution, as shown in Table 7.1.

By using a synthetic storm event, it was possible to estimate the increases in flows during less frequent storm events. This will assist in identifying locations with hydraulic restrictions and/or areas that are at risk of basement flooding during periods of intense rainfall.

The simulated flows during these events provide an indication of the magnitude of flows that could be expected within the limitations of the model calibration. Since the model was calibrated using a storm event with a frequency of less than 1:2 years, the level of confidence in the model during the 1:2 year event is higher than that for the 1:10 year event. Thus, the simulated flows during the 1:10 year rainfall event may represent flows during a less frequent event such as a 1:25 year event. Due to hydraulic restrictions in the flow path (e.g. openings in manholes, weeping tile flow paths, etc.), it is more likely

that the model is overestimating the flows during the 1:5 and 1:10 year events than underestimating them.

To assess the hydraulic capacity of existing sewers, the 1:5 year event was selected. Where basement flooding was a concern, the 1:10 year event was checked.

7.3 Hydraulic Modelling

Hydraulic modelling was conducted for the existing Wastewater Collection System based on full development in the upstream service areas. For the existing 92 Street Trunk and Ivy Lake Sub-trunk, this included full development of the Crystal Lake area. The Downtown Trunk was assumed to include the existing service area with limited in-fill development. The 675mm Trunk on 76 Avenue was assumed to include full development of Countryside South and Country Club Estates.

For the existing 108 Street Trunk and the recently completed Northwest Trunk (along the Highway 43 Bypass), the fully developed service area was assumed to include:

- Existing and future developments east of Bear Creek serviced to the Northwest Trunk and 525mm trunk running through college
- Existing developments west of the creek.

The hydraulic modeling results for the 1:2, 1:5 and 1:10 year rainfall events are illustrated in Figures 7.2 through 7.4, respectively.

7.4 Existing System Assessment

92 Street Trunk

The 92 Street Trunk generally has adequate capacity during the 1:5 year event, with some localized surcharging occurring between 1.8 and 3.0 m below ground. The extent of surcharging is greater during the 1:10 year event, but the hydraulic gradeline is greater than 1.8 m below ground. Thus the risk of basement flooding is considered to be relatively low.

Based on these modeling results, the 92 Street Trunk is considered to be adequate for its current design service area, including the 30 ha of proposed development in

Cobblestone (southeast of 100 Avenue and 92 Street). Hydraulic upgrading is not required for this trunk, based on the simulation results at full servicing.

However, the inclusion of additional development areas east of 92 Street (south of 100 Avenue) or east of 88 Street (north of 100 Avenue) would increase the level of risk and is not recommended. These new development areas should be serviced to the 88 Street Trunk as proposed in the 1995 Wastewater Master Plan. The exception to this would be if a portion of the upstream service area was diverted to the 88 Street Trunk as discussed later in this report.

Ivy Lake Sub-Trunk

The Ivy Lake Sub-trunk runs along the northwest side of Ivy Lake servicing the Ivy Lake and East Crystal Lake areas. As identified in the 1995 Master Plan, the section of this trunk immediately northwest of Ivy Lake is at risk of basement flooding. The HGL is between 1.8 and 3.0 m below ground during the simulated 1:5 year event and within 1.8 m of ground during the 1:10 year event.

It is recommended that a portion of the upstream flows be diverted around this location during ultimate development conditions. This diversion could be made to either the 92 Street Trunk as outlined in the 1995 Master Plan, or to the proposed 88 Street Trunk. If the diversion was made to the 88 Street Trunk, it would free up hydraulic capacity in the 92 Street Trunk for servicing lands immediately east of 92 Street.

Given the identified basement flooding risk northwest of Ivy Lake, the use of this sub-trunk for interim servicing would need to be strictly controlled. It is understood that off-peak pumping to this sewer has been proposed by developers east of 88 Street who wish to commence development. This issue is addressed in Section 10.

108 Street Trunk and Northwest Trunk

Due to proposed multiple trunks for the west side of Grande Prairie, the existing and proposed trunks will be designated based on their specific location. Thus, the term 'West Trunk' used in the 1995, will not be used.

The 108 Street Trunk refers to the 675mm trunk running from the college (approx. 104 Avenue) to 84 Avenue, and includes the downstream 750mm/900mm trunk running through the Mission Heights development. Its current service area includes:

- North Grande Prairie via the Northwest Trunk
- Hidden Valley/Northridge neighbourhoods (south of 116 Avenue) via the 525mm trunk north of the college
- Gateway Commercial area (north of 100 Avenue and west of 108 Street)
- Richmond Industrial area (84 to 100 Avenues and west of 108 Street)
- New developments in Westpoint and Pinnacle Ridge.

The Northwest Trunk was constructed in 1999 to provide servicing to the lands north of 116 Street and west of 100 Avenue, and to divert existing flows originating north of 116 Street (east and west of 100 Street) away from the city centre.

One simulation was conducted for the existing level of development. The simulation indicated that the existing Northwest / 108 Street Trunk system has adequate capacity for the existing level of development. No surcharging was present under a 5 year storm event anywhere in the Northwest/108 Street Trunk.

To assess the hydraulic capacity of these trunks for full development, it was assumed that the Northwest Trunk flows would continue to flow through the 108 Street Trunk, along with the existing connected developments. The rationale for this approach was to include only the areas that are dependent on these trunks for ultimate servicing.

The hydraulic analysis indicated that the Northwest Trunk has adequate capacity with only localized areas experiencing surcharging during the 1:5 or 1:10 year storm. With the exception of the creek syphon locations, the HGL was greater than 1.8 m below ground in all cases. There were a number of surcharged locations along the 108 Street Trunk (north of 84 Avenue) due to the full development of the Northwest Trunk service area. This is not a concern due to the depth of this sewer and the lack of basements in the area.

The full utilization of the 108 Street Trunk in this area confirms the need for an additional trunk sewer for servicing the future development areas to the west. This is consistent with the 1995 Master Plan and servicing alternatives are described in Section 9.

Downstream of 84 Avenue, the 750mm/900mm trunk did not surcharge. Thus the downstream sections of this trunk can accommodate some additional development. The downstream section will be adequate for all infill development. The 375mm line along 68 Avenue will collect all new development and the 900mm/1200mm section of trunk downstream of its tie-in to the 108 Street Trunk, is adequate to handle these additional flows.

Potential new development areas outside the existing city boundary between 104 and 112 Street north of 132 Avenue were also considered for servicing to the existing Northwest/108 Street Trunk. In the event that the two quarter sections considered are developed, the surcharge level in the trunk will increase by up to 0.5m maximum near 100 Avenue and 108 Street. This increased surcharge will still be generally below 3m from ground and should not cause basement flooding.

North End

The 1995 Master Plan indicated the 200mm sewer in the lane east of 100 Street (120 to 128 Avenue), was subject to high degrees of surcharging. The 1995 Master Plan recommended that an I/I reduction program be implemented in lieu of sewer upgrading measures to address this issue. It is understood that the following measures have been undertaken to reduce I/I in the area:

- Partial replacement of MH 171002, 171004, 171016, 170002, 170004, and 170011
- Replaced brick/mortar with riser rings at MH 171013
- Replaced steel shims with concrete riser ring at MH 171045

Flow monitoring has been conducted along this sewer at MH 169901 from August 1 to September 30, 2000. Monitored flows showed 8 L/s peak dry weather flow and 11L/s peak wet weather flow as compared to an available pipe capacity of 29L/s in this area. The monitored flows are well below the pipe capacity, but the wet weather event was very minor (<2 year return period). By comparison, the model shows peaks of over 27L/s in this area for the 5 year Huff storm. Since this sewer is so shallow, any surcharge could cause flooding problems. As such the area remains a concern.

It is understood that several new commercial developments have connected to this sewer or are proposed to connect to this sewer in the near future, including a 100 suite

hotel. To minimize the risk of sewer backup in this area, it is proposed to divert a portion of these flows either around the bottleneck or to the existing 250mm/300mm sewer in the lane west of 100 Street. It is presently proposed to construct a pipe across 100 Street at approximately 124 Avenue to offload excess flows under wet weather conditions to prevent pipe surcharging in the bottleneck near 118 Avenue. Sewer inverts in the 200mm pipe east of 100 Street are approximately 3.5m higher than those in the 300mm line in the lane west of 100 Street, so this type of diversion is feasible.

Downtown Trunk

The hydraulic model indicated that the existing 675mm Downtown Trunk (101 Street from 76 to 89 Avenue) was subject to surcharging during the 1:2 and 1:5 year events and excessive surcharging during the 1:10 year event. This trunk is relatively deep and thus the simulated surcharging does not extend to within 1.8 m of the ground.

Given the relatively low level of confidence in the hydrologic model in the older sections of the City (due to lack of flow monitoring data for calibration), it would be premature to provide specific recommendations regarding the need for trunk upgrading. However, it would be prudent to conduct flow monitoring along this trunk to define the total flows generated from the central and north-central parts of the City.

Country Club Estates and 88 Street / 60 Avenue Trunk

At present, in the southeast of the city, Country Club Estates and Countryside South are serviced by gravity to the pump station near 92 Street and 60 Avenue. Any additional construction north of Countryside South will be similarly serviced on an interim basis. Countryside South and any additional construction to the north will ultimately be serviced by a trunk sewer along 60 Avenue and then north along 88 Street. At present, there is a portion of this sewer that has been constructed along 60 Avenue from 100 Street east to 88 Street, with a gap of 800m between 92 Street and 96 Street. Once this gap section is constructed, only the Country Club Estates region will be serviced by the pump station. Country Club Estates cannot be serviced by gravity to the new trunk sewer as the inverts in sewers near the pump station are 2m lower than those of the new trunk sewer. The City has indicated that in the future the lift station is intended to discharge into the 60 Avenue Trunk, but this is not feasible due to capacity constraints in the existing 900mm section west of 96 Street. Presently, the pump station can accommodate a pumping rate of 57L/s and anything in excess of that will back up in

local service lines along 60 Avenue in Country Club Estates in the MH 340011 area. No wastewater can be stored in the existing portion of the trunk sewer along 60 Avenue east of 92 Street since it is 2m higher than the existing local lines. Clearly, as development progresses in Countryside South and to the north, the condition of this sewer will deteriorate in terms of the level of surcharge as the pump station capacity is exceeded.

To determine the condition of this area, the model was run under 2003 and 2008 development conditions. Under the 5 Year Huff storm, the capacity of the pump station was exceeded in both situations. With 2003 development, there would be 0.6m surcharging in the sewer near MH 340011. By 2008, the surcharge would increase to 1.0m. This puts basements at risk in Country Club Estates near MH 340011. This is not an acceptable situation.

Two options exist for dealing with this surcharging. First of all, Aquatera could build the gap section in the trunk sewer along 60 Avenue so that Countryside South, and any new development, would be serviced to the trunk sewer and no longer drain to the pump station. This would be the preferred alternative and should be completed no later than 2008. It is recommended, if this option is chosen, that the gap section be constructed as soon as possible. Alternatively, Aquatera could consider building an orifice in the existing 60 Avenue Trunk sewer at MH 400018 to restrict the amount of flows that could enter the local sewer near the pump station to 15L/s. This would utilize storage available in the 750/900mm 60 Avenue Trunk sewer near Countryside South, and allow the pump station to deal with the excess flows over a longer period of time, thus reducing the risk of basement flooding in Country Club Estates. This is not recommended due to the high maintenance requirements and high risk of upstream basement flooding if not properly maintained. Furthermore, due to population growth in the northeast, the City would need to construct the 88 Street Trunk to the Ivy Lake area by approximately 2008, and that would necessitate the construction of the gap section in the 60 Avenue Trunk.

7.5 Implementation Plan

Based on the preceding analysis, the following implementation plan is required to address the existing wastewater collection system:

1. A comprehensive rainfall and flow monitoring program should be carried out.
The proposed flow monitoring locations are shown in Figure 6.1.

2. The Crystal Lake Estates area should ultimately be diverted to 88 Street Trunk

3. The 400m wide strip of land immediately east of 92 Street may be allowed to connect to the 92 Street Trunk, but would require storage and off-peak pumping to do so as discussed later in Section 9.

4. Flows in 200mm sewer in lane east of 100 Street should be diverted to the 300mm sewer in the lane west of 100 Street. The proposed diversion should be located at 124 Avenue

5. The remaining section of 60 Avenue Trunk should be constructed as soon as possible.

8.0 Ultimate Servicing of New Development Areas

8.1 East Grande Prairie and 88 Street Trunk

The hydraulic analysis of the existing system confirmed the need for a new trunk sewer to service the new development areas along the east edge of Grande Prairie. This trunk will be referred to as the 88 Street Trunk.

Horizontal Alignment

The preferred alignment for this trunk is along 60 Avenue to 88 Street, then turning north along 88 Street to 100 Avenue. North of 100 Avenue, the trunk should divert to the east to conform to the on-site servicing needs of local development. The proposed alignment of the trunk is shown in Figure 8.1. Several sections of this trunk have already been constructed to provide servicing for new developments in the southeast part of the City. These sections are also shown in Figure 8.1.

Service Area

The proposed service area for the 88 Street Trunk is also shown in Figure 8.1, and includes:

- New development areas south of 60 Avenue that will be serviced by a local pump station
- New development areas between 88 and 92 Streets south of 100 Avenue
- New development areas east of 88 Street north of 100 Avenue, including lands north of 132 Avenue
- Future development areas outside of the City limits east of 88 Street and south of 100 Avenue that can be serviced to the trunk by gravity (88 to 84 Streets)
- Existing and new development areas in Ivy Lake Estates and Crystal Lake Estates that can be diverted to the 88 Street Trunk.

Assuming that the Ivy Lake Estates and Crystal Lake Estates areas are diverted to the 88 Street Trunk, the lands immediately east of 92 Street could be serviced to the 92 Street

Trunk. It is recommended that the developers be given the opportunity to service this area to either trunk. If the new developments are connected to the 92 Street Trunk, it should be noted that the hydraulic capacity is being indirectly provided by the 88 Street Trunk through the diversion of upstream flows.

For trunk sizing and cost estimating purposes, it was assumed that the lands east of 92 Street would connect to the 88 Street Trunk except for the Cobblestone Development. It is, however, recognized that a portion of the lands south of Cobblestone may need to connect to the 92 Street Trunk, in lieu of the 88 Street Trunk, due to potential topographical limitations.

Vertical Alignment

The vertical alignment for this trunk is limited by the sections of trunk that have already been constructed. The missing section along 60 Avenue can have a maximum grade of 0.07%, based on upstream and downstream invert elevations of 644.66m and 644.09m, respectively. The section of trunk along 88 Street should be constructed at minimum grade, at least to 92 Avenue to facilitate servicing of the lands to the west. This will result in deep cuts through a knoll north of 68 Avenue.

The quarter section between 84 and 92 Avenues and west of 88 Street, is very flat and may not be serviceable to either the 92 Street Trunk or the 88 Street Trunk by gravity. This was confirmed in discussions with the developer's representative. The remainder of the service area (except lands south of 60 Avenue as previously noted) should be serviceable to the 88 Street Trunk by gravity.

The vertical alignment north of 100 Avenue is relatively flexible as there is adequate topographic relief available. Trunk depths are therefore expected to be relatively shallow.

Trunk Sizing

The trunk sizing for the remaining stages of the 88 Street Trunk were computed in three ways:

- Scenario 1: Sizing for the new development areas within the current City Limits
- Scenario 2: Incremental sizing to allow the Ivy Lake Estates and Crystal Lake Estates areas to be diverted to the 88 Street Trunk

- Scenario 3: Incremental sizing to allow the future development areas outside the City to connect to the trunk with the offload from Scenario 2 included.

Trunk sizing was established using the hydraulic model for the 1:5 year event. Unfortunately, the existing sections of trunk are slightly undersized based on this design event and proposed service area. The section from 88 Street and 84 Avenue to the existing system south of 68 Avenue was considered for oversizing to a 1050mm line instead of the proposed 900mm. This would reduce the risk of surcharging in the trunk under large storm events and increase the level of protection. Hydraulic modelling indicated that oversizing reduced surcharging from 1.06m to 0.94m above pipe crown at node ET29. At a cost of approximately \$2 million, and a depth to crown of nearly 5m, this is clearly not cost effective.

Cost Estimates

The estimated construction cost of the 88 Street Trunk sewer is shown in Table 8.1.

8.2 West Grande Prairie and 116 Street Trunk

As noted in Section 7, the existing 108 Street Trunk only has adequate capacity to service the existing areas west of the creek plus the existing and future developments east of the creek. It will therefore be necessary to construct a new trunk to service the new development areas west of 108 Street. This is consistent with the findings in the 1995 Master Plan.

Four different servicing alternatives were investigated for the west side of Grande Prairie:

- Constructing a new trunk along 108 Street with a sub-trunk along 100 Avenue, as proposed in the 1995 Master Plan
- Constructing a new trunk around the Richmond Industrial area, servicing the Northwest Trunk by gravity, as proposed as an alternate in the 1995 Master Plan
- Constructing a new trunk on 116 Street, similar to Alternative 2, except servicing the Northwest Trunk through the 108 Street Trunk
- Similar to Alternative 3 except using a pump station to divert excess wet weather flows from the Northwest Trunk to the proposed 116 Street Trunk.

Preliminary pipe sizes were developed for each of the above alternatives, including the local trunks that would provide servicing to each quarter section. Preliminary cost estimates were prepared based on these sizes, the estimated depth of construction and the surface conditions. These construction cost estimates are presented in Table 8.2.

Alternative 1 would require in excess of 1.6 km of trenchless sewer construction due to the intensity of development along 108 Street / Wapiti Road. Also, the existing 375mm trunk on 100 Avenue would have to be twinned (presumably on 108 Avenue) to provide servicing to the new development areas west of 108 Street. Based on the high costs of these trunks, this alternative was ruled out.

Alternatives 2, 3 and 4 are very similar in concept, and the only different variable is how the ultimate flows in the Northwest Trunk would be handled. Deepening and oversizing the 116 Street Trunk as proposed in Alternative 2 would be very costly and may not be necessary based on the existing system modeling. Allowing the Northwest Trunk flows to continue to flow through the 108 Street Trunk is clearly the least expensive alternative but has some inherent risks relative to the magnitude of the future wet weather flows. To address these risks, a portion of these flows could be diverted to the 116 Street Trunk via a pump station and forcemain as proposed in Alternative 4. The advantage of this alternative is that the pump station and forcemain could be postponed for several years and need only be built if necessary. However, Alternative 4 would be reliant on the operation of the pump station during a major wet weather event which increases the operating risks.

Based on discussions with Aquatera and a review of the overall costs and risks, Alternative 3 is recommended. Should future flow monitoring indicate that the 108 Street Trunk cannot accommodate the ultimate system flows from the Northwest Trunk, hydraulic upgrading of the 108 Street Trunk could be undertaken.

8.3 Southwest Grande Prairie

The ground elevation drops sharply south of 68 Avenue on the west side of the creek. It is understood that the northerly 16.28 ha of the O'Brien Lake subdivision on the west side of 108 Street has been serviced to the 375mm sub-trunk on 68 Avenue. Servicing

the remainder of this area and the lands to the east (between 108 Street and the creek) to the 375mm sub-trunk cannot be accomplished because of these grade constraints.

Four alternatives were investigated for servicing the remaining areas south of 68 Avenue:

- Constructing a local sewer approximately 400 m south of 68 Avenue and connecting to the 1200mm trunk immediately west of the creek at MH 200002
- Diverting the 116 Street Trunk 400 m south of 68 Avenue and connecting to the 1200mm trunk at the same location
- Constructing a pump station with a forcemain discharging to either the existing 375mm sub-trunk or future 116 Street Trunk on 68 Avenue
- Constructing a trunk alignment outside the city providing direct servicing to the O'Brien Lake area and requiring a smaller collector for the Keddy/McLeod/Tink area east of 108 Street.

Based on the existing invert elevation of 644.66m at MH 200002 immediately west of the creek, either of the first two alternatives are feasible. A local trunk would allow ground elevations of 651.46m at the east end of O'Brien Lake to be serviced. This would be adequate to service the remainder of the subdivision as well as the lands to the east. With the flatter slope in the 900mm 116 Street Trunk, ground elevations of 649.76m could be serviced at the connection. This would also allow for complete servicing of the subdivision. As well, a good portion of the quarter section to the west and the lands to the east of O'Brien Lake could also be serviced. A pump station could also be feasible, but would restrict the amount of future development to the west that could be serviced without upgrading the pump station. There would also be an operating risk generated in the case of pump station failure. Additionally, the lands east of 108 Street would still require servicing as their elevations are lower than those around O'Brien Lake and thus, could not be serviced by the pump station. This would require additional costs to develop a local trunk to service these areas. The fourth alternative would also be feasible, but could only be assessed in cost when considering the total cost of the 116 Street Trunk sewer alignment outside the city against other proposed alignments.

Table 8.3 gives cost estimates for the first two alternatives. The south alignment of the 116 Street Trunk to approximately 64 Avenue is the least expensive of the in-city options. This is recommended if an in-city alignment for the 116 Street Trunk is chosen, but can

only be compared to an alignment outside the city in terms of total trunk sewer construction costs. This is discussed in the next section.

8.4 Service Area for 116 Street Trunk

The proposed service area for the 116 Street Trunk includes all future development areas west of 108 Street that can be economically serviced to the 116 Street Trunk. To establish the most economical service area, a number of invert elevations were considered for the trunk at 116 Street and 100 Avenue. The recommended invert elevation at this location is 656.8 m, which should allow all the lands west of 108 Street not currently serviced to be serviced to this trunk, except for areas below 663m in the College Lands area just east of 108 Street to the south of the creek. This leaves approximately 30 ha of land that must be serviced to the 108 Street Trunk.

To maximize the areas that are serviced to the 116 Street Trunk, it may be necessary to re-design the on-site sanitary servicing and local trunks north of 100 Avenue and west of 108 Street. As much as possible, these areas should be serviced to 116 Street and 100 Avenue. These areas can be serviced through the 375mm 100 Avenue Sub-trunk on an interim basis, as discussed in Section 10.

Horizontal Alignment

Five proposed alignments were considered for the 116 Street Trunk. As shown in Figure 8.2, all five alternatives follow 116 Street from 100 Avenue to 84 Avenue, and continue as follows:

- 1a) 84 Avenue east to 113 Street, then south and east to 112 Street and 76 Avenue, then east to 108 Street, then south to 68 Avenue and East to MH 200002
- 1b) Same as 1a, but continues south past 68 Avenue to 64 Avenue to service O'Brien Lake
- 2a) 84 Avenue east to 108 Street, then south to 68 Avenue and east to MH 200002
- 2b) Same as 2a, but continues south to 68 Avenue to 64 Avenue to service O'Brien Lake
- 3) Continues south along 116 Street to 68 Avenue, then east to 108 Street, south to 60 Avenue, and finally east to the wastewater treatment plant.

In the southwest part of the City, the proposed horizontal alignment 1a generally follows the alternate alignment proposed in the 1995 Master Plan. This includes sections along the 84 and 68 Avenue and 116 and 108 Street rights-of-way, as well as through the Westpoint subdivision and immediately north of the Pinnacle Ridge development. Aquatera and the City of Grande Prairie have expressed some concerns regarding the alignment through these developments, so alternate alignments along arterial roadways (84 Ave to 108 Street then south and 116 Street south to 68 Avenue) were considered.

Alignment 2a is feasible using trenchless construction technology. This alignment is more expensive based on the long length of trenchless construction of approximately 1600m. However, this option may be preferred due to its greater flexibility for interim servicing.

Alignments 1b and 2b deviate south of 68 Avenue to service the O'Brien Lake subdivision as previously discussed. This will remove the need for a collector for O'Brien Lake and reduces costs over alternatives 1a and 2a respectively. All five of these alignments will, however, require a pump station and forcemain to ultimately service the quarter section west of O'Brien Lake to the pipe servicing O'Brien Lake. This quarter section is presently outside the city limits.

Alignment 3 has fewer conflicts with existing infrastructure / development. However, the alignment would require an inverted siphon crossing a watercourse draining to Bear Creek, along with extensive sections of engineered fill. Thus the construction of this alignment would be more complex than the other alignments. Additionally, this alignment would need to be built in one stage as no interim servicing is feasible downstream of 100 Avenue. This alignment would, however remove the need for a pump station and forcemain in the quarter section west of O'Brien Lake, and would also service O'Brien Lake. The only additional need would be a smaller collector than the proposed one for O'Brien Lake, to service the Keddy/McLeod/Tink areas east of O'Brien Lake.

Based on the servicing costs in Tables 8.4 through 8.9, the preferred alignment for the 116 Street Trunk is #3. Please note that due to uncertainty regarding the elevation contours in the area that Alignment 3 passes through, that the estimated cost could increase based on future verified contours. It is estimated that contours will be raised by

1m, thus increasing cost incrementally to reflect this change. The costs presented here only take into account the cost at present. Please note that these tables contain construction dates that are presented in Section 9. These construction dates are based on staging for the different alignments. Based on this, Table 9.2 in the next section gives the net present value based on this staging. Based upon net present value, the preferred alignment becomes Alternative 1b. This is the recommended alignment for the 116 Street Trunk based on all information considered.

116 Street Trunk Sizing

The recommended sizing for the 116 Street Trunk is shown in Figure 8.2. This sizing is based on hydraulic modeling results for the 1:5 year rainfall event. The design flows, trunk sizing and slope are listed in Tables 8.4 through 8.9 with cost estimates for each section.

9.0 Interim Servicing of New Development Areas

9.1 East Side

Servicing Alternatives

Interim servicing for new development areas in east Grande Prairie can be provided in three ways:

1. Construction of the 88 Street Trunk in stages, with development progressing from south to north as the trunk is constructed;
2. Development proceeding prior to the construction of the 88 Street Trunk, with interim servicing to the 92 Street Trunk or Ivy Lake Sub-trunk; or
3. Construction of 3 to 5 km of the 88 Street Trunk to service a number of development areas simultaneously.

The first alternative has no interim “throw away” costs and involves the least level of risk in terms of investment in municipal infrastructure (little risk if development slows dramatically). By connecting to the 88 Street Trunk, it also has a relatively low risk to the existing sewers. Unfortunately, it is the least acceptable alternative to developers in the northern portions of the service area.

The second alternative would allow development of individual areas to proceed independent of downstream areas and would be the least expensive alternative for any individual developer. However, it is also the most risky alternative in terms of excessive flows in the downstream system, especially if connection is made north of Ivy Lake. As a result of this risk, a very conservative approach needs to be taken for interim servicing, which will result in high throw away costs.

The third alternative has the highest front-end cost, and the greatest risk in terms of investment in municipal infrastructure. However, it has no throw away costs and would allow several areas to develop concurrently.

The key element in the selection of an interim servicing concept is the cost that developers and/or Aquatera would have to spend to facilitate development. Proposed design criteria and estimated costs for interim servicing to existing trunks are described below.

Interim Servicing to Ivy Lake Sub-trunk

Based on the existing risk of basement flooding in the Ivy Lake Sub-trunk, interim servicing should be limited to discharges during off-peak periods only. To facilitate this, it will be necessary for the developer(s) to provide on-site storage and pumping facilities. Pumping would only be permitted between midnight and 6:00 am, with the total flow (existing trunk low flows plus pumped flows) no greater than the peak dry weather flow in the Ivy Lake Sub-trunk. Based on the DWF modeling results, the maximum pumping rate to this sub-trunk would be 14 L/s.

The volume of storage required for each lot developed should be estimated as follows:

- 1 day of domestic wastewater (assume that majority of flow generated between 6:00 am and midnight) * 275L/cap/day * 3.2 persons/lot = 880L
- 18 hours of inflow/infiltration * 0.28 L/s/ha * / (10 lots/ha) * 3600 sec/hr = 1800 L

Thus, the required storage per lot should be 2680 L or 2.7 m³. This is slightly higher than the 1.6 m³/lot currently being used by the City of Edmonton for storage systems in deep trunks. The assumption that the I/I rate is maintained for 18 hours is conservative, but is recommended given the risks involved. This volume could be provided in either concrete storage tanks or in sections of the 88 Street Trunk. The latter approach is used extensively in Edmonton in its sanitary servicing strategy for new development areas.

By restricting the timing of off-peak pumping and the pumping capacity, the maximum lands that can be serviced can be estimated. Based on an allowable pumping rate of 14 L/s for 6 hours, a total of 302 m³ could be pumped. Based on 2.7 m³/lot, a total of 112 lots could be developed.

Interim Servicing to 92 Street Trunk

The interim servicing concept for the 92 Street Trunk would be similar to the Ivy Lake Sub-trunk, with 2.7 m³ of storage required for each lot developed. Based upon the same methodology as for Ivy Lake, the total off-peak pumping rate allowed to the 92 Street Trunk would be 16.5L/s. Based on 2.7m³/lot, this would allow for 132 lots to be developed. It must be noted that these 132 lots include the allowed 112 lots to the Ivy Lake sub-trunk as all Ivy Lake flows proceed down the 92 Street Trunk. As such, if 112 lots are serviced in this manner near Ivy Lake, only an additional 20 lots can be serviced to the 92 Street Trunk directly.

Proposed Interim Servicing Concept

Based on discussions with developers and their consultants during this study, it is understood that at least two developers are considering spending up to \$0.5 million for an interim pump station and forcemain to facilitate interim sanitary servicing. It is also understood that Aquatera has committed approximately \$200,000 to construct a forcemain along 100 Avenue from 88 Street to 90A Street, in order to bypass pumped flows around Ivy Lake. The above infrastructure would only provide interim servicing, resulting in combined throw away costs in the order of \$1 million.

Based on the total costs of constructing the 88 Street Trunk to 116 Avenue of less than \$3.8 million, spending \$1 million in throw away costs is not practical. It is therefore recommended that Aquatera work with local developers to share the high front-end costs associated with constructing the 88 Street Trunk (Alternative 3 above). The following concept is proposed for discussion purposes:

- Developers at the south end of the service area should be required to construct the section of trunk abutting their developments
- Other developers would be required to front-end an equivalent cost to their interim servicing costs to connect to the 92 Street Trunk or Ivy Lake Sub-trunk
- Aquatera / City would provide the amount already budgeted for the 100 Avenue forcemain
- Aquatera / City would fund the remaining costs from the Revolving Trunk Fund and/or other levies / connection charges
- The developers and Aquatera / City would then be reimbursed by future developments through levies or connection charges.

Should area developers and Aquatera / City not be able to negotiate a suitable agreement for cost sharing the trunk, the following approach is recommended:

- Developers at the south end of the service area should be required to construct the section of trunk abutting their developments
- Aquatera should fund the construction of the 88 Street Trunk in stages based on funds available in the Revolving Trunk Fund and/or similar funds
- The City / Aquatera should allow developers to provide interim servicing to the 92 Street Trunk or Ivy Lake Sub-trunk.

The staged improvements for immediate 5 year, 10 year, and ultimate construction for the 88 Street Trunk can be found in Figures 9.1, 9.2, 9.3 and 9.4, respectively.

9.2 West Side

Interim Servicing Concept

Interim servicing to the west side of Grande Prairie is relatively straight forward based on the available capacity in the 100 Avenue Sub-trunk and 108 Street Trunk. The following interim servicing concept is proposed:

- Continue using the 108 Street Trunk for servicing the lands west of Bear Creek, as well as the lands east of the creek via the Northwest Trunk
- Extend the 100 Avenue Sub-trunk to 116 Street and use this sub-trunk to service the lands west of 116 Street or north of 100 Avenue on an interim basis
- Construct the 116 Street Trunk in stages based on the projected utilization of the 100 Avenue Sub-trunk and 108 Street Trunk
- Monitor the flows in the 100 Avenue Sub-trunk and 108 Street Trunk to confirm the required timing of the 116 Street Trunk
- Utilize an interim pump station to service the lands west of 116 Street to the 100 Avenue Sub-trunk, eventually converting to gravity servicing to the 116 Street Trunk
- Consider using the available volume in the 116 Street Trunk for interim storage, with pumping to existing trunks at controlled rates
- Interim discharge locations should include the 375mm 100 Avenue Sub-trunk and the 375mm 68 Avenue sub-trunk.

Timing of 116 Street Trunk

The required timing of the 116 Street Trunk (south of 100 Avenue) is dependent on the available capacity in the 100 Avenue Sub-Trunk for interim servicing. This sub-trunk has a capacity of 99 L/s between 108 and 116 Streets. Using the proposed design standards, this capacity would be able to service approximately 140 ha of commercial, industrial and residential development. Accounting for the 50 ha that will be serviced to this sub-trunk on a permanent basis, roughly an additional 90 ha can be serviced on an interim basis.

Based on the presumed timing of development outlined in Section 9.0, this interim capacity is expected to be exceeded by approximately 2007. This can be seen in Table 9.1. By 2008, the pipe would approximately be 15L/s over capacity compared to the 38L/s available at 2003 development levels. Thus, it would be necessary to complete construction of an additional section of the 116 Street Trunk by approximately 2007.

At this time, it would depend on which of the alternatives for the 116 Street Trunk was chosen. For each alternative, the Airport collector would need to be constructed in 2005. The north sections of the 116 Street Trunk (north of 100 Avenue) would be built starting in 2012, and the servicing to the quarter section west of O'Brien lake via pump station/forcemain would need to be built in 2020 for alternatives 1a, 1b, 2a, and 2b. The remaining servicing for each alternative is as follows (refer to Section 8.4 for list and details).

Alternative 1a or 1b

Initially, a section from 116 Street and 100 Avenue to the existing 375mm pipe at 108 Street and 68 Avenue could be constructed with segments constructed in 2005 through 2007. This would cost approximately \$9.1 million. The final stage in this alternative would be to connect to MH 200002 at 102 Street and 66 Avenue. This would need to be constructed by 2013 approximately with sections constructed in 2011 through 2013. This would cost \$2.6 million if the trunk was taken to 64 Avenue to O'Brien Lake, or \$2.9 million for the trunk (\$1.8 million) and O'Brien collector (\$1.1 million).

Alternative 2a or 2b

Initially, a section from 116 Street and 100 Avenue to the existing 750mm trunk sewer at 108 Street and 84 Avenue could be constructed by 2007 in sections from 2005 through

2007. This would cost \$6.2 million. It would also be necessary to construct an orifice that would only allow 40L/s to enter the 750mm trunk sewer, in order to reduce the risk of surcharging downstream. The next stage would continue south to 68 Avenue at the 375mm line. This would need to be built by 2010 at a cost of \$5.1 million. The final stage to MH 200002 would be constructed by 2013. The cost would be \$2.6 million for the south alignment to O'Brien Lake, or \$2.9 million for the trunk and O'Brien Lake collector.

Alternative 3

Since this alternative gives no real option for interim servicing, the entire trunk from 116 Street and 100 Avenue to the wastewater treatment plant would need to be constructed. This would cost \$11.8 million by 2007. The collector for areas east of O'Brien Lake would need to be constructed after 2013 as they developed. It is estimated for a 2015 construction date with a cost of \$630,000.

Net present value analyses were calculated for the three different staging alternatives. The results can be found in Table 9.2. Based on these results, alternative 1b results in the cheapest net present value and becomes the preferred alternative.

In the event that the front-end costs of this alternative are too high, the project could be delayed. This would result in higher flooding risks along existing trunk sewers and potential flooding losses.

Staging plans for the 116 Street Trunk construction can be found for immediate, 5 year, 10 year, and ultimate horizons in Figures 9.5, 9.6, 9.7 and 9.8, respectively.

10.0 Funding Mechanism

10.1 Introduction to Service Connection Fees

To provide growth related sewer infrastructure in and around the City of Grande Prairie, Aquatera is considering charging new service users with a service connection fee. The service connection fee would be charged to any developments that require new or expanded sewer service. The service connection fee would be charged to the developer at the time that the development connects to an Aquatera sewer service. As such Aquatera would be required to front-end construction of sewer infrastructure facilities in new growth areas in and surrounding the City of Grande Prairie in anticipation of future development.

This section of the report considers the development of service connection fees used to recover sewer infrastructure constructed by Aquatera to support service growth. This report describes future growth areas, the infrastructure required to service these areas, the service connection fees required to recover infrastructure investments as well as the impact of infrastructure front-ending on Aquatera.

10.2 New Growth Service Area

In order to establish sewer service connection fees in the new development growth must be defined. These areas have been identified and are itemized in terms of area in Table 10.1. Future growth areas are also shown in Figure 10.1. In estimating areas that are available for future development allowances for non-developable lands within the new growth areas has been estimated. Table 10.1 provides an estimate of this net development area by deducting allowances for municipal reserves and environmental reserves from gross land areas. The following summarizes the total area in new growth areas, non-developable allowances and resulting development areas:

	Area (ha.)
Gross Area	3,006.90
Municipal Reserves	202.35
Other Reserve Allowances	183.01
Net Developable Area	2,621.54

On reviewing the new grow service area it should be noted that the development area is very large and capable of supporting growth that is approximately 1.8 times greater than the current population of the City of Grande Prairie. We estimate that the new grow service area is capable of supporting a population of over 72,000 based upon development densities used in this study. Base upon current growth rates this represents a service area that will not be fully developed for 60 to 80 years into the future.

10.3 New Growth Service Area Land Use

Development within the new grow service area can take various forms which will directly impact the demand for water and sewer services. We estimated the future form that development will take and as such have categorized all new growth service areas into industrial, commercial, single-family residential and multifamily residential land use classifications. Table 10.2 outlines land use in new growth service area. The following summarizes the net development area for each land use classification:

	Net Area (ha.)
Industrial Land Area	624.86
Commercial Land Area	402.54
Single Family Land Area	1,232.62
Multi Family Land Area	361.52
Net Developable Area	2,621.54

10.4 Sewer Demand Factors

As previously stated differences in land use will results in variations in demand for sewer services. That is, a hectare of multi family development will require different sewer service support than a hectare of single-family development. The following table outlines sewer demand factors for each land use classification defined earlier. The table considers sewer service demand based upon developments relationship to single family

and sewer demand factor of 1, while a hectore of industrial land development is considered a demand equivalent to single family development and a hectare of multi-family development is considered to have 4 times the sewer service as a hectare of single family development.

Land Use	Sewer Demand Factor
Industrial Land Use	1.00
Commercial Land Use	2.00
Single Family Land Use	1.00
Multi Family Land Use	4.00

10.5 Development Densities

In order to determine development charge values we have assumed a common development densities for each land use classification. These development densities will be used to determine the number of service connections that can be anticipated for each land use. The following table outlines common development densities that have been used in the development of water and sewer service connection rates.

Land Use	Description
Single Family	10 lots per ha.
Multi Family	4 complexes per ha.
Commerical	3 lots per ha.
Industrial	3 lots per ha.

10.6 New Growth Area Sewer Infrastructure

Table 10.3 outlines sewer related infrastructure required to support growth in the new growth service area. Sewer infrastructure has been classified into new growth area earmarked projects as well as a share of system wide sewer infrastructure improvements. Sewer capital projects that are earmarked specifically to support the new growth service area, total \$19,959,000 while the proportionate share of system wide sewer infrastructure improvements total \$16,200,000. Again the allocation of system wide sewer improvement has been based upon population of the existing and new growth areas. The total of all sewer capital improvements in the new growth area is \$36,159,000 (see table 10.4).

10.7 New Growth Area Sewer Service Connection Fees

Based upon the growth service areas as defined, projected land uses, development densities and related service demand factors for sewer we have developed sewer service connection rates to recover growth area sewer infrastructure costs. The sewer rates that have been developed assume that all growth area sewer related infrastructure is recovered through the application of costs over a single collection basin. Table 10.5 in outlines the sewer connection rates that are applicable for each development area. The rates for each land use are outlined as follows:

Land Use	Sewer Fee Per Connection
Industrial Land Use	\$ 2,893.95
Commercial Land Use	\$ 5,787.91
Single Family Land Use	\$ 869.06
Multi Family Land Use	\$ 8,690.55

Again it should be noted that the rates state above are those rates required to meet current infrastructure costs assigned to the new development areas. Any change in infrastructure plans or costs will result in the need to amend sewer service connection rates.

10.8 Impact of Service Connection Rates on Aquatera

We have considered the impact of front-ending construction and subsequently collecting service connection fees on Aquatera finances. Front-ending growth related infrastructure can be onerous to a utility particularly when significant infrastructure must be front-ended and when related development recovery is slated to occur over a lengthy period. Both of these circumstances exist in Aquatera's case. In order to permit area growth significant infrastructure investment is required near immediately. Over a 10-year period Aquatera will make over \$23.3 M in infrastructure related disbursements (both earmarked water and sewer investments and a proportionate share of system wide water and sewer infrastructure improvements) to serve development in the new growth area. As previously noted recovery of these funds is made more difficult as the size of the development being served will not be fully developed for 60 to 80 years.

In our analysis we have assumed that the previously mentioned rates are applied to the growth service area and also to any infill development within exist service areas contained within the City of Grande Prairie. Our analysis assumes that approximately 744 new development permits will be subject to sewer service connection fees in 2005 and that this level of development will grow by 3% per annum over our 10-year review time frame. We have further assumed that all fees gained from sewer service connection fees will be used to finance both earmarked growth area infrastructure requirements as well as the growth areas proportion share of system wide infrastructure improvements.

Table 10.6 outlines the cash flow impact of sewer service connection proceeds and sewer growth area infrastructure disbursements. As shown in the table, receipts for the growth and infill areas are insufficient to finance sewer related infrastructure investments. The financial requirements of sewer infrastructure investments in the new growth area far exceed receipts that are generated by development. By the end of the 10-year review period the cash impact on Aquatera is estimated to be \$16.9M deficit.

We have considered the financial impact of water and sewer proceeds and disbursements jointly. Table 10.7 shows the financial impacts of receipts and disbursements on a combined water / sewer basis. As shown in the table the magnitude of the sewer cash flow deficit has been reduced by the favourable position of the water fund surplus however over the 10-year review period a significant deficit remains.

Based upon our analysis water and sewer service connection fees that will recover the costs of new growth infrastructure are inadequate to support the cash flow demands associated with infrastructure construction. That is the funding of infrastructure investment requirements service connection fees and an interim financing source. Based upon our analysis it is likely that Aquatera would require as much as \$8 M to \$10 M in interim financing for a 10 to 15 years while water and sewer infrastructure improvements are being constructed and new growth area service fees amass.

10.9 Sensitivity Analysis of Service Connection Rates

During the course of our review we were requested to determine service connections fee impacts if service connection fees were to meet water and sewer infrastructure investment without Aquatera undertaking any interim financing arrangements. Table

10.8 provides cash flow impacts that result from a lower water service connection fee rate and a higher sewer service connection fee rate. The service connection fee rates that best meet cash flow requirements are outlined below.

Land Use	Water Fees	Sewer Fees	Total Fees
Industrial	\$ 1,665.00	\$ 8,325.00	\$ 9,990.00
Commercial	\$ 4,328.98	\$ 16,650.00	\$ 20,978.98
Single Family	\$ 500.00	\$ 2,500.00	\$ 3,000.00
Multi Family	\$ 5,000.00	\$ 25,000.00	\$ 30,000.00

As indicated in Table 10.8, while interim financing cannot be completely eliminated (year one and two infrastructure investments are far greater than development proceeds stemming from any near reasonable service connection fee rates), the 10-year capital infrastructure cash flow requirements can be better met.

We have attempted to demonstrate the impact on cash flow over the 10-year review period if the relationship between water and sewer service connection fees are maintained and overall cash requirements are managed as best as possible. In this regard a continuum of possibilities exist. In our analysis we have attempted to highlight rates that we believe are near the extreme ends of service connection fee limits. In this regard we have illustrated impacts from a maximum combined fee rate of \$3,000 and a minimum fee rate of \$2,300 based upon single family development. Table 10.13 of Appendix A outlines the cash flow impacts of the combined maximum \$3,000 rate and table 10.14 outlines impacts associated with the minimum \$2,300 single family rate.

Our analysis of the maximum rate situation indicates that a combined \$3,000 single-family rate can reduce interim financing down to a minimal level of \$4.8M (as indicated in Table 10.9). However the service connection fee rates result in a very large surplus at the end of the 10-year review period of \$16.2 M. As such we are concerned that the maximum rate of \$3,000 would be judged as excessive if reviewed by an external objective party. The service connection water and sewer fee rates related to the combined \$3,000 maximum are outlined below.

Land Use	Water Fees	Sewer Fees	Total Fees
Industrial	\$ 4,254.82	\$ 5,724.36	\$ 9,979.18
Commercial	\$ 11,062.47	\$ 11,468.52	\$ 22,530.99
Single Family	\$ 1,277.72	\$ 1,722.00	\$ 2,999.72
Multi Family	\$ 12,777.24	\$ 17,220.00	\$ 29,997.24

As indicated in Table 10.10 a combined minimum rate of \$2,300 will have a marginally higher interim financing requirement (\$7.2 M) for Aquatera. It should be noted that interim financing requirements are short-lived in that by the end of the 10-year review period a surplus of only \$6.0 M will have amassed. While this service connection fee rate places some stress on Aquatera's financing abilities we believe that the rate is a far more reasonable response in rate establishment. The service connection water and sewer fee rates related to the combined \$2,300 minimum are outlined below.

Land Use	Water Fees	Sewer Fees	Total Fees
Industrial	\$ 3,261.54	\$ 4,395.60	\$ 7,657.14
Commercial	\$ 8,479.94	\$ 8,791.20	\$ 17,271.14
Single Family	\$ 979.44	\$ 1,320.00	\$ 2,299.44
Multi Family	\$ 9,794.40	\$ 13,220.00	\$ 23,014.40

10.10 Service Connection Rate Conclusion

As indicated earlier, front-ending development infrastructure can be onerous to a utility particularly where front-end investments are large and when infrastructure recovery is slated to occur over an extended period. It is our opinion that while the service connection rates for water (\$644.88 single family rate) and sewer (\$869.06 single family rate) will fully recover water and sewer new growth area infrastructure investments, the period of recovery (60 to 80 years) is unreasonable. As such we believe that Aquatera should consider service connection fees that closer to the minimums and maximums established in our sensitivity analysis. These rates should not only be determined based upon financial requirements and financing limitation but upon the market impact of rates on development in the City of Grande Prairie and surrounding area.

11.0 Wastewater Best Practice

11.1 Introduction

Most efficient and effective organizations have adopted a continuous improvement strategy around their operations. A critical element of this improvement strategy is to gain traction from the experiences of other industry partners to identify and adopt leading industry practices.

Aquatera wants to ensure at the outset of its operation that the leading practices of other wastewater utilities have been considered. This section of the report outlines a number of leading practices that have been gathered through review of various Canadian and U.S. utilities, as well as management studies of utilities and similar industries. Topics include:

- Good Governance in Water Utilities
- Reducing the Impact of Variations in Financial Plans
- Adoption of s Strategic Planning Framework
- Strategies for competitive advantage
- Best Practices for Maintenance Work Orders
- Wastewater Environmental Objectives
- Total Productive Organization
- Program Driven Maintenance
- Treatment Plant Performance Dashboard
- Selecting Wastewater System Rehabilitation Technologies

Each topic area is discussed more fully below.

11.2 Good Governance in Wastewater Utilities

Good governance is about achieving the desired results in the right way. The importance of good governance is widely accepted yet in many utilities the principles of governance are not clearly articulated and the relationship between utility stakeholders is unclear. Good governance is characterized by a set of ranked principles that guide

decision-making processes and management practices. Principles of good governance and the prioritization of each principle may vary between organizations and jurisdictions.

The governance principles are listed as follows:

- Public safety is the paramount principle
- Public accountability for decisions related to the wastewater system
- Effective exercise of owner's oversight responsibilities
- Competence and effectiveness in the management and operation of the system
- Full transparency in decision-making.

Some of the most frequently occurring good governance principles in wastewater management include:

- Protection of public health and safety
- Environmental protection
- Accountability for stewardship and performance
- Transparency
- User participation
- Balancing equity, efficiency and effectiveness in performance
- Financial sustainability.

11.3 Reducing the Impact of Variations in Financial Plans

The financial planning process for wastewater utilities, can vary significantly from the actual results.

- Extremes in wet or dry weather can have a dramatic impact on residential and some commercial customers water consumption habits and utility revenues and hence the amount of wastewater flow.
- Over heated development growth can expedite capital infrastructure plans.
- Regulatory changes can result in new infrastructure, rehabilitation or operational requirements.
- Capital project variances can result in unforeseen of wastewater flow..

The development and maintenance of various reserve funds can provide the continuity for these unpredicted events. Several reserve funds that should be considered by a utility includes:

- **Rate Stabilization Fund** – The rate stabilization fund is used to minimize the rate impact of extraordinary cost increases such as large increases in debt borrowings or related interest rates. Normally the rate stabilization fund is drawn upon when rate increase requirements exceed defined hurdle rate percentage. Typically rate stabilization funds are replenished, up to a defined limit, through an allocation of a part of any annual operating surpluses. Aggressive management and utilization of the fund permits development and adoption of long-range rate strategies and avoids the political tension associated with short range rate planning.
- **Operating Reserve Fund** – The operating reserve fund is used as a point of fall back for any years operating losses that may occur as a result of temporary customer consumption drops and the like. The fund is established and replenished when operating surpluses arise. The fund typically is set marginally greater than the largest history loss position providing support for a single catastrophic operating year and multiple years where operations are marginally below predictions.
- **Insurance Reserve Fund** – Most utilities are self-insured. This fund is used to pay for ordinary losses incurred through operations etc. that are a result of accident, theft and usual liability claims (Note extra-ordinary claims are usually covered through insurance held with private insurance companies). This fund is created and replenished through self applied insurance premiums levied annually as part of operating budgets. The fund is typically set according to long term claims history with reserve adjustments made through periodic review of the asset base being insured.
- **Renewal and Replacement Reserve** – This fund is established to replace and rehabilitate existing infrastructure when the asset has reached the end of its useful life. The reserve is established and replenished through depreciation charges in the annual operating budget and through special requisitions to budget that recognize that replacement assets may have escalated from the time they were originally booked. Reserve funds are usually earmarked for use through the annual capital budget process. The creation and use of these funds

are also important to the development and adoption of long range financial plans and rate strategies.

11.4 Adoption of a Strategic Planning Framework

A strategic planning framework involves establishing a set of linked cyclical management processes, which together form a planning, implementing and measurement tool set to satisfy both long term as well as short term needs. The following illustration shows typical strategic plan elements.



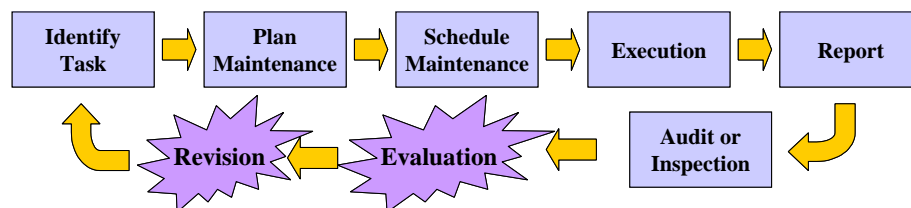
11.5 Strategies for Competitive Advantage

Deregulation of the wastewater environment and with it the added pressures of opened market competition places greater and greater emphasis on those actions that can have immediate positive impact on the bottom line. The maintenance and reliability of wastewater operations is coming under minute scrutiny. Modern tools and systems have created maintenance and operational opportunities that were not possible in the

past. The following is a listing and explanation of some of the identified maintenance “best practices” being used in the utility industry.

- **Benchmarking** – One tool used effectively today by many utilities is that of benchmarking. The process of benchmarking can take many forms. One method of benchmarking is to compare oneself to a single leading wastewater system. Using indicative data from operations and maintenance differences are highlighted for potential opportunity. This approach also has the appeal of allowing key management and staff to visit the maintenance and operations of the benchmark wastewater system to understand first hand cultural and environmental factors that play a vital role in achieving a highly efficient operation. Another approach is to use an outside firm to provide comparison data across many wastewater system operations so that standards can be developed and anomalies from these standards highlight. This approach has the advantage of quickly highlighting potential target areas.
- **Work Flow Control** – Workflow control involves flow-charting, measurement, analysis and assessment and standardization of the steps involved in typical maintenance and operational activities. These activities might include what transpires in maintenance callouts for emergency items or routine maintenance activities like sewer flushing or hydrant testing.

A controlled maintenance work stream should involve the following steps.



- **Preventive Maintenance** – Everyone has a preventative maintenance program however most programs are over burdened with wants versus needs. The result is that the preventative program may not be applied to the “right” facilities and infrastructure, preventative maintenance routines are not being fulfilled because of “higher priority” needs, preventative strategies have not be kept current and

scheduled down time is viewed as intrusive rather than a planned outage. In most instances preventive maintenance routines have to be pared and non-effective tasks removed and other tasks added.

- **Operations Responsibilities** – Maintenance of operating equipment in a treatment plant etc. is an operating responsibility. The degree to which equipment or a facility is maintained is determined and approved by operations. The expertise and capabilities of the maintenance force and its management determine the quality and quantity of work performed. In many cases in today's competitive market operators are used to perform basic maintenance tasks where skill is not a factor.

- **Materials Management** – The most efficient plants recognize that management of the materials used in the performance of maintenance is a definite best practice. Practice considerations include locating maintenance inventory relative to the work areas, spare part delivery schemes, vendor managed inventories etc.

- **Diagnostics** – Establish the use of diagnostics with follow-up corrective actions associated with prior notification of failure leads to improved reliability and bottom line performance.

- **Training** – Aging workforces, unaccustomed to change are finding their skills are deficient in providing the level of repair and diagnostics needed for today's plant equipment.

11.6 Best Practices for Maintenance Work Orders

There is an aversion to developing and using standard procedures for maintenance tasks. Arguments are presented that the workforce is mature, performing the same tasks for an number of years “successfully” and as such there is no compelling reason to have detailed procedures. However the inefficiencies that existing within and surrounding the every day maintenance work order offer many improvement opportunities. The following are some best practices surrounding maintenance work orders.

- **The Risks and Benefits of an Aging Workforce** – Most utility organizations have a very experienced maintenance workforce. This experienced workforce does not require maintenance procedures or protocols to handle many of the utilities critical infrastructure, plant systems and equipment maintenance requirements. Because of this ready availability of experience and knowledge it is important that maintenance procedures and protocols be documented and preserved now. Human resource studies have shown that a large knowledge drain will occur in the work place as baby boomers exit the workforce. Utilities are not immune from the experience drain and as such establishing robust maintenance procedures and methods now will lessen the impact of the loss of knowledge and experience that is occurring.

- **Safety Considered On Every Work Order** – Primary information required for every maintenance task or work order is the safety issues associated with completion of the task. Safety worksheets should be provided and linked to every work order to ensure that maintenance staff, regardless of experience level, fully understands how to accomplish the maintenance task in a safe manner.

- **Linkage of Tools, Materials and Equipment on Every Work Order** – Studies have shown that a typical maintenance worker only spends 24% of their workday performing maintenance work. A good portion of residual time is spent waiting for or lost as tools, materials and equipment is not readily available for the task at hand. Discussions with maintenance workers will highlight simple tasks that can be undertaken to dramatically improve productivity.

- **Acceptance Criteria** – Many work order procedures already define how inspection should be performed as well as how to perform it safely the most commonly missing information however is an acceptable range of results that advises a maintenance worker when a task may not be required or when the job is functionally complete.

- **Mean-time-to-repair** – Maintenance work order descriptions should also include the standard hours the task typically requires to complete. This information is not only essential for effective task planning and scheduling but can assist

workers to maintain pace or to document situations where abnormal maintenance efforts may be required.

11.7 Wastewater Environmental Goals and Objectives

Goals and objectives provide the organization with a “north star” or point to focus organizational energies. The following are leading environmental goals and objectives of wastewater systems:

- No overflows or diversions
- No permit or regulatory violations
- Protection of the environment by identifying and meeting sewage system needs with due consideration of system impacts on air, land and water
- Maximize reuse and conservation of resources generated from waste water treatment including biosolids, digester gas and water.

11.8 Total Productive Organization

Total Productive Organization refers to the concept of integrating operation and maintenance functions. Historically, large public wastewater utilities have not integrated the operation and maintenance functions. When the functions are segregated, operators of process equipment are often unfamiliar with the maintenance of that equipment. Further maintenance personnel, who are responsible for repairing process equipment, are usually unfamiliar with its operation. With the utilization of the Total Productive Organization concept, wastewater utilities pursue an integrated approach that results in reduced staffing and increased efficiency.

Critical factors that can limit management from achieving the desired level of operations and maintenance integration include collective agreement limitations and reluctant staff attitudes. Overcoming these limitations requires direct discussions with unions and workers on the productivity gains that may be achieved and their overall importance to the utilities operation. Union and staff may be enticed to adopt this concept by offering them economic incentives such as a share of productivity gains realized from employing the concept.

A vital step in establishing the Total Productive Organization concept is the development of formalized operations and maintenance cross training programs. Cross-training is an

effective technique that can be used by wastewater utilities to enhance productivity and optimize plant performance. As wastewater plants downsize, it is even more important to have a versatile staff that has a thorough understanding of all plant processes and the ability to perform many tasks. Cross training also promotes a better work environment as staff learns to appreciate the importance of each other's jobs. Cross training is also a useful technique in fostering a team approach and eliminating "it's not my job" mentality.

11.9 Program Driven Maintenance Programs

Emergency repairs are costly to wastewater utilities as they are disruptive, impacting customer services as critical infrastructure is taken out of service at inopportune times, and costly as resources are drawn away from other tasks to enact corrective or emergent repairs, resulting in greater maintenance set up costs and repairs being often undertaken at a premium cost (overtime).

Program driven maintenance programs use predictive and preventative maintenance management approaches to reduce / eliminate these more costly emergent and corrective maintenance activities. Preventive maintenance programs establishes scheduled inspections and maintenance over the life cycle of utility infrastructure while predictive maintenance establishes sensors, such as oil and vibration testing on system pumps and large motors, and other monitoring mechanisms that test and interpret changes in processes and equipment performance.

Review of maintenance programs of various wastewater treatment plants, by Black and Veatch Corporation has resulted in maintenance efficiency benchmarks for planned versus reactive maintenance efforts. Black and Veatch found that high performance wastewater plants were found to have less than 25% of total generated work orders to be corrective in nature (emergent or unplanned repairs) and in excess of 75% of total wastewater plant maintenance work orders resulting from planned or driven maintenance programs.

With regard to wastewater collection systems the Water Environment Research Federation, American Society of Civil Engineers, and the Environmental Protection Agency provide measures on the effectiveness of wastewater collective system maintenance. The following is a summary of some of the benchmarks:

Description	Benchmark
Percentage of sewers rehabilitated per year.	0.5%
Percentage of sewers inspected per year.	6.5%
Percentage of sewers cleaned per year.	20.5%
Crew size for repairs	3.2
Percentage of work orders completed in 1 day.	62%
Percentage of work orders completed in 2 days.	84%
Flow evaluation (percentage of system)	53%
Manhole inspection (percentage of system)	37%
Flow monitoring (percentage of system)	20.5%

11.10 Treatment Plant Performance Dashboard

A system to measure and manage the performance of a wastewater system is integral to the systems long-term success. Most wastewater utilities have been largely driven by government regulations and plant reliability considerations. With the watchwords of “no violations” and “no plant failures” little attention was given to cost efficiency or service effectiveness. Today’s forces still include increasing government regulations, but O&M costs are usually the major consideration due to competition from private contract operators. “Getting competitive” means reducing these O&M costs while maintaining or even improving effluent quality and service reliability. Tools are needed to analyze performance over time to achieve greater O&M efficiencies, and to predict when action or intervention is necessary to maintain service reliability or quality. A performance measurement system must be carefully designed and built to present the right information to aid in achieving O&M efficiencies.

Well-designed and properly implemented performance measurement systems can enable utilities to achieve these new levels of performance. These information systems are typically enterprise-wide, meaning they must integrate measures from several “front-end” information and control systems. Performance measures are many times consolidated, aggregate measures, which combine costs and production information to enable near real-time adjustments in utility business processes.

The “balanced scorecard” approach to performance measurement is an approach currently used in many organizations to monitor efficiency and effectiveness. The approach may be applied with success in a wastewater treatment plant setting. This approach allows for measures at various levels of the plant in order to provide different views of information from various performance centers.

The balanced scorecard is based on measures of efficiency, quality, and effectiveness at each level of the performance framework. The types of measures for each level include:

Efficiency - measure of ratio of production (output) to the resources required (input), typically cost or time of input resources per unit of output (e.g. \$/Ml treated).

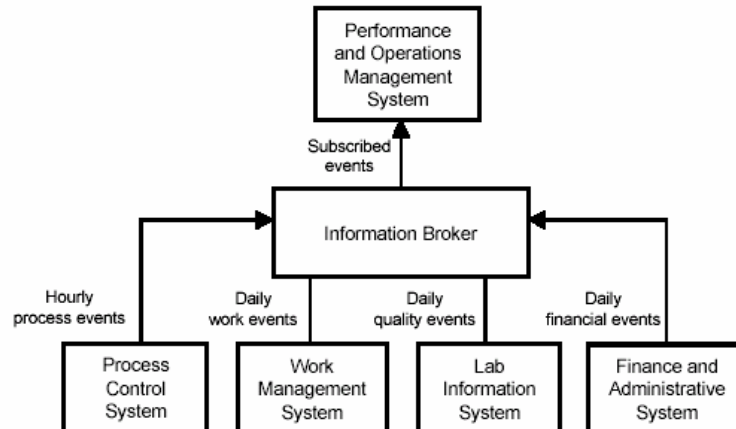
Quality – measure of the characteristics of the output that adds value to a customer or stakeholder, often measured against a target value or compliance standard (e.g. effluent suspended solids, mg/l).

Effectiveness – measure of a process or service to meet demand or achieve outcomes, typically expressed as capacity effectiveness (relationship between demand and input, i.e. capacity available) or production effectiveness (relationship between demand and output, i.e. production results available.) Effectiveness is usually measured at a level above the process or service where related efficiency or quality is measured.

Each type of measure must be defined for each service or process at a given level, in order to have a “balanced scorecard”. The challenge is to be selective in defining these measures to establish the smallest number of most relevant measures. An example of a scorecard is provided below:

Service or Process	Efficiency Measure	Quality Measure	Effectiveness Measure
Plant Facility	Cost to Treat (\$/Ml)	Effluent BOD, Total Suspended Solids etc. (mg/l)	Treatment Capacity Available
Solids Handling	Cost to Treat / Dispose (\$ per dry tonne)	Aggregate Concentration (% solids)	Process Unit Availability (% available)
Sludge Dewatering	Polymer Dosage (kg / dry tonne)	Aggregate Concentration (% solids)	Dosage Control Accuracy (“+ or –” target dosage)

Performance Measurement Systems gather information from key performance centers on a hourly, daily, weekly or monthly basis. An example of a collection network is outlined below.



11.11 Selecting Wastewater System Rehabilitation Technologies

Sewer rehabilitation or replacement needs are often not known to a municipality as this infrastructure is underground. It takes a regular program of inspection, usually through closed-circuit television (CCTV), to maintain awareness of the physical condition of the sewers. Operational issues (e.g., sewer backup, basement flooding, overflows, and odour complaints) are often indicators of rehabilitation needs in the system. A municipality can choose a balance between reactive rehabilitation (responding to pipe collapses that cause backups and/or road surface failures) and proactive rehabilitation (investing in lower cost rehabilitation when internal inspection shows early signs of physical distress but the pipe has not yet failed). Choosing to preserve the physical condition of sewers ahead of time is cost-effective since reactive repairs are several times the cost of proactive rehabilitation. It is therefore, an annual inspection and flushing program should be established.

Critical items that should be considered before selecting an appropriate rehabilitation or replacement technology include impact on users, (user disruption, user inconvenience etc.), local availability of the types of technologies, surface conditions, the density of sewer laterals, and the depth of the sewers and laterals being considered for remedial

action. The options available to a municipality focus on two possible alternatives: replacement / structural rehabilitation, or non-structural and semistructural rehabilitation. Within both of these rehabilitation alternatives trenchless and open cut methods exist. The following is an overview of methods and their relative advantages and disadvantages.

11.11.1 Open Cut Construction

The installation of new or replacement sewers by trenching is frequently referred to as the open cut method. The installation of new pipe should be undertaken when the review of all potential technologies has been completed and the open cut method is ranked as the best alternative.

Advantages:

- A new sewer is installed, complete with all new appurtenances. This provides a longer expected life than obtainable through most trenchless methods.
- The alignment of the sewer can be set to meet the needs of the local area.
- Sewer connections can be replaced to meet current standards.
- The sewer sizing and/or grade can be changed to meet current and future hydraulic requirements.
- Other infrastructure can be rehabilitated or replaced at the same time, allowing for coordination of work and sharing of costs.
- Combined sewers can be separated.
- Storm sewer laterals currently connected to the sanitary system can be disconnected.

Disadvantages:

- The cost of the open cut method can be substantial compared to some newer technologies.
- Construction is usually longer than with most trenchless technologies due to the quantity of disturbance to other infrastructure and traffic, and the amount of reinstatement work required following the installation of the sewer.
- There are more safety concerns due to traffic issues on road rights-of-way, the number of excavations required, and the large equipment needed to perform the work.

- There can be disturbances to other surface and buried infrastructure.
- The social and economic costs of major open cut projects can be substantial during construction.

11.11.2 Sliplining

Sliplining is the insertion of flexible liners directly into the sewer. Either continuous or jointed discrete lengths of pipe are pulled or pushed through the existing pipe. Sliplining creates a new pipe inside the old sewer without a complete excavation. The sliplined pipe is then reconnected to the existing sewer at both ends.

PVC and HDPE (high density polyethylene) pipe is primarily used in sewer sliplining applications. With PVC pipes, joints are traditional push-on joints. HDPE pipe are either butt fused (thermal process), or joined together by electrofusion in various possible lengths above ground, then inserted into the host sewer at entry points.

Once the new sliplined pipe has been inserted into the host pipe, grouting is generally required to fill the void between the new and old pipes. Grouting is an important step in the sliplining process to maintain the structural stability of the new pipe. A sliplined pipe substantially reduces the cross-sectional area of the pipe. However, the reduction in friction with the lined pipe compared to the previous, old unlined pipe can partially compensate for the reduced internal diameter. Hydraulic requirements must be considered carefully before selecting sliplining as a preferred alternative.

Advantages:

- Sliplining can be applied to most types of pipe.
- It is rapid and causes little disturbance to other utilities.
- It is most successful with few connections.
- It usually provides an improved friction coefficient for improved hydraulic performance.
- Depending on flows, installations can be done in live lines without bypass pumping.

Disadvantages:

- The sliplined pipe is usually sized so its outside diameter is at least 10 percent smaller than the inside diameter to allow for smooth insertion. This reduction, in association with the wall thickness of the pipe, leads to the loss of cross-sectional capacity.
- Sliplining requires a long assembly/lay down area.
- When short pipe sections are used, there is an increased cost in the jointing techniques.
- Poorly controlled grouting to the annular space can lead to buckling of the liner pipe.
- Many excavations may be required if many service and branch reconnections are involved.
- Because the liners used for sliplining do not turn through elbows, the alignment of the unlined pipe must be considered before selecting this technique.

11.11.3 Diameter Reduction Sliplining

Close fit sliplining involves inserting a thermoplastic tube that has been temporarily deformed to allow sufficient clearance for insertion into the host pipe. The tube is subsequently returned to its original shape and diameter, providing a close fit in the host pipe. The outside diameter of the tube is the same or slightly larger than the inside diameter of the host pipe. The tube is passed through a set of dies (referred to as “swageing”) or through an array of compression rollers, to reduce the tube diameter to allow for insertion by winching. The tube then reverts to its original dimensions once the winch tension is released, and in most cases, with the help of internal pressure.

Advantages:

- Close fit diameter reduction sliplining can be applied to most types of pipe.
- It is rapid and causes little disturbance to other utilities.
- It is most successful when there are long runs with few connections.
- It usually provides an improved friction coefficient for improved hydraulic performance.
- There is minimal loss of pipe diameter and no grouting requirement when compared to the traditional sliplining technique.

- The liner can be selected to provide either full structural integrity or semi-structural integrity, depending on the condition of the host pipe.

Disadvantages:

- The energy required to reduce the pipe diameter increases dramatically with larger pipe sizes and greater wall thicknesses.
- The installation of the tube may get hung up during the installation of pipes that are deformed, have dimensional irregularities or displaced joints.
- Manufactured pipe for insertion usually requires special extrusion dies due to non-standard pipe diameters.
- The host pipe needs surveying, cleaning, and preparation.
- Sufficient site space is required to accommodate butt-fusion welding of pipes before the diameter reduction and during insertion.
- As with standard sliplining, the alignment of the host pipe must be considered before selecting the diameter reduction technique, as the winched pipe does not turn well through elbows.

11.11.4 Fold and Form Sliplining

This technique is based on the liner being heated and folded at the manufacturer's factory, and then transported to the installation site. The folded liner is then inched into the host pipe and re-rounded using a combination of heat and pressure and, at times, a device propelled through the liner. PE liners are preferred for pressure applications while PVC systems are mainly used for gravity sewers.

Advantages:

- Fold and form sliplining can be applied to most types of pipe.
- It is rapid and causes little disturbance to other utilities.
- It is most successful with few connections.
- It usually provides an improved friction coefficient for improved hydraulic performance.
- There is minimal loss of pipe diameter and no grouting requirement when compared to traditional sliplining techniques.
- The liner can be selected to provide either full structural integrity or semi-structural stability depending on the condition of the host pipe.

- The cutting and reinstatement of service connections can be done remotely with robotic equipment reducing surface excavations.
- Some liners can be used in host pipes with bends up to 45°, with some internal wrinkling.
- The site-folded technique is less sensitive to the variations in diameter or pipe with dimensional irregularities, when compared to the diameter reduction technique.

Disadvantages:

- The folding and re-rounding process of the liner may affect the long-term pressure capability of the liner.
- Sometimes, the reversion process may not be completed fully.
- The liner may move in relation to the host pipe due to stresses that may be developed in the liner (e.g., due to thermal expansion or contraction).
- The liner cannot be used in bends of more than 45°.
- The host pipe needs surveying, cleaning, and preparation.
- For full structural applications, the folding and re-rounding process of the installed liner must be carefully monitored to avoid long-term liner problems.
- Pre-grouting may be necessary in damaged areas or where there are voids.

11.11.5 Cured in Place Pipe (CIPP)

CIPP is frequently referred to as in situ relining. A fabric tube is impregnated with a thermosetting or ambient-cured polyester or epoxy resin before being inserted into the host pipe. The resin is then cured to produce a rigid pipe within the host pipe. The combination of the fabric material, with or without fibres, and the resin can be designed to produce a new pipe that has full structural capabilities or semi-structural capabilities. The fabric material to be used can be tailored in the factory to suit the diameter of the host pipe. Non-circular sections can also be lined if required. CIPP liners can also negotiate 90° bends within the host pipe.

There are three main groups of CIPP systems. These are available independently or in combination: felt-based, woven hose, and membrane systems. All three are usually installed by inversion, in which the liner is fed through the host pipe and turned inside out by water or air pressure. Some CIPP liners can also be installed by winching the liner through the host pipe and then inflating it.

Advantages:

- Installation is relatively fast with minimal excavation required.
- Access to the sewer is normally gained from an existing access hole.
- It offers a choice of different resins to suit the application.
- The system can accommodate very long lengths as well as a variety of diameters and can negotiate bends.
- Service connections can be reinstated by robotic cutters, reducing excavation requirements.
- It fits in very tightly to the host pipe, and resists thermal expansions or contractions.
- An improved interior friction coefficient usually increases hydraulic capabilities.
- It can be used in structural, semi-structural, and non-structural applications.
- CIPP is widely available.

Disadvantages:

- The host pipe needs extensive surveying, cleaning, and preparation.
- Sizes smaller than 100mm or larger than 600mm have greater difficulty of installation.
- Partial buckling and/or ovality may occur during installation.
- For full structural applications, liner preparation and installation processes must be carefully monitored to avoid long-term liner problems.
- Pre-grouting may be necessary in damaged areas or where there are voids.

11.11.6 Pipe-bursting

Pipe bursting is a trenchless technology that replaces a sewer by breaking and displacing the existing pipe and installing a replacement pipe in the void created. The system uses a pneumatic, hydraulic, or static bursting unit to split and break up the existing pipe, compressing the materials into the surrounding soil as it progresses. The new replacement pipe is simultaneously pulled or pushed with the bursting head to fill the void created. It is possible to upsize to about 30 percent greater than the diameter of the existing pipe, but this depends on soil conditions, the proximity of other existing structures, and the depth of cover. The replacement pipe must be installed in one continuous length and, as such, butt-fused PE pipe is used in most cases. Service connections and other appurtenances connected to the sewer to be rehabilitated must

be excavated and exposed before starting the pipe bursting. As well, all pipes and underground structures within one metre of the sewer to be rehabilitated by bursting must be excavated and exposed to avoid damage due to the forces transmitted through the soil during the pipe-bursting process.

Advantages:

- Cleaning of the existing pipe is not necessary.
- A larger diameter pipe can be inserted. This, in conjunction with the improved interior friction coefficient, can substantially increase the hydraulic capabilities of the new sewer.
- It provides for full structural rehabilitation.
- It is most successful when there are long runs with few connections.
- Continuous pipe (HDPE) or discrete, joined pipe, such as PVC or DI can be used.

Disadvantages:

- Pit excavations are normally required to accommodate the replacement of pipe sections.
- All sewer appurtenances must be excavated before bursting, and reconnected to the new sewer afterward.
- All underground structures within one metre of the existing sewer to be rehabilitated must be excavated to avoid damage that may occur due to the force being transmitted, and the displacement of soil, by the bursting technique.
- Any rigid obstructions in the host pipe bedding will deflect the new pipe.
- This method is not recommended where grade is critical.
- Ground surface heaving can occur if the depth of cover is too little.

11.11.7 Horizontal Drilling

Horizontal drilling, frequently referred to as HDD (horizontal directional drilling), consists of several stages for installation. First, a pilot bore is made with a suitably sized drilling rig. The bore is steered to create an initial hole at the required line and grade. Successive reamers are then pulled back to enlarge the hole diameter to the desired size. During the last stage of the reaming, the service pipe is pulled back into the bore. This method is primarily employed when an open cut excavation is completely

unsuitable (e.g., at a railway crossing) and a new sewer alignment is desired. Most sewer force mains installed by this method are continuously welded PE pipe, although steel, ductile iron and PVC have also been used. Gravity sewer installations are also possible.

Advantages:

- There is reduced disruption to surface operations, such as major thoroughfares, railway tracks, rivers, buildings, and trees.
- There is less disruption to buried infrastructure compared to the open cut method.
- It allows for a new sewer alignment.
- This method usually has lower restoration costs compared to the open cut method.

Disadvantages:

- Exact pipe alignment can be difficult to attain, although still fairly accurate.
- Cobble and gravel seams might cause difficulties during the pilot bore and pullback stages.
- On large installations, large quantities of drilling mud are used creating the potential risk of frac-out and costly slurry management actions (i.e., recycling, containment, and disposal).
- HDD requires consistent and good soil conditions (e.g., firm clay, boulderless cohesive tills) for good performance.

11.11.8 Internal Joint Seals

Internal joint seal make the inside surfaces of leaking concrete pipe joints watertight. This technique is primarily used in pressure applications, such as force mains or water mains. The seal's flexibility ensures a bottle-tight seal around the entire pipe joint, while its low profile and graded edge allows water to flow without creating turbulence. Internal joint seals are made of EPDM a synthetic rubber. This technique requires people to access the sewer to perform the work and, as such, pipe diameters of sufficiently large size are good candidates for this technology. Internal preparation of the pipe is really critical for internal joint seals to perform to specifications. Complete pipe preparation is appropriate for the working environment of the workers. The pipe joints must be

completely cleared of debris and dust. Complete preparation of the area on either side of the joint is also required to accommodate the lip of the seal. Once the cleaning is completed, a cement grout is used to fill the joint gap completely and made flush with the internal surface of the sewer. Next, the area must be cleaned with a dry brush and coated with a lubricant soap compatible with the type of seal being used. The lubricant soap is only an aid for installing the seal. The seal is then placed in position, spanning the gap. Stainless steel retaining bands are then installed in the grooves of each seal. A hydraulic expanding device is used to apply the correct pressure to the retaining bands, thereby keeping the seal in place.

Advantages:

- This technology is specific to pipe joint issues only.
- Minimal working space is required at the surface.
- It is a low-cost alternative.

Disadvantages:

- It can only be used in pipe sizes suitable for human access.
- It does not address other possible pipeline deficiencies.
- Bypass pumping is required.

11.11.9 Panel or Section Insert Liners

Panel or section insert liners are used only where person entry to the sewer is available. Various materials can be used, but GRP (glass reinforced plastic), GRC (glass reinforced concrete) and Ferro-cement are primarily designed for this type of application. When panels are used, they are designed to form a close fit, with fixed spacers, then grouted in place. The panels are relatively light and are designed to pass through access holes. Larger diameter sewers can be lined with sections rather than panels. These would be carried into the pipe and joined in situ. The sections should also be grouted in place.

Advantages:

- These technologies can be applied for structural or non-structural purposes.
- The liner can be designed to match the original host pipe diameter, thereby minimizing the loss in capacity.

- The liner can be effectively laid to a required grade as individual pipes can be fixed within the host pipe by spacers.
- There is reduced infiltration.
- There is minimal disruption at the surface as access can take place from existing access holes (manholes).

Disadvantages:

- Bypass pumping is required.
- It is a labour-intensive technology.
- There is a loss of cross-sectional diameter in the existing pipe due to the installation of the panels or sections and the grouting space required.

11.11.10 Chemical Grouting

Chemical grouting is a technology primarily used for spot repairs to seal joints and non-structural cracks. Chemical grouting reduces or stops water infiltration and exfiltration. The chemical grout builds up an external, flexible, and impermeable mass in the soil surrounding the spot repair location. Chemical grouting is used primarily for cracks in pipes, at leaky joints, and in access holes. One of the main benefits of grouting includes the fact that sections of sewer can remain in service during the rehabilitation process.

Advantages:

- It is a cost-effective method to stop water infiltration by filling voids and sealing fissures in fractured soil.
- It can prevent future structural damage.
- Chemical grouting is effective when used with other technologies.

Disadvantages:

- Its application is restricted due to potential harmful effects.
- There is the potential of ground water pollution (selection of grout type is a major consideration).
- It does not provide structural repair.

11.11.11 Full Tunnelling and Micro-Tunnelling

Full tunnelling and micro-tunnelling are techniques normally used for very deep installations. Although primarily used for new installations, applications have been included for rerouting existing sewers. Experts in this field should be engaged for application of this technology. Full tunnelling is a construction method of excavating an opening beneath the ground without continuous disturbance of the ground surface and of sufficient diameter to allow individuals to access and erect a ground support system at the location of the material excavation. Micro-tunnelling is different than full tunnelling in that the process uses a remotely controlled boring machine combined with the pipe jacking technique to install pipelines directly.

Advantages:

- There is a high level of accuracy due to the laser-guided installation.
- Non-human entry reduces safety considerations (micro-tunnelling).
- There is continuous tunnel support. Micro-tunnelling is suitable in unstable ground conditions.
- The method is applicable in deep sewer installations.

Disadvantages:

- Tunnelling needs a minimum depth of cover.
- A tail tunnel is required for effective spoil removal (full tunnelling).
- It is expensive for short stretches.
- Extensive geotechnical information is required.
- The potential exists for ground settlement.
- High-level operator experience is needed.

11.11.12 Auger Boring

Auger boring is the process of simultaneously jacking casing through the earth between two pre-sunk shafts while removing the spoil inside the encasement with a rotating flight auger. The casing supports the surrounding soil as spoil is systematically removed. As a general rule, auger boring has poor steering capabilities. There are two types of auger boring: track type and cradle type. The track type method consists of a track system, machine, casing pipe, cutting head, and augers. The boring operation is cyclic, as pipe segments and auger flights are added after a prescribed auger flight length is installed.

No rotation is applied to the casing as it is jacked through the soil by hydraulic thrust rams located at the rear of the machine. Lubrication is used to reduce skin friction and to aid with soil cutting and transport. An additional common measure to reduce skin friction includes an over excavation in the order of 25mm to 50mm. Pipe diameters range from 200mm to 1200mm, and overall installation lengths are typically limited to 100 m.

Advantages:

- The technology is well established and widely available.
- There is minimum surface disruption. Boring is suitable for road and railway crossings.
- The steel casing remains in the bore after the drilling operation is complete; it can be used as a conduit or as the host pipe.

Disadvantages:

- Steering capability is limited after installation is initiated.
- It cannot be used in loose sand and very soft clay/organic soils.

11.11.13 Pipe Eating

Pipe Eating is a technique based on micro-tunnelling, in which a defective pipe is excavated together with the surrounding soil. The micro-tunnelling shield machine will usually need some crushing capability to perform effectively. The replacement pipes are connected to the back of the tunnelling shield. The defective pipe may initially be filled with grout to improve steering performance of the shield machine. The pipe fragments can be removed by either vacuum excavation or by slurry pumping.

Advantages:

- This method permits in-line replacement and upsizing of sewers with reduced potential for disturbing paved surfaces or adjacent utilities.
- No fragments from the old pipe are left in the ground.
- It enables sagging sewers to be realigned.
- Some systems allow the wastewater to be pumped through the shield during installation, thus eliminating the need for a bypass.
- Steering is possible within limits, and can follow the alignment of existing pipe.

Disadvantages:

- The method is not suitable for the replacement of metallic or thermoplastic pipes.
- It can be costly in comparison with pipe bursting.
- Working space is needed above ground for ancillary construction equipment

12.0 Conclusions and Recommendations

12.1 Conclusions

1. *Model Limitations*

- Rainfall data obtained did not include any large storms with greater than 2 year return period. This will affect accuracy of the model in less frequent storm events. The model may, in fact, over-estimate flows in less frequent storm events. For example, when running the 10 year storm event, flows may, in fact, be representative of a 25 year storm event.
- Flow monitoring data was of good quality in newer residential areas. No quality data was obtained from older areas of the city or industrial/commercial areas. Model accuracy is questionable in the older areas of the city where quality flow monitoring data was not available. However, in newer residential areas, the model is considered quite effective.

2. *Existing System Assessment*

- The 92 Street Trunk is at its maximum capacity for its current service area. It appears that the trunk sewer can still service a small future development. However, this would create surcharge connection in the trunk sewer and increase potential risk.
- The Ivy Lake Sub-Trunk is a potential source of basement flooding and measures to reduce flows in this area should be considered.
- The 108 Street Trunk is deemed adequate to service infill development of all areas presently serviced by it. If new developments in the northwest are allowed to connect into this trunk sewer, slight surcharge condition will occur in the trunk sewer. However, the surcharge condition is not significant enough to create a problem.
- As identified in the 1995 Master Plan, the area along the 200mm sewer east of 100 Street, north of 116 Avenue has a risk for basement flooding. Despite the inflow/infiltration reduction procedures undertaken by the City in the past, these areas are still subject to the risk of basement flooding and measures to mitigate flows in this sewer are necessary.

- The downtown area appears to have substantial surcharging throughout. Additionally, the 675mm trunk south of downtown also appears to be experiencing severe surcharging. However, due to a low confidence level in the model in this area, improvements were not proposed. Further study of this area will be warranted.
- The local sewer upstream of the Country Club Estates pump station experiences surcharging condition during wet weather. This situation will steadily worsen until such time at the 60 Avenue Trunk is completed.
- Spare capacity is available in 375mm sub-trunks along 100 Avenue on the west end, and along 68 Avenue in the southwest. The spare capacity will allow for interim servicing for new development areas until a new trunk is built along 116 Street.

12.2 Recommendations

1. Wastewater Collection System Design Criteria

- The Harmon Residential Peaking Factor currently utilized by Aquatera / the City of Grande Prairie should be changed to the City of Edmonton's residential peaking factor formula. This recommendation was based on an analysis of the flow monitoring data which indicated that the City of Edmonton Peaking Factor formula is a better representation of actual wastewater flows in the City of Grande Prairie. The peaking factor will not be less than 1.5 and greater than 4.0.
- The peaking factor of 2 applied for commercial/industrial areas, should be changed to coincide with the formula given by the provincial standards. This will ensure more accurate representation of flows from these areas.
- Inflow/Infiltration (I/I) allowance should be changed from 0.10L/s/ha to 0.28L/s/ha. Based on the analysis of flow monitoring data for a small event of less than 2 year return period, the I/I exceeded the present standard. This standard has been adopted by many municipalities in Alberta.
- Pipe design flow should be increased over peak flow by a factor of 1/0.864. This will conform to provincial standards and provide an additional factor of safety in the system design.

2. Wastewater Modelling Software

- Based on an analysis of commercially available software for modelling sanitary sewer systems, it is recommended that the City adopt XP-SWMM Version 8.52 for modelling software because of the price and ease of use.

3. Future Flow Monitoring/Rainfall Data Collection

- Rainfall monitoring should be carried out in the future at multiple locations in the City to ensure that rainfall data corresponding to flow monitoring data will be available, particularly for a storm event of lower frequency. The additional rainfall monitoring data will improve model accuracy.
- Flow monitoring should be conducted in the future, as specified in Figure 6.1., to ensure flow monitoring data will be available for older residential areas and commercial areas. These areas will be more accurately represented in future model revisions when the new flow monitoring data is available.

4. Existing System Upgrading for Infill Development

- The Crystal Lake Estates area should ultimately be diverted to the 88 Street Trunk. This will reduce the risk in the existing 92 Street Trunk as well as in the areas around Ivy Lake.
- Land within 400m of the existing 92 Street Trunk should be allowed to connect to this trunk. This can be made feasible by the diversion of the Crystal Lake Estates area.
- To reduce risk from surcharging in the 200mm sewer in the lane east of 100 Street north of 116 Avenue, a diversion should be constructed from the 200mm sewer west across 100 Street to the 300mm sewer located there. This diversion should be constructed near 124 Avenue.
- Accounting for infill development rates in the southeast as well as the existing capacity of the pump station located near Country Club Estates, the remaining section of the 60 Avenue Trunk sewer should be constructed no later than 2008. This will ensure that the risk to the local sewer near 60 Avenue could be minimized.

5. Ultimate Servicing of New Development Areas

- The new 88 Street Trunk Sewer should be constructed as shown in Figure 8.1. This trunk sewer should be sized to accommodate the Crystal Lake Estates area so that the risk in the Ivy Lake area can be reduced. Additionally, the trunk sewer should be sized to accommodate the 5 ¼ sections just east of 88 Street for potential future development. This total construction cost is approximately \$6.2 million.
- Service for the area on the northwest and west of Grande Prairie can be accomplished by constructing a new trunk sewer along 116 Street similar to the alternative alignment proposed in the 1995 Master Plan. This sewer will be designed to service all new developments west of 108 Street. Areas local to the Northwest Trunk, including new areas north of the Bear Creek, will be allowed to flow down the existing 108 Street Trunk. It is determined that there will be little risk along the 108 Street Trunk Sewer.
- Based on Net Present Value analysis, Alignment 1b is recommended for the new 116 Street Trunk Sewer as shown in Figure 8.2, although Alignment 3 has lower construction cost than Alignment 1b. However, Alignment 1b could provide interim servicing, and the costs are distributed far enough into the future to overcome the comparative cost advantage to Alignment 3. Therefore, Alignment 1b is recommended for implementation. In addition, Alignment 1b can also accommodate the development of southwest Grande Prairie. The estimated construction cost of this Alignment 1b is \$15.7 million for ultimate servicing.

6. Wastewater Trunk Sewer

- Aquatera should work with developers to fund construction of the south end of the 88 Street Trunk from 66 Avenue to 100 Avenue as it is extended northward as development occurs. Developers that are upstream of this future trunk and wish to proceed in advance of this construction are to develop where interim measures are available. In these cases, the developer will pay costs for interim servicing and the portion of the trunk sewer adjacent to or within the development. Aquatera will pay oversizing costs to developers who construct trunk sewers and recover them through future levies or charges.
- The 116 Street Trunk Sewer south of 100 Avenue to 68 Avenue shall be implemented in 2007 or earlier.

- The 116 Street Trunk Sewer north of 100 Avenue shall be constructed in 2012.
- The 116 Street Trunk Sewer south of 68 Avenue shall be constructed in 2013.

Corporate Authorization

This document entitled “Wastewater Collection System Master Plan” was prepared by Infrastructure Systems Ltd.



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<p>PERMIT TO PRACTICE <i>Infrastructure Systems Ltd.</i></p> <p>Signature _____ Date _____</p> <p>PERMIT NUMBER: P 4741</p> <p>The Association of Professional Engineers, Geologists and Geophysicists of Alberta</p>

Appendix A

Master Plan Growth Projects and I / I Contributions

Appendix B

Monitored Site Flow Plots

Appendix C

Wastewater Models CD

Appendix D

Digital Report Files