TOWNSHIP OF WELLINGTON NORTH

CLASS ENVIRONMENTAL ASSESSMENT MASTER PLAN STUDY FOR WATER SUPPLY AND SANITARY SEWAGE SYSTEMS

COMMUNITY OF ARTHUR

JANUARY 2012
(Revision 2)

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FOR
WATER SUPPLY AND SANITARY SEWAGE SYSTEMS

COMMUNITY OF ARTHUR

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CLASS ENVIRONMENTAL ASSESSMENT MASTER PLAN STUDY
FOR
WATER SUPPLY AND SANITARY SEWAGE SYSTEMS

COMMUNITY OF ARTHUR

1.0 INTRODUCTION

1.1 Study Scope:

The community of Arthur is a growing urban community located within the Township of Wellington North (Township) and is currently serviced by municipal water and sanitary services. In order to maintain this level of service in the future, the Township identified the need to complete a technical review of the water and sanitary infrastructure in the community of Arthur. This review has included the following components:

- Compile background information for existing infrastructure systems.
- Establish development scenarios for analysis (i.e. existing and future).
- Evaluate the performance of the systems based on existing and future development scenarios.
- Identify system infrastructure upgrades/extension alternatives required to service future development.
- Formulate a long-term servicing strategy which will identify specific infrastructure required to service future development within Arthur.

In order to complete the evaluation component of the review, it was necessary to create computer models for both the water distribution and sanitary collection systems. The details of these models and the results of the analysis completed are discussed in this report.

1.2 Class EA Process:

This review has been completed in accordance with the Class Environmental Assessment (Class EA) Master Plan (MP) process as outlined in the Municipal Class Environmental Assessment document prepared by the Municipal Engineers Association (MEA) amended in 2007. This Master Plan is intended to address Phase 1 and 2 of the Class EA planning process which are defined as follows:

Phase 1: Identification and description of the problem (deficiency) or opportunity.

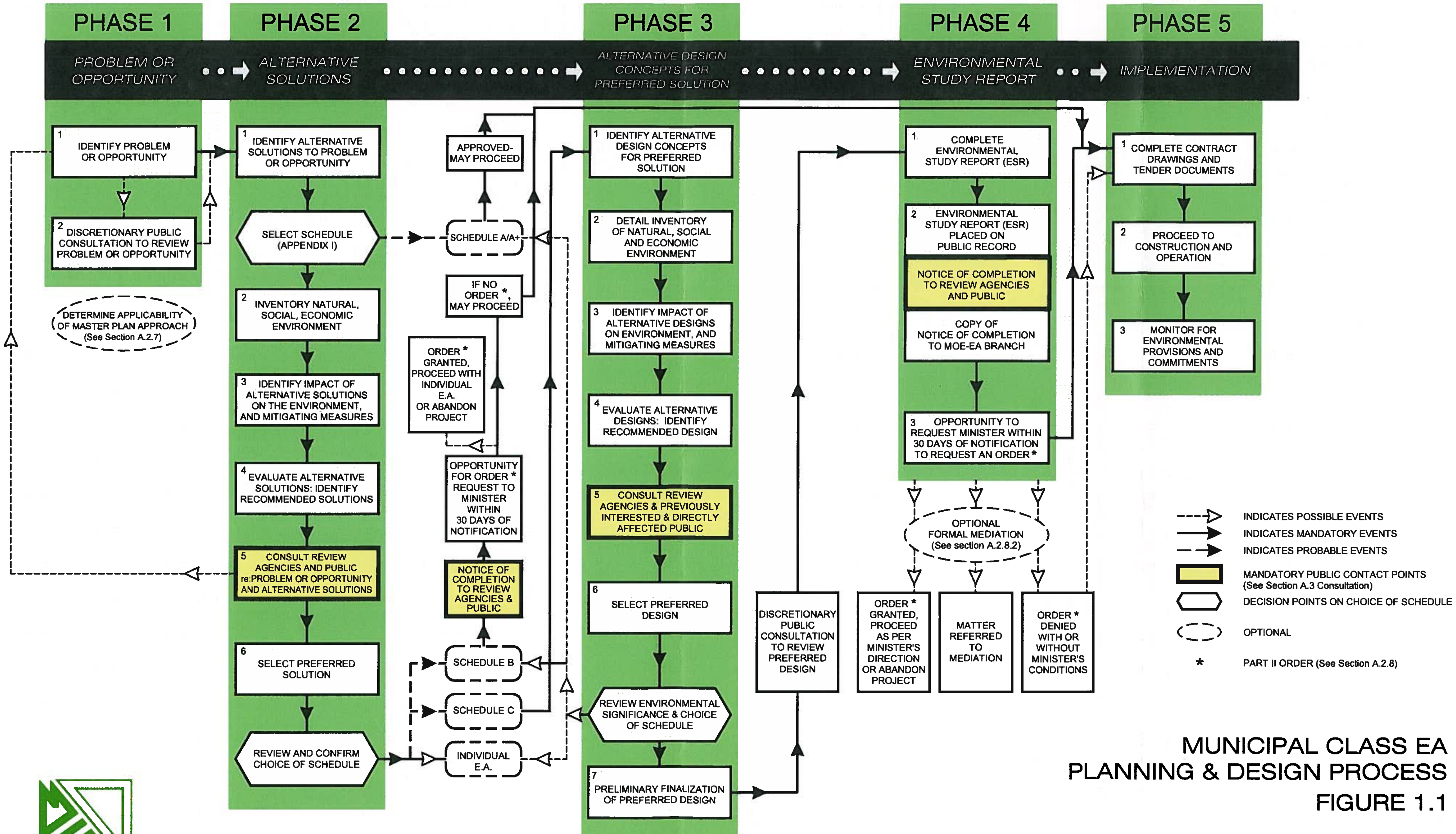
Phase 2: Identify and evaluate alternative solutions to address the problem or opportunity. Establish the preferred solution. This Phase is to take into consideration the existing environment including natural, social and economic aspects, public input and review agency comments.

The Master Plan will be used in support of further work carried out for specific Schedule B and C projects identified in the study. Depending on the scope and analysis completed for a specific Schedule B project within the Master Plan, the Class EA requirements may be satisfied. However, if the assessment details of the project change, the Schedule B project would be required to fulfil the consultation and documentation requirements. For Schedule C projects, it will be necessary to fulfil the additional requirements of Phases 3 and 4 of the Class EA process.

The Municipal Class Environmental Assessment planning process is illustrated in Figure 1.1.

MUNICIPAL CLASS EA PLANNING AND DESIGN PROCESS

NOTE: *This flow chart is to be read in conjunction with Part A of the Municipal Class EA*



MUNICIPAL CLASS EA
PLANNING & DESIGN PROCESS
FIGURE 1.1

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1.3 Problem Statement:

In accordance with the Class EA process and in consultation with the Township, the following problem statement and definition has been identified:

Develop a water supply and sanitary sewage servicing strategy for the existing and future development areas of Arthur that will allow the integration and coordination of new services with other on-going municipal infrastructure needs.

To achieve the requirements of the Class EA and this problem statement, the following tasks were included in this Master Plan:

- Identify/inventory the existing infrastructure, environment constraints and opportunities, interests of stakeholder/interest groups.
- Estimate future development servicing requirements within the community.
- Evaluate the performance of the systems based on existing and future development scenarios.
- Identify infrastructure deficiencies under existing and future development scenarios.
- Provide recommendations and an implementation strategy for infrastructure upgrades and extensions required to address these servicing needs.
- Document the study process in compliance with the Class EA process.

1.4 Public Consultation:

Public consultation was an integral part of the Master Plan process which included the following components:

- General Public input was solicited through an initial notification in local newspapers including the Arthur Enterprise News (February 18th and 25th, 2011) and the Wellington Advertiser (February 18th and 25th, 2011). Further, an Open House was held on March 2nd, 2011 to present preliminary findings and background information. A Notice of Completion will be placed in the local newspapers following Township approval of this Master Plan document.

- Notifications were sent to 28 contacts from various review agencies and interest groups requesting input specific to their organization's area of interest. Initial notification was sent out on February 1, 2011 which included an invitation to the Open House. Notice of Completion notification letters will be sent following Township approval of this Master Plan.

Copies of the Public Consultation information including notices, displays and comments received are included in Appendix A.

1.5 Description of Study Area:

The Township of Wellington North is an amalgamated municipality which incorporated the former municipalities of the Village of Arthur, Town of Mount Forest and Townships of West Luther and Arthur. The community of Arthur is located near the centre of Wellington County at the intersection of County Road 109 and Highway 6.

Arthur is a fully (i.e., water and sanitary) serviced urban community with a population of 2,563 representing a total area of 456 hectares. The existing development within the community is as follows:

- *Residential*: primarily single family detached with a small number of semi and multiple family units.

- *Commercial*: majority is located in the downtown core along George Street with additional highway commercial along County Road 109.

- *Industrial*: the majority is located at the western limit of the urban boundary.

- *Institutional*: includes two primary schools, nursing home, recreational facilities and several churches.

Topography of Arthur ranges between an elevation of 440m and 470m, with the overall slope generally from north to south toward the Conestogo River which is situated along the south limit of the developed area. The Conestogo River is situated above a sand base that lines the waterway through town and is more heavily concentrated in the southwest portion of the developed area. The majority of the Arthur area is comprised of a till material that is prominent in the region surrounding Arthur.

As indicated, Arthur is fully serviced by water and sanitary systems. The water system is a single pressure zone watermain distribution network that is pressurized by two elevated water towers. Water is supplied to the system from three bedrock wells. The sanitary system includes a sewer collection system, two sewage pumping stations, a lagoon storage facility and a treatment plant. This system discharges treated effluent to the Conestogo River.

The Arthur urban centre is primarily surrounded by agricultural area although there is some environmental/floodplain areas adjacent to the Conestogo River.

2.0 FUTURE DEVELOPMENT PROJECTIONS

2.1 General:

The objective of the Master Plan is to provide recommendations and an implementation strategy for infrastructure upgrades and extensions required to address these future servicing needs. In order to achieve this objective, it is necessary to accurately estimate future servicing requirements which is dependant upon predicting development within the community. In order to prioritize infrastructure upgrades/extensions, it is necessary to consider development at different planning periods. This review has considered development at 2031 and a long-term period beyond the official planning period.

The 2031 development scenario has been established using planning information provided by the County of Wellington. For development information beyond the time horizon considered in the Comprehensive Review, County and Township staff were consulted specifically to establish a reasonable long-term development scenario.

2.2 2031 Development Scenario:

The 2031 Development Scenario (2031 Scenario) has been established using planning information provided by Wellington County Planning and Development Department from the report entitled "Comprehensive Review of Residential and Employment Growth, Township of Wellington North" dated October 14, 2009 (County Report). This scenario included the following growth components;

- 494 new residential units is referenced in the County Report inventory of residential supply. However, the projected demand from 2006 is only 390 units, therefore a 104 unit oversupply exists. Based on discussions with municipal staff, and in order to better reflect the 2031 demand, WN-52 (91 units) was moved to the Long-term Development Scenario. Given this, the 2031 scenario includes total of 403 residential units. It should be noted that a number of these units have been constructed to date and the servicing analysis has been adjusted accordingly to reflect this.

- 62.1 acres (25.1 hectares) of industrial land.

- 8.6 acres (3.5 hectares) of highway commercial land

- 45 equivalent residential units have been added to reflect the remaining unused portion of the Golden Valley allocation.

The locations and limits of residential and non-residential developments are provided in Figure 2.1 and Figure 2.2 respectively both of which are contained in the County Report. A detailed breakdown of the specific development areas is included in Appendix B. These figures and breakdowns were taken from the County Report.

Residential development identified is comprised of 14 separate parcels distributed throughout the existing Urban Centre Boundary. These parcels vary in size from individual infill lots to a 129 lot subdivision, development types include low, medium and high density configurations.

The County Report does not provide a breakdown for non-residential development demand for each urban area, only a total forecast for the Township. This forecast translates into a demand for 71 acres

Arthur
Residential Supply Inventory
Figure 2.1
Map 7

Residential Supply Category

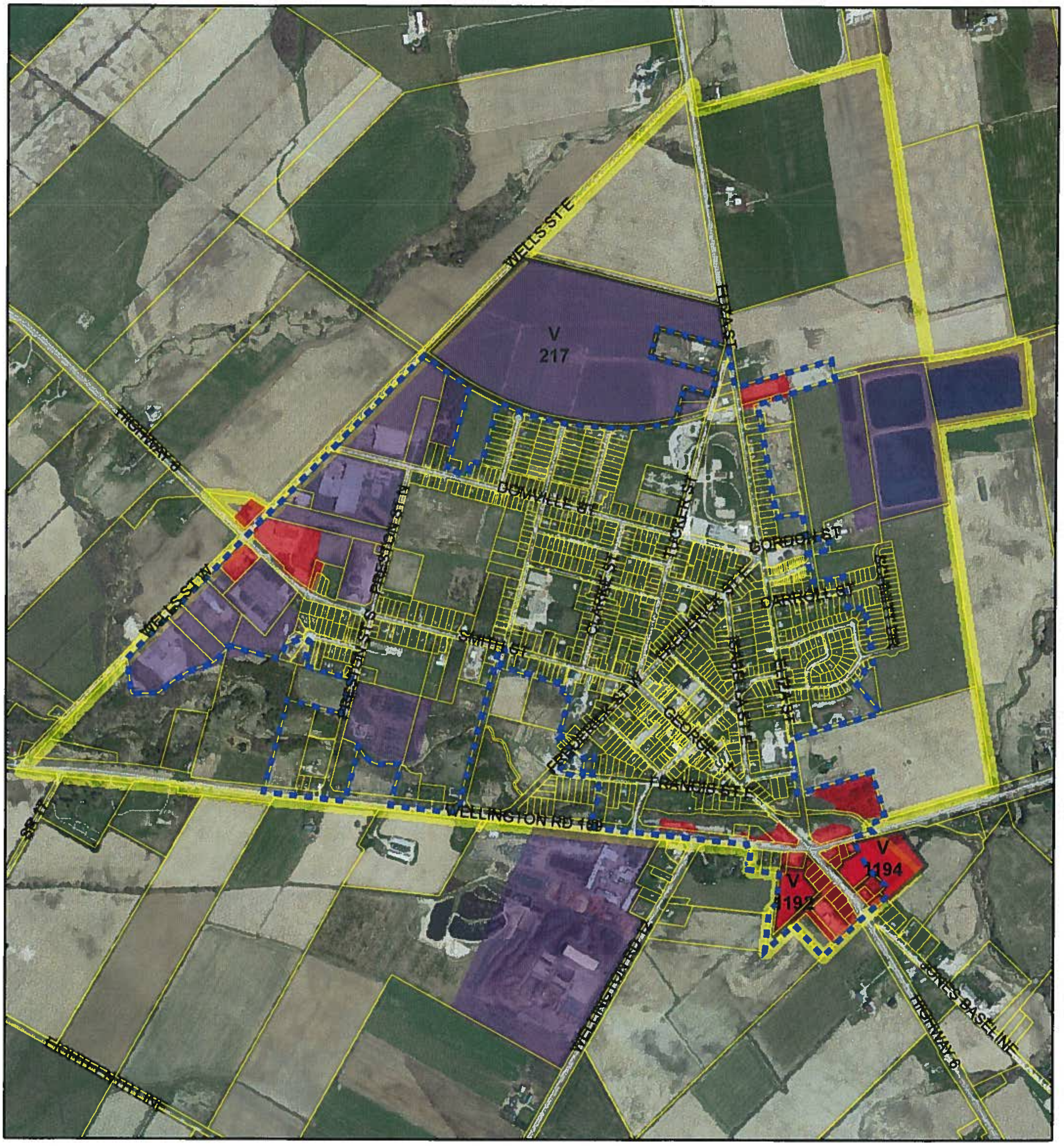
- Vacant Designated
- Application under review
- Draft Approved or Provisional
- Registered
- Built Boundary

N

0 95 190 380
Metres

Date Prepared: April 1, 2009.
File: F:\Land Information System\Draw\Mary\
Arthur-Built-Boundary-2008.mxd

Sources:
County of Wellington Planning and Development Department 2008. Ministry of Natural Resources.
Grand River Conservation Authority.
Upper Grand District School Board 2008.
First Base Solutions Licensed by the County of Wellington
Ortho imagery flown April 2008.



Designated Employment Lands
Arthur and Surrounding Area
Township of Wellington North

Map 4

Figure 2.2

Legend

OP Designation

- Industrial
- Highway Commercial
- V Vacant Parcel
- Urban Centre Boundary
- Provincial Growth Plan Built Boundary

Note: Numbers on the map represent Parcel Number

N

0 200 400 800

Metres

Date Prepared: January 23, 2009.
File: F:\Land Information System\
Draw\
Mary McElroy\Wellington North Comprehensive
Employment Review - Arthur 1.mxd

Sources:
County of Wellington Planning and Development
Department 2009. Teranet 2002.
Ministry of Natural Resources,
Grand River Conservation Authority.
Upper Grand District School Board 2009.
First Base Solutions Licensed by the County of Wellington
Ortho Imagery flown April 2006.

of industrial and 31 acres of highway commercial. Arthur future industrial development supply identified is limited to one large parcel (77.6 ha gross area, 62.1 ha developable area) that is located at the north end of the Arthur Urban Centre Boundary. This single parcel was included in the 2031 Scenario considered in this study. In addition, the two highway commercial land parcels identified in the County Report are included in the 2031 Scenario, both are located at the intersection of County Road 109 and Highway 6 with each parcel having a developable area of just over 4 hectares in size. Parcel 1192 (Avcom Investment Inc.) has been developed but is currently unoccupied.

2.3 Long-term Development Scenario:

The long-term development scenario (Long-term Scenario) was included in the servicing review in order to identify infrastructure servicing requirements beyond the 2031 Scenario, particularly in areas where servicing extensions are being considered. Although this scenario does not reflect a specific planning horizon, the additional areas included beyond the 2031 Scenario are considered as potential development areas that may affect design requirements of short-term infrastructure upgrades being considered. This scenario included the following growth components;

- 91 units of residential development (WN-52).
- 80 units of residential development (Future Eastridge, future phase of WN-54).
- 18.3 hectares of industrial development to the north of McCaulay St. between Wells Street East and Eliza Street.
- 19 hectares of industrial development along the Wells Street East frontage.

Future phase of WN-54 (Eastridge) has not been designated as residential at this time and is not identified in the County Report, however, there is a concept plan for the expansion of this development and it is felt that this development will eventually proceed. Given this, the development has been included in the servicing analysis to identify potential constraints.

Although these industrial parcels were not identified in the County Report, it was decided that additional industrial development areas should be included in this Long-term Scenario, since the entire industrial allocation included in the 2031 Scenario falls under the ownership of one land owner. These areas were selected in consultation with Township and County staff and are deemed to be a reasonable expansion of the existing industrial area.

The locations and limits of these assumed residential and non-residential developments are provided in Figure 2.3.

TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

LEGEND

INDUSTRIAL DEVELOPMENT

RESIDENTIAL DEVELOPMENT

FIGURE 2.3

LONG-TERM DEVELOPMENT SCENARIO

SCALE 1:10,000
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3.0 WATER SUPPLY, STORAGE AND DISTRIBUTION

3.1 Existing System

The Arthur water system is a single pressure zone watermain distribution network that is pressurized by two elevated water towers. Water is supplied to the system from three bedrock wells. This water system infrastructure is shown in Figure 3.1. This system currently provides service to 929 homes and 103 ICI (Industrial/Commercial /Institutional) properties according to Township records. The system also provides fire protection to the entire service area.

Operation of the system is controlled by a Supervisory Control And Data Acquisition (SCADA) system. Generally, well pumps are called on and turned off based on water levels in the storage towers. These levels are set by the operators based on storage requirements, volume turnover needs and well pumping constraints. The SCADA system also provides realtime monitoring and record keeping.

3.2 Source Capacity

Water supply is provided by three bedrock wells described below:

Well No. 7B:

- Located at 109 Wells Street West near the Conestogo River.
- Commissioned in 1998.
- 46 m deep drilled.
- Well pump is a submersible type complete with a 30kW (40 hp) motor which discharges directly to the distribution system (i.e., no highlift pumps).
- Rated capacity is 22.7 l/s and 1961 m^3/day .
- Disinfection using sodium hypochlorite. Contact time is provided by oversized discharge main.
- Iron sequestering treatment provided.

Well No. 8A/8B:

- Located on Part of Lots 20 and 21, Concession A approximately 1.15 km south of County Road 109 and 235 m east of Highway 6.
- Commissioned in 2005.
- Depth of wells are 61.9 m and 62.2 m for 8A and 8B respectively.
- Well pumps are submersible type complete with a 30kW (40 hp) motor which discharge directly to the distribution system (i.e., no highlift pumps).
- Rated capacities are 26.1 l/s (2255 m^3/day) each, however, the Permit to Take Water (PTTW) allows for the operator to pump either Well 8A or Well 8B, but not both wells concurrently. Therefore, total production from this facility is limited to 2255 m^3/day
- Disinfection using sodium hypochlorite. Contact time is provided by oversized discharge main.
- Treatment for high manganese is provided.
- Stand-by power provided by a diesel generator.

Based on the above, the total production capability (i.e., Source Capacity) of the system is 3930 m^3/day.

According to well records, well production from the three wells was not balanced, however, it is our understanding that the Township now operates the system to achieve balanced production from the

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TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

WATERMAIN SIZING:

- ≤100mm
- 150mm
- 200mm
- 250mm
- 300mm

FIGURE 3.1

WATER SYSTEM WATERMAIN SIZE

SCALE 1:10,000
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various water sources.

Based on well production records from 2008, 2009, and 2010, the three (3) year Average Day Demand (ADD) for the Arthur system was 908 m^3/day. Of this, it is estimated that approximately 40% of this demand is for ICI usage, with 40% of this used by Golden Valley Farms located on Wells Street West. The remainder of the water usage is residential or unaccounted for (i.e., leakage or flushing). These figures equate to an average day residential demand (i.e. ICI demand netted out) of 216 L/day per capita. Maximum Day Demand (MDD) of the system is calculated by averaging the maximum day production in each of the years considered. The MDD for the same 3 year period was 1422m^3/day which equates to a maximum day residential demand of 335 L/day per capita. The ratio of MDD to ADD is 1.57 for the Arthur water during this period. The data used to compile these figures is provided in Appendix C. It is worth noting that these figures are comparable to those provided in the document entitled, "Survey of Municipal Water Rates and Operations Benchmarking in Ontario (1999)" prepared by the Ontario Waterworks Association (OWWA Survey). This document has high and low average day residential usage of 351 L/d and 218L/d per capita, respectively, and an average MDD to ADD ratio of 1.75. It is reasonable that Arthur's current water demands would be at the lower end of the range given the introduction of watering restrictions and public awareness of water conservation since the 1999 study was completed.

In order to establish water demands for the analysis of the future growth scenarios being considered in this Master Plan, assumptions regarding future residential and ICI demands have been made. Residential demands were based on the average of the OWWA Survey, which equates to an ADD of 285L/day per capita. This figure was converted to a demand per residential unit using the current 2.76 persons per residential unit rate. Ministry of the Environment (MOE) Design Guidelines for Drinking-Water Systems 2008 (Guidelines) were used to establish ICI demands, a demand rate of 28 m^3/day/ha was used for the analysis. A MDD to ADD ratio of 1.75 was used to establish MDD for the future scenarios. The resultant demand rates (i.e., ADD and MDD) for the future development scenarios are as outlined in Table 3.1. Details of this calculation are included as Appendix C.

Table 3.1 Water Usage Projections

Scenario	Average Day Demand (m^3/day)	Maximum Day Demand (m^3/day)	MDD/ADD
Current	908	1422	1.57
2031	2078	3637	1.75
Long-Term	3255	5696	1.75

MOE Guidelines recommend that the Source Capacity should be greater than MDD so that daily demand can be met if storage is off-line. This criteria has been used to assess the capability of Arthur's water production infrastructure (i.e., wells) to satisfy future growth. As indicated by Table 3.1, it is apparent that the current Source Capacity of 3930 m^3/day is sufficient to meet the 2031 Scenario needs of the community. However, additional Source Capacity may be required to meet long-term development needs, these requirements should be confirmed periodically (i.e., every 5 years) as actual development/ usage progresses. Alternatively, it is recommended that annual Water Supply Reserve Capacity Calculations be completed in order that current usage and future needs can be monitored regularly. Despite the current surplus of source capacity, it should be noted that this capacity is obtained from only three sources. If any of these were compromised it would have a significant impact on the systems ability to meet MDD requirements. Therefore, it is crucial that the Township be proactive in securing future water sources since establishing these new sources, and the infrastructure required to deliver this water to the system, can be a lengthy, arduous process.

3.3 Storage

Storage for the Arthur water system is provided by two elevated facilities, a description of each is provided below:

Charles Street Tower:

- Located near the intersection of Charles and Isabella Streets in the southeast part of the system (195 Isabella Street South).
- Multi-legged steel tank
- Commissioned in 1932
- Volume is 227 m^3
- Operation range: 494.2 m - 499.6 m

Freud (Spheroid) Tower:

- Located just north of Smith Street between Preston and Wells Streets in the northwest part of the system (460 Smith Street).
- All steel spheroid tank
- Commissioned in 1967
- Volume is 1137 m^3
- Operation range: 494.0 m - 499.2 m

Based on discussions with Township staff, these facilities have been inspected recently and there were no significant deficiencies noted.The total system storage volume currently available is 1364 m^3.

Storage requirements for the water system are based on MOE Guidelines, the calculation is as follows:

Total Treated Water Storage Requirement = A + B + C

Where: A = Fire Storage (MOE suggested flow/duration based on population)
B = Equalization Storage (25% of MDD)
C = Emergency Storage (25% of A+B)

The calculated storage requirements for the current and future development scenarios are as outlined in Table 3.2.

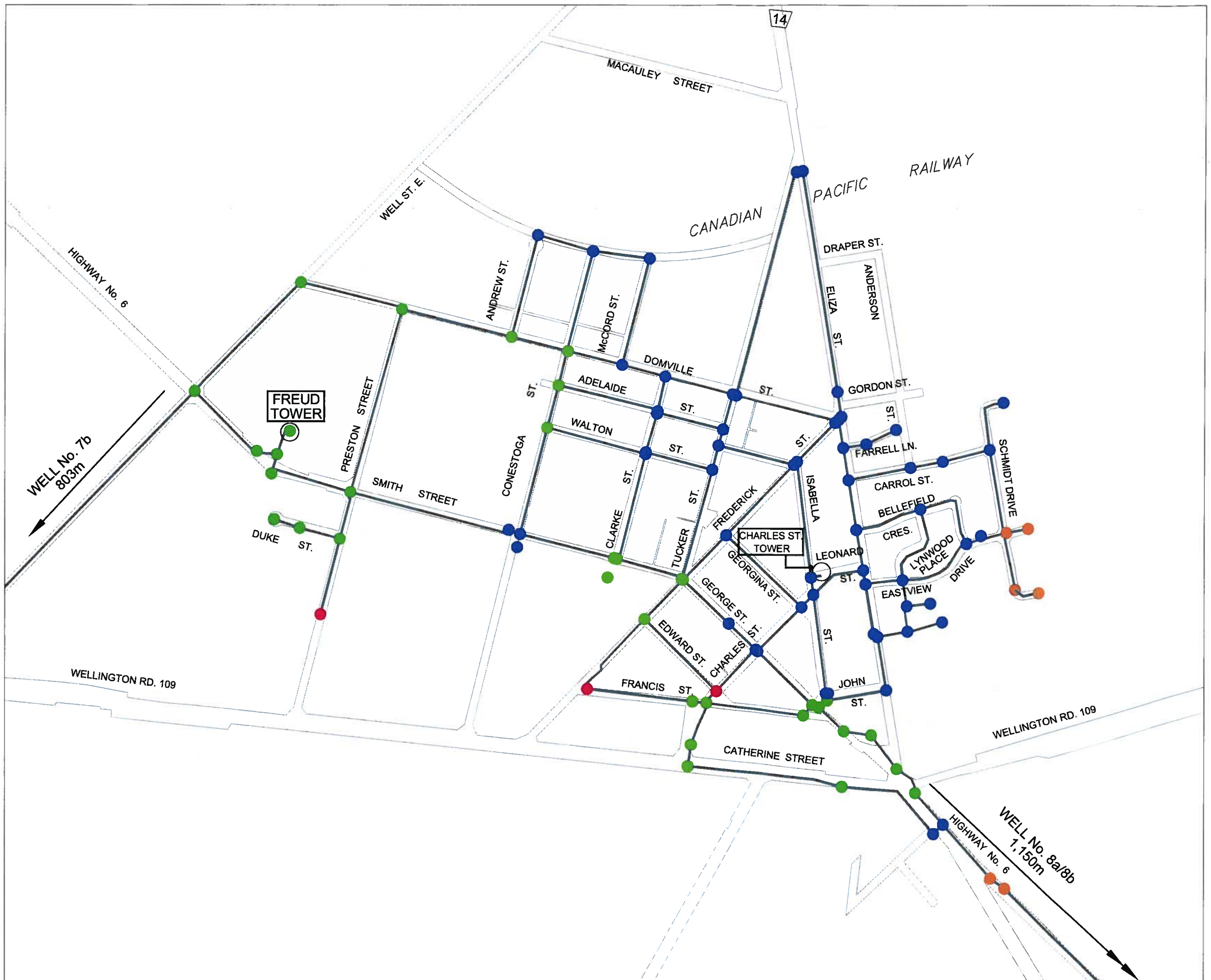
Table 3.2 Water Storage Requirement Summary

Scenario	Maximum Day Demand (m^3/day)	Suggested Fire Flow (L/s)	Storage Required (m^3)
Current	1422	103	1371
2031	3637	125	2262
Long-Term	5696	158	3558

Based on Table 3.2, the current storage volume available is equal to the storage requirement for the existing development. However, the MOE Guidelines indicate that the calculation is for systems where the water supply system is capable of satisfying only the MDD, and that where the supply available is greater than MDD, the storage requirement can be reduced accordingly. Given that the current source capacity is significantly greater than the MDD, the current storage volumes are considered adequate for the immediate future.

Considering the 2031 Scenario, the storage requirement is significantly above the current volume available, and unlike the current situation, the 2031 MDD is approaching the Source Capacity. Therefore, additional storage should be added to the system prior to the 2031. Other factors such as retirement of the existing water towers (i.e., Charles Street Tower) should be considered when planning for a new facility, both the volume and timing of such a facility. Calculation details are provided in Appendix D. It should be noted that the current Development Charges (DC) Bylaw includes provisions for a new water tower in 2025. The Development Charges Bylaw cost allowance provided for this facility should be updated with the next DC Bylaw amendment to ensure it reflects current costs.

As part of this Master Plan, a computer simulation model (i.e., WaterCAD V8i) of the Arthur water system was created. This model indicates that the normal pressures throughout the existing distribution system range from approximately 45 psi to 80 psi for the current tower operating range. *Figure 3.2* provides pressures throughout the system under normal operating conditions. These pressures are within an acceptable range. However, it should be noted that the potential future development areas to the north, east and south of the existing urban area are located at higher topographic elevations. Given this, it is recommended that consideration be given to increasing the operating range of a future elevated tower to adequately service these areas. An increase of approximately 10 m, to a maximum operating level of 510 m, would provide an overall pressure increase of 14 psi to the system. This would also improve the fire flow capability within the system. The maximum system pressure under this operating level would be 92 psi which is acceptable. The difficulty of implementing this strategy is that the existing towers would operate at a lower level but would be in the same pressure zone. Therefore, the existing towers would have to be decommissioned, or separate pressure zones would need to be established. Creating multiple pressure zones in a system increases the operating complexity and infrastructure requirements and therefore should be avoided if possible. Normally decommissioning both water towers at the same time would not be recommended, however, in considering the age and size of the existing towers and the long term benefit of increased operating levels, replacing these towers in the short-term (i.e., 10 -15 Years) with one at a higher operating level and with sufficient volume to meet future demands is reasonable.



TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

JUNCTION PRESSURE RANGES

- 40 - 50 psi
- 50 - 60 psi
- 60 -70 psi
- 70 - 80 psi

FIGURE 3.2

TYPICAL WATER SYSTEM PRESSURE

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In summary, the reasoning for this recommended approach is outlined below:

- Decommissioning of the Charles Street Tower is recommended given it's age, limited volume and on-going remedial/maintenance costs.

- Freud Tower is well into it's service life and may require significant upgrades (i.e., mixing/ rechlorination system) and maintenance (i.e., painting, corrosion treatment) within the planning horizon of the required additional storage.

- The increase in volume required to service future development would result in a relatively small tower size. This size would have a higher cost per unit volume than a larger sized tower which could provide the entire required volume. Therefore, the cost of replacing the existing volume in a new tower would likely be less than the cost to remediate and upgrade the existing towers.

- The operating complexities of three towers in a system verses one.

- Retaining the existing towers in the system would not allow for the higher operating range unless a separate pressure zone is created. A dual pressure would increase the complexity of the system and limit the benefit of the increased operating level.

- The new facility would be equipped with improved features (i.e., separate fill/draw risers, provisions for mixing/re-chlorination, improved safety/accessibility).

With respect to the location of a new tower, utilizing one of the existing sites was considered. The Charles Street Tower site is too small and constrained by existing development. The Freud Tower site is feasible, however, there are several considerations as outlined:

  - The site may be large enough to accommodate a new larger tower, however, there is not enough room for the existing tower to remain in place while the new tower is constructed. Therefore, the available storage in the system would be reduced significantly (i.e., to volume of Charles Tower) for an extended period (i.e., up to a year) while the Freud tower is removed and the new tower constructed. Although this may not impact day to day servicing, it would have an impact on the fire flows available from the system during this period.

  - It is situated near the industrial area and located adjacent to a trunk watermain, therefore, it will provide adequate fire flows to critical areas of the system.

  - The site is at a moderate ground elevation, therefore, the height of the new tower, assuming the higher operating level, would be considerable (i.e., approx. 52m) resulting in an increased cost for the facility.

Alternately, a new site could be selected which would allow the existing facilities to remain in service during construction. Preferably a site at a higher elevation to reduce the tower height, located in or near existing/future industrial development would be preferable (i.e., north end of system). If a new site is considered, it is important that the replacement or extension of watermains consider the new site and are sized accordingly.

Establishment of a new water storage facility and the decommissioning of the existing facilities are both considered Schedule B projects under the Municipal Class EA. This Class EA would be completed in the future as development warrants the new facility. At that time details regarding sizing, configuration, operating levels and location can be confirmed along with the status of the existing facilities will be considered.

3.4 Distribution Network

The distribution network presently services all existing developed areas within the urban centre boundary. The network includes approximately 19.6 km of watermain ranging in size from 50 mm to 600 mm with 1032 services. Type of watermain used to construct the network has varied over the years and includes cast iron, ductile iron and PVC. Any recent upgrades or extensions have been PVC. A breakdown of the network by size and type is provided in Table 3.3 and Table 3.4 and illustrated in Figure 3.1 and Figure 3.3, respectively. The network includes a trunk main consisting of 250 mm and 300 mm pipe which runs from the Wells 8A/8B along Jones Baseline, Highway 6, George Street and Smith Street past the Freud Tower to Wells Street and along Wells Street to Well 7B. A complete watermain inventory and reference plan is provided in Appendix E.

Table 3.3 Watermain Size Breakdown

Description	Length (m)	Percentage (%)
50 mm Dia.	34	0.15
150 mm Dia.	13377	68.3
200 mm Dia.	1478	7.5
250 mm Dia.	1428	7.3
300 mm Dia.	3170	16.2
500 mm Dia.	69	0.35
600 mm Dia.	40	0.2
Total Main Length (m)	**19596**	

Table 3.4 Watermain Material Breakdown

Description	Length (m)	Percentage (%)
PVC	15218	77.6
Ductile Iron	937	4.8
Cast Iron	3440	17.6

The Watercad model was used to estimate the fire flows capabilities throughout the network, results of this analysis are illustrated in Figure 3.4 with details provided in Appendix E. This analysis uses 20 psi

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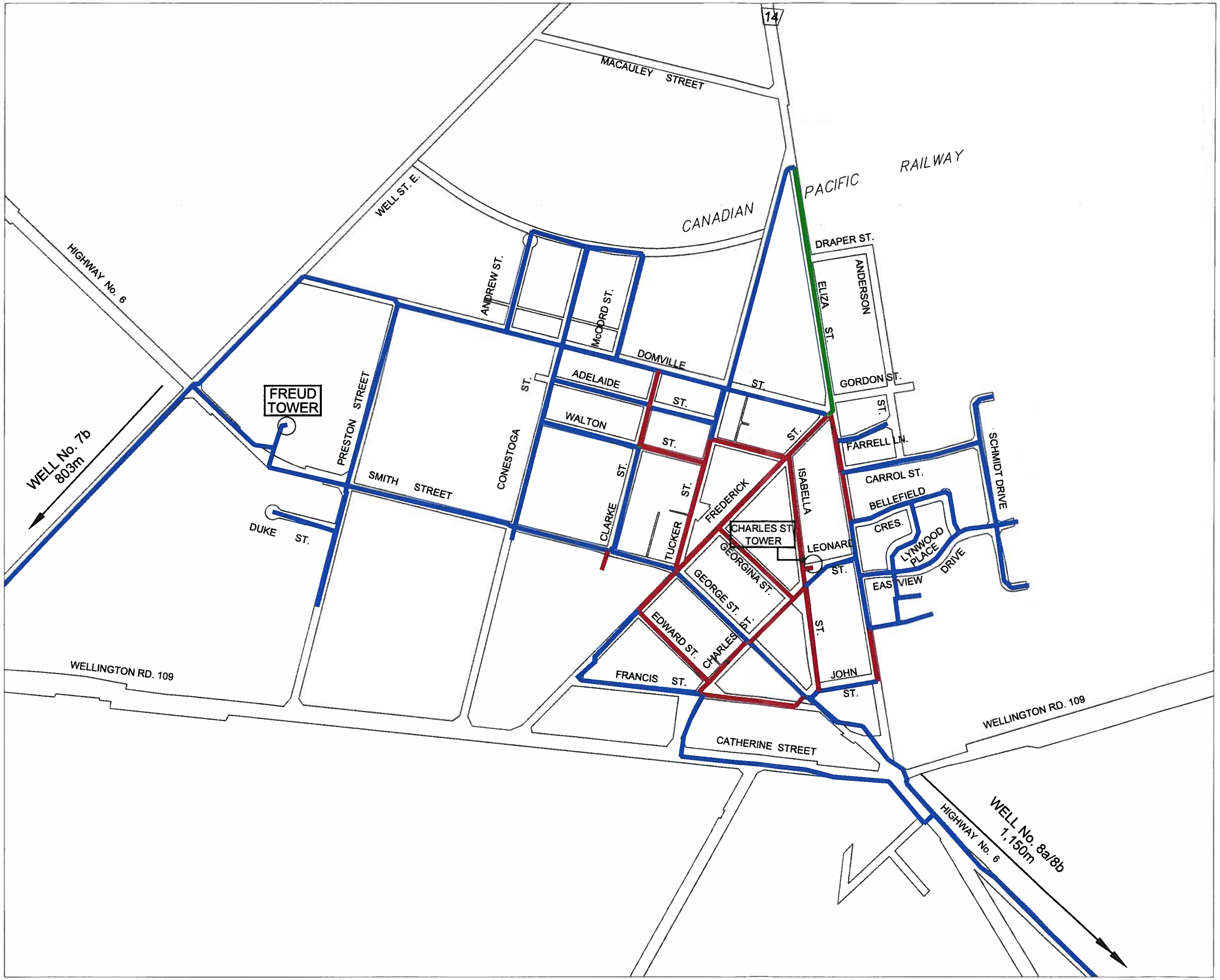

WATERMAIN MATERIAL

- PVC
- CAST IRON
- DUCTILE IRON

FIGURE 3.3

WATER SYSTEM WATERMAIN MATERIAL

SCALE 1:10,000
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TOWNSHIP OF WELLINGTON NORTH

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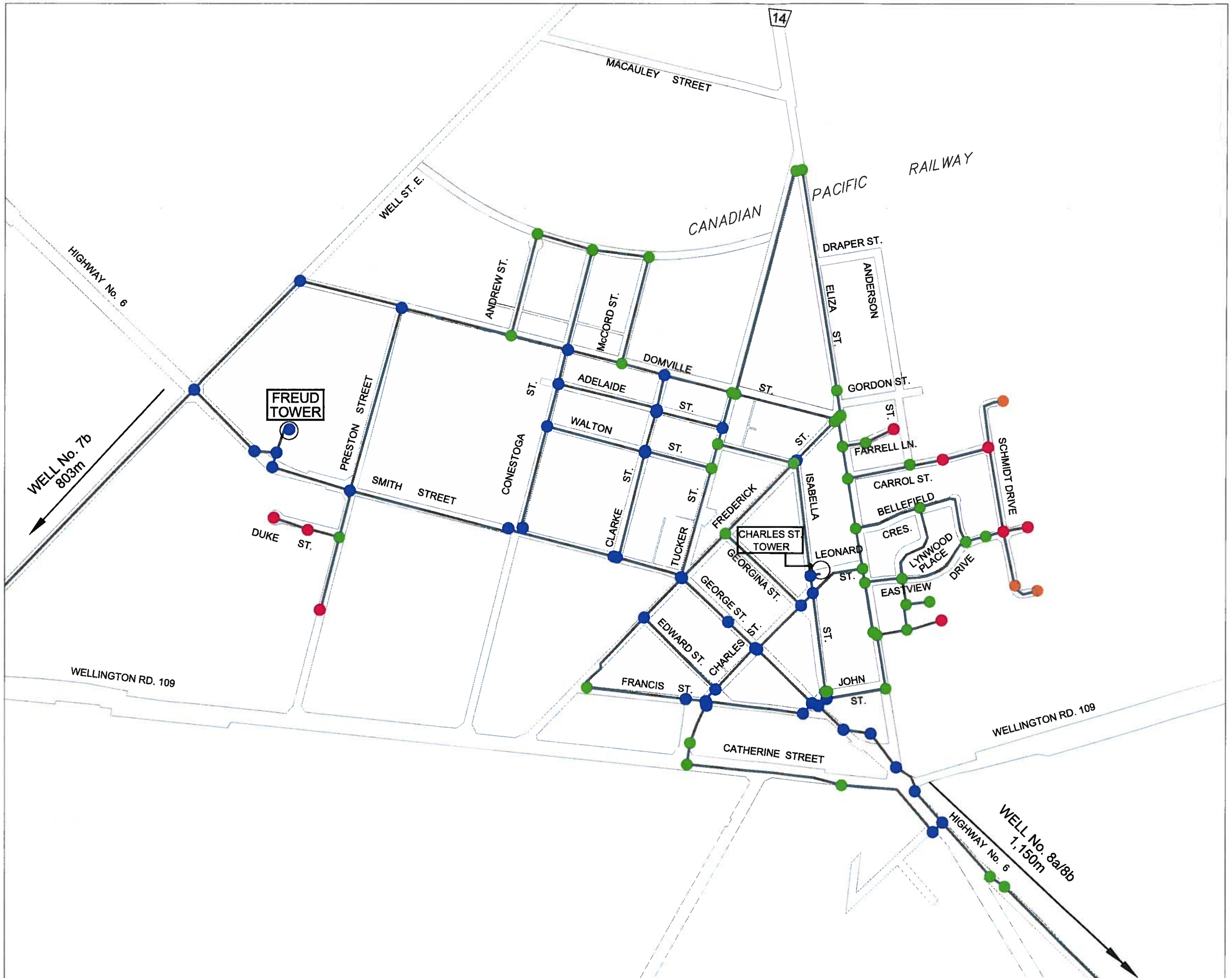

FIGURE 3.4

FIRE FLOW CAPABILITIES

SCALE 1:10,000
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as the minimum residual pressure to establish water taking capability which is the normal accepted industry standard for firefighting. Further, the scenario illustrated in this figure assumes maximum day demand on the system and that municipal wells are running to support the fire effort. It should be noted that this analysis reflects the available flow from the mains at a location in the system, not necessarily at a specific hydrant(s). Hydrant flow would be dependant on the number of hydrants and ports used. Another important consideration is that this analysis provides the available flow at the start of the fire, with the limited volume of the Charles Street tower the flow capability in the area adjacent to it will drop off quickly.

Generally, higher fire flows are required in industrial and high density commercial area (i.e., downtown core), with actual fireflow requirements being site specific. For evaluation of the existing system, the MOE population based recommended figure of 103 l/s was used. Further, the MOE recommends a minimum 30 l/s capability throughout a system that provides firefighting service. Based on these criteria, the Arthur system does provide adequate fire flows throughout the network. The lower flow areas are generally restricted to deadend areas. Fire flow scenarios can be simulated using the model to assess system performance for specific properties/developments or system conditions, as required.

Recommendations for network improvements include the following:

- *Replacement of older cast iron mains*:

  These pipes have reached the end of their service life and become high maintenance items. Also, corrosion and scale buildup is normally excessive resulting in poor hydraulic performance. These mains should be replaced as these streets are reconstructed. Figure 3.5 illustrates the locations of these mains. It is noted that there is only one ductile main on the system (i.e., Eliza Street from Tucker to Domville). The condition of this main should be reviewed prior to the reconstruction of this section of road.

- *Looping of dead-end mains*:

  Dead end mains are of concern since water tends to stagnate unless there is sufficient local demand, this results in low chlorine residual. Often watermain flushing is required to maintain water quality which is an additional maintenance and a waste of water. Another issue is that there is only one feed to the main, therefore, if the main has to be temporarily turned off (i.e., maintenance or break), the entire length of the street will be without water. Generally, the Arthur network is well looped with few dead-ends, those that do exist (i.e., Preston Street South, Duke Street) are situated such that looping options are not readily available. However, this issue should be addressed as feasible during system extensions and future development.

- *Extension of trunk mains*:

  It is important that the network has a sufficient capacity to convey large volumes of water to/from critical areas such as storage facilities, industrial area and the downtown core. In addition, it is important that the system is “looped” to ensure that there is redundancy in case a trunk main has to be shutdown (i.e. maintenance or breakage). As indicated by the modeling, the existing system has adequate conveyance capacity to service the existing development . With the future expansion of the industrial area, it is recommended that trunk mains (i.e., 300 mm diameter pipes) be extended on Wells Street East to Macauley Street, along Macauley Street to Eliza and on Eliza

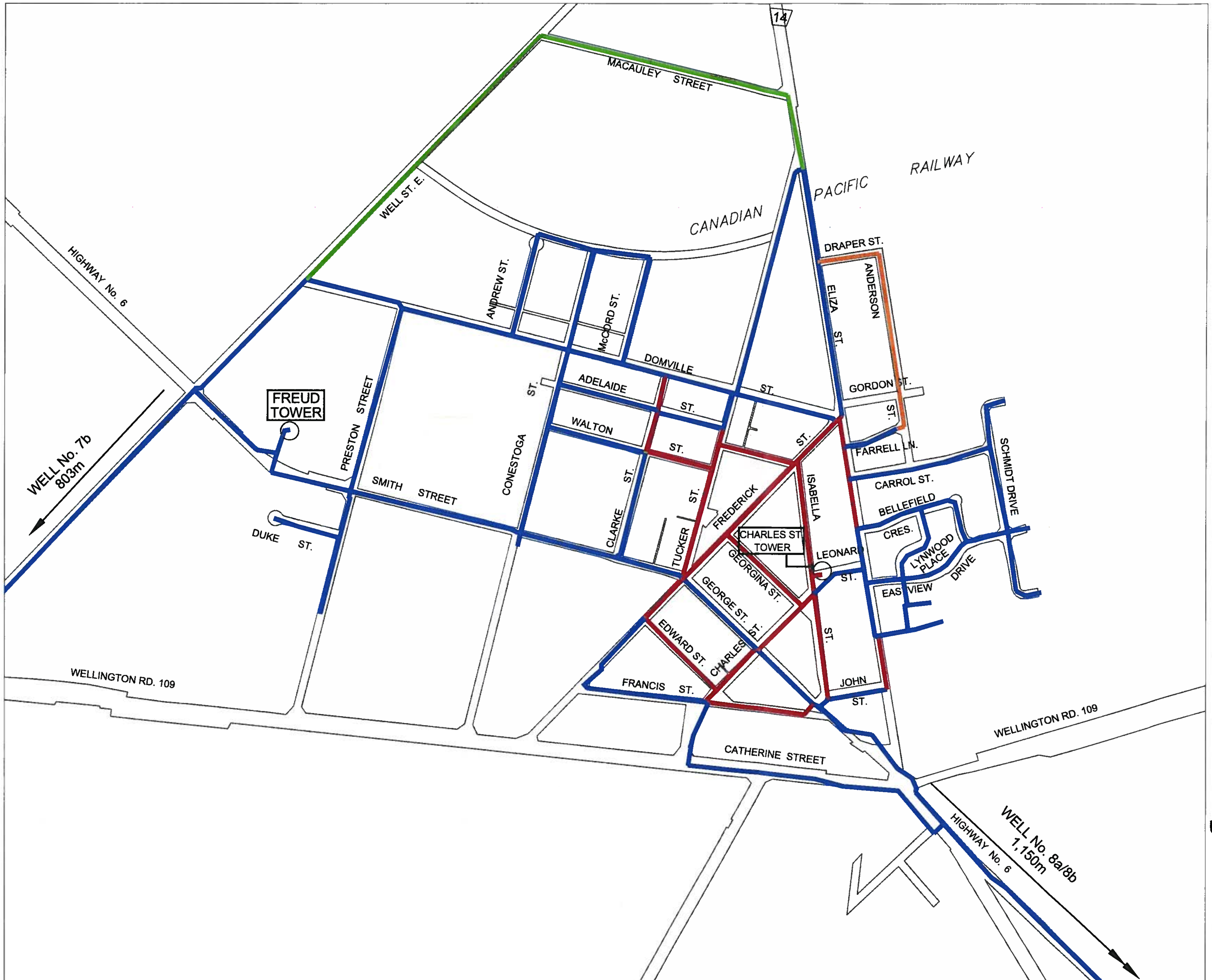
Street to complete the loop. However, the optimum routing for these trunks may depend on the configuration of the developments to a certain extent and should be reviewed in the context of future development proposals. These trunk main loops will not only provide servicing to the future development areas but will provide redundancy and improve flows to the entire. Local extensions of the system will be required to accommodate future development areas (i.e., Anderson and Draper Streets), these mains are to be 150 mm diameter. It should be noted that not all of these projects are currently included in the Development Charges (DC) Bylaw, or the details of the projects may have changed. Therefore, it is recommended that the DC Bylaw be amended to reflect servicing strategy and projects outlined in this report.

Figure 3.5 Illustrates the locations of these improvements and Table 3.5 provides details.

Table 3.5 Recommended Water System Upgrades and Extensions

Location	Pipe ID	Size (mm)	Length (m)	Cost ($)
Cast Iron Watermain Replacements				
Edward St. (Frederick St. to Charles St.)	P-138	150	255	$176,000
Charles St. (Edward St. to Isabella St.)	P-113,P-114,P-117,P-65	150	346	$240,000
Georgina St. (Frederick to Charles Sts.)	P-66	150	264	$183,000
Isabella St. (Frederick St. to John St.)	P-57, P-58, P-64	150	576	$398,000
Old Tower (Isabella St. into Old Tower)	P-59	150	25	$18,000
Frederick St. (Edward St. to Eliza St.)	P-136, P-67,P-50, P-49, P-137	150	677	$467,000
Isabella St. (Tucker St. to Frederick St.)	P-51	150	198	$137,000
Walton St. (Clarke St. to Tucker St.)	P-127	150	171	$118,000
Eliza St.(Frederick to Bellefield Cres.)	P-36, P-38, P-41	150	281	$194,000
Clarke St. (Walton St. to Domville St.)	P-69, P-129	150	191	$132,000
Eliza St. (John St. to Carresant Care)	P-14	150	136	$94,000
Tucker St. (George St. to Adelaide St.)	P-126, P-130, P-71	150	389	$269,000
System Extensions				
Wells St. (Domville St. to Macauley St.)	P-198	300	850	$676,000
Macauley St. (Wells St. to Eliza St.)	P-200	300	640	$510,000
Eliza St. (Macauley St. to Tucker St.)	P-201	300	191	$152,000
Draper St. (Eliza St. to Anderson St.)	P-196	150	155	$107,000
Anderson St. (Draper St. to Farrell Lane)	P-197	150	440	$305,000

Note: Cost estimates are based on 2010 construction costs and assume that watermain is installed as part of roads works. Allowances for engineering, contingency and HST have been included.



TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

LEGEND

- EXISTING WATERMAIN TO REMAIN
- REPLACEMENT OF CAST IRON
- TRUNK MAIN EXTENSION
- LOCAL MAIN EXTENSION

FIGURE 3.5

RECOMMENDED WATER UPGRADES AND EXTENSIONS

SCALE 1:10,000
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4.0 WASTEWATER COLLECTION AND TREATMENT

4.1 Existing System

The Arthur wastewater system includes a dedicated sanitary sewer collection network, two sewage pumping stations (SPS), a wastewater treatment plant (WWTP) and a effluent storage lagoon facility. The wastewater system infrastructure is shown in Figure 4.1. This system currently provides service to 929 homes and 103 ICI properties according to Township records. The system services the entire developed area of Arthur.

The collection network is divided into three (3) service areas as follows:

Wells Street SPS:

> The Wells Street SPS pumps via a 1 km - 150 mm diameter PVC/AC forcemain to a manhole at the intersection of Preston and Smith Streets. This SPS receives primarily industrial flows from the industry located in the west side of the town.

Preston Street Trunk Sewer:

> Preston Street trunk sewer services Preston Street and the western portion of Domville Street along with the Wells SPS discharge. This area flows by gravity directly into the Arthur WWTP and services a mix of residential and industrial users.

Frederick Street SPS:

> The Frederick Street SPS receives the majority of the flows in the community including the central, southern and eastern portions of the system. It pumps directly into the WWTP through a 750 m long, 250 mm diameter forcemain. This SPS services primarily commercial and residential flows.

The Arthur WWTP discharges to the Conestogo River, however, due to assimilative capacity limitations of the river, the discharging to the river is restricted to September 16 - April 30. The remainder of the year WWTP effluent must be pumped to the effluent storage lagoons to be stored until discharging to the river is permitted. Effluent from the effluent storage lagoons is fed via gravity through the forcemain into the WWTP for filtration and disinfection prior to discharge to the river. The original outfall sewer from the lagoons to the river still exists but only acts as an emergency overflow.

4.2 Sanitary Sewer and Forcemain Collection Network

The network presently services all existing developed areas within the urban centre boundary. The network includes approximately 19.1 km of sewer ranging in size from 150 mm to 450 mm, 4.4 km of 150 - 250 mm forcemain and 1032 services . The type of sewer pipe used to construct the network has varied over the years and presently includes asbestos cement, concrete and PVC. Any recent upgrades or extensions have been PVC. The forcemain is constructed of PVC and asbestos cement. A breakdown of the network by size and type is provided in Table 4.1 and Table 4.2 and illustrated in Figure 4.1 and Figure 4.2, respectively. A complete sanitary sewer/forcemain inventory and reference plan is provided in Appendix F.

TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

SEWER SIZING:

- 200mm
- 250mm
- 300mm
- 375mm
- 450mm
- FORCEMAINS
- LAGOON OVERFLOW

FIGURE 4.1

SANITARY SYSTEM SEWER SIZE

SCALE 1:10,000
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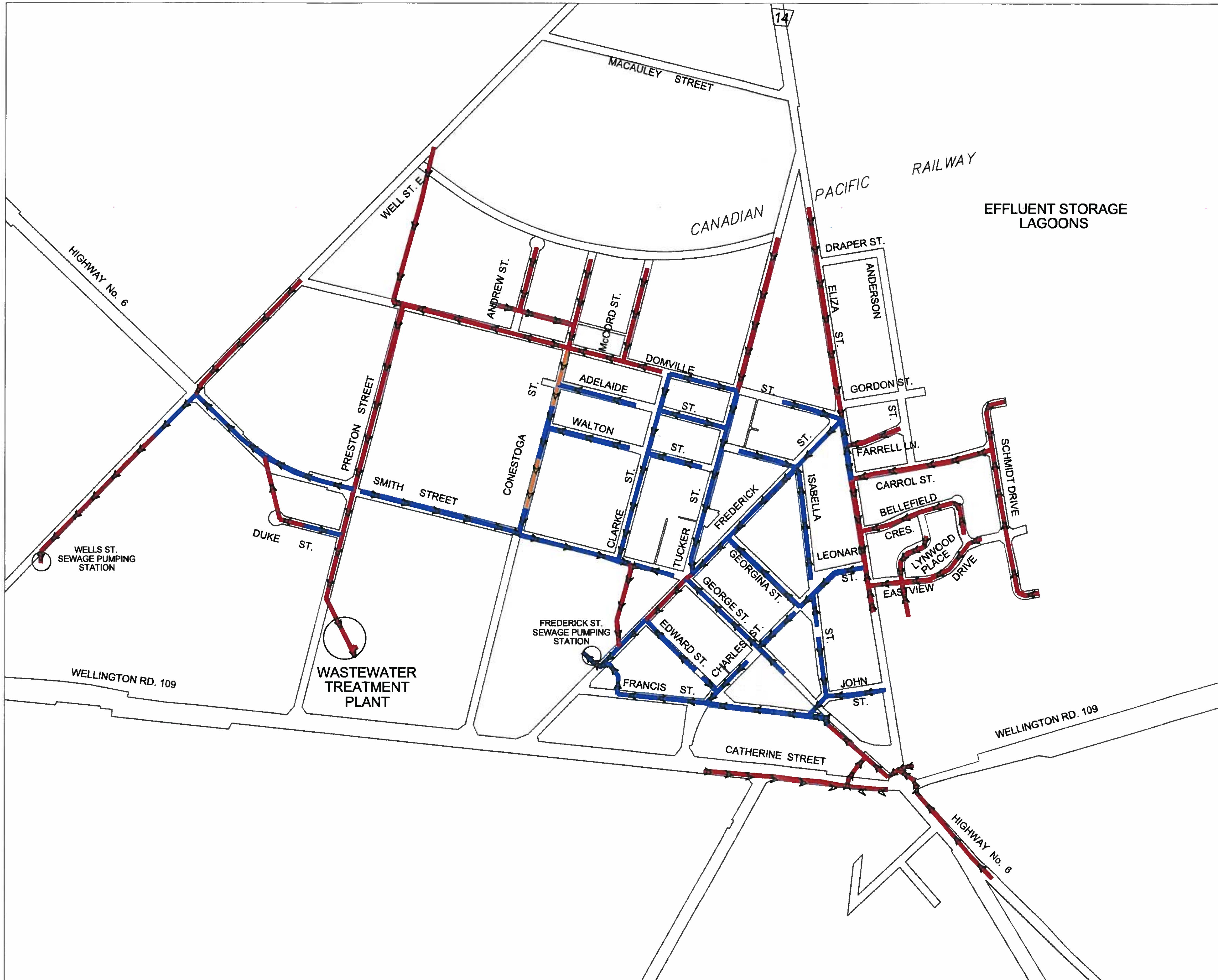
TOWNSHIP OF WELLINGTON NORTH

SANITARY MASTER PLAN (ARTHUR)



LEGEND:

- CONCRETE
- PVC
- ASBESTOS CEMENT

FIGURE 4.2

SANITARY SEWER MATERIAL

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Table 4.1 Sanitary Collection System Size Breakdown

Description	Length (m)	Percentage (%)
Sewer		
150 mm Dia.	80	0.4
200 mm Dia.	14992	78.5
250 mm Dia.	1616	8.5
300 mm Dia.	1068	5.6
375 mm Dia.	923	4.8
450 mm Dia.	429	2.2
Total Main Length (m)	19108	
Forcemain		
150 mm Dia. (Wells St. SPS)	1040	24
250 mm Dia. (Frederick St. SPS)	786	18.1
200/ 250 mm Dia. WWTP To/From Lagoon	2510	57.9
Total Main Length (m)	**4336**	

Table 4.2 Sanitary Collection System Material Breakdown

Description	Length (m)	Percentage (%)
Conduit Material		
Asbestos Cement	9163	48
Concrete	280	1.5
PVC	9665	50.5
Pressure Pipe		
PVC	**2336**	
Asbestos Cement	**2000**	

As part of this Master Plan, a computer simulation model (i.e., SewerCAD V8i) of the Arthur sanitary collection system was created. The SewerCAD model (i.e., "model") was used to estimate peak flows throughout the network and compare them to the hydraulic capacity of the various sewers and forcemains. This model is a very useful tool in troubleshooting existing problems, assessing system capabilities, and as a planning tool for short and long term sewer infrastructure upgrades.

Accurate estimation of flows is essential to realize the full benefit of the model, however, this can be a difficult process since sewer flows are not normally monitored. Although, wastewater flow records at the WWTP are available and if used to calibrate the model would provide good correlation for the normal day flows, these are daily averages are not particularly useful since sanitary sewer capability is assessed under peak flows during extreme conditions (i.e., high inflow). Generally, estimation assumes that the water usage will be similar to the wastewater discharged from the various users connected to the system. Peaking factors are applied to these flows to generate daily peak flows. In addition, an infiltration/inflow (I/I) allowance is added to reflect infiltration through sewers and manholes, and for non-sanitary connections (i.e., sump pumps, roof leaders, storm connections). Despite efforts to achieve an accurate representation of peak flows throughout the system, the process is reliant on empirical data and assumptions which may not be entirely representative of all components of the system. Ideally, an extensive flow monitoring and inspection program is needed to confirm the accuracy of the model, however, this is a costly and time consuming endeavour. Although, this study did not undertake such an extensive program, a limited sewer monitoring program was undertaken by the Township in order to assist in the calibration of the model.

The monitoring was completed using continuous flow monitoring equipment installed in manhole MH174 located on Frederick Street between Francis and Edward Street. The monitoring included the inflow streams from MH 172 (Frederick Street) and MH 173 (Frederick-Smith easement). MH174 manhole is just upstream from the Frederick Street pumping station and represents a majority of the flow that enters the pumping station. Flow monitoring occurred from March 11 to May 19, 2010. Unfortunately, the rainfall during the monitoring period was extremely low for the spring season with a limited number of significant events. Therefore, contributions to sewer flows from I/I sources are expected to be lower than spring season normals.

The monitoring data reflected a fairly consistent flow pattern throughout each day with peaks as expected (i.e., early morning and evening). The assessment of the monitoring data included a comparison with precipitation for the area. As expected, this comparison does indicate a correlation to rainfall events, with peak flows increasing 50 - 100% during significant (i.e., >12 mm rainfall) events. Although, this is a significant increase in peaks, it is not to the scale that would suggest that there are major stormwater cross connections in the system. Also, the data appears to indicate that there is a delay (i.e., 12-24 hours) between the sewer peaks and the rainfall event, however, given the limitations of rainfall data available this is difficult to confirm. This delay in flow response suggests that the inflow is more likely due to infiltration in the system and foundation drain connections rather than direct storm connections (i.e., storm sewer connection or roof leaders). It should be noted that the Village of Arthur completed a review of the sanitary sewers in 1993 to identify deficiencies and stormwater cross connections that contributed to the I/I flow experienced at that time. Subsequently, the Village repaired these deficiencies and disconnected storm water cross connections. This would suggest that the majority of the I/I flow is emanating from the services not the sewer itself. However, a more intensive monitoring program would be required in order to confirm the exact location(s) and nature of the I/I flow. The monitoring results and estimation of flows is discussed further in Appendix G.

In addition to monitoring results, personnel experienced with the system flows were consulted. According to Township works staff, in 2010 there were incidents of sewer surcharging upstream of the Frederick - Smith Easement. Although it is difficult to confirm, this problem was deemed to have been caused by hydraulic inefficiencies at the “T” connection at the top end of the Smith-Frederick trunk sewer easement and possibly the sewer capacity itself. In the fall of 2010, this 200 mm sewer was replaced with a 300mm diameter pipe. In addition, the manhole orientation was adjusted to create a "Y" junction thereby improving the flow characteristics from the Smith Street sewers to the easement trunk. It should be noted that Township and Triton personnel involved with this project had witnessed that the existing 200mm pipe had considerable flow (i.e., ½ full) on occasion. This would imply that flows are considerably higher than those reflected in the monitoring data. There haven't been any reports of surcharging since the easement sewer was replaced.

Using a combination of water use records, WWTP flows, monitoring results and historical experience, peak flows were derived for the model to be used in the analysis. To assess the hydraulic adequacy of the collection network, the peak flows including I/I inflows were compared to theoretical sewer capacities. Results of this analysis under peak flow conditions are provided in Figures 4.3-4.5. Based on this analysis, the collection system has sufficient hydraulic capacity under existing development conditions. However, under the 2031 and Long-term Scenarios, hydraulic capacity in some sewers may be exceeded depending on the sewer routing of future development. Specifically, if the industrial development area 217 (between CPR and MacCauley Street) was routed into either the Tucker, McCord or Andrew Street sewers there may be capacities constraints in downstream sewers (i.e., Conestoga Street). Therefore, it is recommended that this area be directed to the Preston Street trunk sewer to the extent possible to ensure that available sewer capacity is preserved for other development areas in the system which would not have access to the Preston Trunk. Figures 4.4 and 4.5 reflect this servicing strategy. Further, directing flow to the Preston Trunk eliminates the need to pump this wastewater to the WWTP, thereby reducing operating costs and preserving capacity in the Frederick SPS for other development areas. It is recognized that some of development area 217 may have to be directed to the other sewers because of the future development configuration and grading. If this does occur, the model could be updated to reflect the development proposal and the available sewer capacity could be confirmed. With respect to the Long-term industrial development areas northeast of MacCauley Street and northwest of Wells Street, these areas should also be directed to the Preston Trunk to the extent possible. However, due to grade restrictions the areas northwest of Wells Street south of Domville Street would have to be serviced by the Wells SPS. Figure 4.6 illustrates possible sewer extensions to service these future development areas utilizing existing right-of-ways. However, depending on the configuration of the developments, these sewers may be extended internal to the development areas. The optimum sewer configuration should be considered as development proposals are brought forward. Table 4.3 provides details regarding the extensions shown in Figure 4.6.

Based on the modelling, the remainder of the existing collection system generally has sufficient available capacity to accommodate the assumed design flows generated by the 2031 and Long-term Scenarios. The model did identify a couple of areas where theoretical flows were approaching sewer capacity under future development conditions. First, one length of sewer along Frederick Street immediately below Georgina Street that is nearing available capacity, upgrading of this sewer down to George Street is recommended to accommodate future development. Similarly, the lower reach of Georgina Street immediately upstream of Frederick Street is nearing capacity under future development flows. Actual flows at these locations should be confirmed with local flow monitoring in the future prior to build-out of the 2031 scenario or replacement of sewers.

Although the assumed design flows in this analysis are relatively conservative, industrial flows are highly variable, therefore, the Township will need to re-evaluate sewer capacity prior to committing to high water use industries in order to identify the impact on municipal infrastructure. As recommended previously, an extensive monitoring and inspection program would be invaluable to confirm design flows and identify I/I, but in it's absence local monitoring on specific areas of concern should be undertaken prior to committing to major developments, or to infrastructure up-grades to confirm necessity and design requirements.

It should be noted that not all of the projects identified in Table 4.3 are currently included in the Development Charges (DC) Bylaw, or the details of the projects may have changed. Therefore, it is recommended that the DC Bylaw be amended to reflect servicing strategy and projects outlined in this report.

Regarding the physical condition of the sewers, it is recommended that all sewers be videoed prior to reconstruction of a street in order to assess the condition of the pipe and to identify any I/I flow sources. It should be noted that much of the system is comprised of Asbestos Cement (AC) piping which was installed in the late 60s or early 70s. Despite it's age, this pipe tends to be in good condition, however, the video should be reviewed carefully to confirm this.

Figure 4.6 Illustrates the locations of these improvements and Table 4.3 provides details.

Table 4.3 Recommended Sanitary Sewer Upgrades and Extensions

Location	Pipe ID	Size (mm)	Length (m)	Cost ($)
System Extensions				
Anderson St. (Gordon St. to Farrell Lane)	N/A	200	433	$292,000
Domville St. (Wells St. to Preston St.)	N/A	200	237	$160,000
Wells St. (CPR to Domville St.)	N/A	200	495	$332,000
Wells St. (Macauley St. To CPR)	N/A	300	360	$272,000
Macauley St.(Wells St. to Eliza St.)	N/A	300	315	$240,000
Recommended System Upgrades				
Georgina St. (Frederick St. To MH-120)	CO-53	250	84	$ 58,000
Frederick St. (George St. to Georgina St.)	CO-52	300	124	$ 95,000

Notes: Cost estimates are based on 2010 construction costs and assume that sewer is installed as part of road works. Allowances for engineering, contingency and HST have been included.

4.3 Sewage Pumping Stations

As indicated there are two sewage pumping stations (SPS) included in the sanitary system, a summary of each is provided below:

Wells Street SPS:

- Located on the west limit of Wells Street West near the Conestogo River.
- Service area includes Wells Street West /East and Smith Street between Wells and Preston Streets.
- Pumps to the Preston Street trunk sewer at the intersection of Preston and Smith Streets through a 1 km long,150 mm diameter PVC/AC forcemain.

- Duplex configuration with two fixed speed pumps rated at 16 l/s @ 31m TDH each.
- Stand-by power provided by diesel generator.

Frederick Street SPS:

- Located near the intersection of Frederick and Francis Streets.
- Service area includes the majority of the serviced area with the exception of the area tributary to the Wells Street SPS and Preston Street Trunk.
- Pumps directly into the Arthur WWTP through a 750 m long, 250mm diameter PVC forcemain.
- Duplex configuration with two variable speed pumps rated at 58.4 l/s @14m TDH each equipped with variable speed drives.
- Stand-by power provided by diesel generator.

Based on discussions with system operators, the Wells Street SPS has been able to convey all flows directed to it with no bypassing. Modeling analysis concurs with this assessment and indicates that just one of the pumps should be capable of conveying existing normal peaks. Further that the forcemain is adequately sized based on acceptable velocities. Assuming that the majority of the industrial development along Wells/MacCauley Streets, as reflected in the Long-term Scenario, is directed to the Preston Street trunk sewer, the Wells SPS and forcemain should be adequate to service Long-term development. However, prior to allowing significant "wet" development along Wells Street below Domville, the capacity of this infrastructure should be reviewed in detail based proposed wastewater loading.

Within the period of approximately 2006-2009, the sewer system near the Frederick Street SPS experienced three bypasses. Each of these occurred during severe inflow events such as rainfall and snow melts. The configuration of the sewer system includes a bypass manhole immediately upstream of the SPS which allows the system to bypass if the SPS cannot accommodate the flows directed to it. A 300 mm sewer existed between the bypass manhole and the SPS, the theoretical hydraulic capacity of this sewer was close to the capacity of the SPS. Given this, it is unclear if the previous bypasses were a result of the SPS not being able to convey the peak flows, or if the limiting capacity of the sewer connection between the bypass manhole and the SPS was causing backup into the bypass manhole. As part of sewer upgrades in 2010, the Township replaced the connecting inlet sewer between the bypass manhole and the SPS with a 375 mm pipe. Although it has only been one year, there has not been any bypasses since this upgrade was completed. Based on assumed peak flows and discussions with operators, this SPS does operate close to capacity during significant inflow events. However, normal wastewater flows are significantly less and the SPS has sufficient capacity. At this point, it is recommended that the bypass issue continue to be monitored until it can be confirmed if a SPS bypass issue actually exists or if recent upgrades have addressed this issue.

If in fact the SPS continues to bypass despite the recent inlet improvements, there are two options available to address this issue; reduce the amount of inflow to the system, or provide equalization storage to contain extreme peak flows. As indicated, the 1993 infiltration/inflow investigation and subsequent remedial work reduced I/I flow from the sewer system itself, however, inflow entering the system through private services appears to be significant during wet conditions. It is assumed that much of this originates from foundation drains and possibly roof leaders. Eliminating these connections would be extremely difficult especially since the majority of residential lots do not have storm services to allow the footing drains to tie into. Ideally these inflow sources should be reduced in order preserve available capacity in the all component of the system including sewers , SPSs, WWTP and the lagoons. However,

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ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

PEAK FLOW / CAPACITY (%)

- ≤25%
- ≤50%
- ≤75%
- ≤100%

FIGURE 4.3

PEAK SEWAGE FLOWS (EXISTING CONDITIONS)

SCALE 1:10,000
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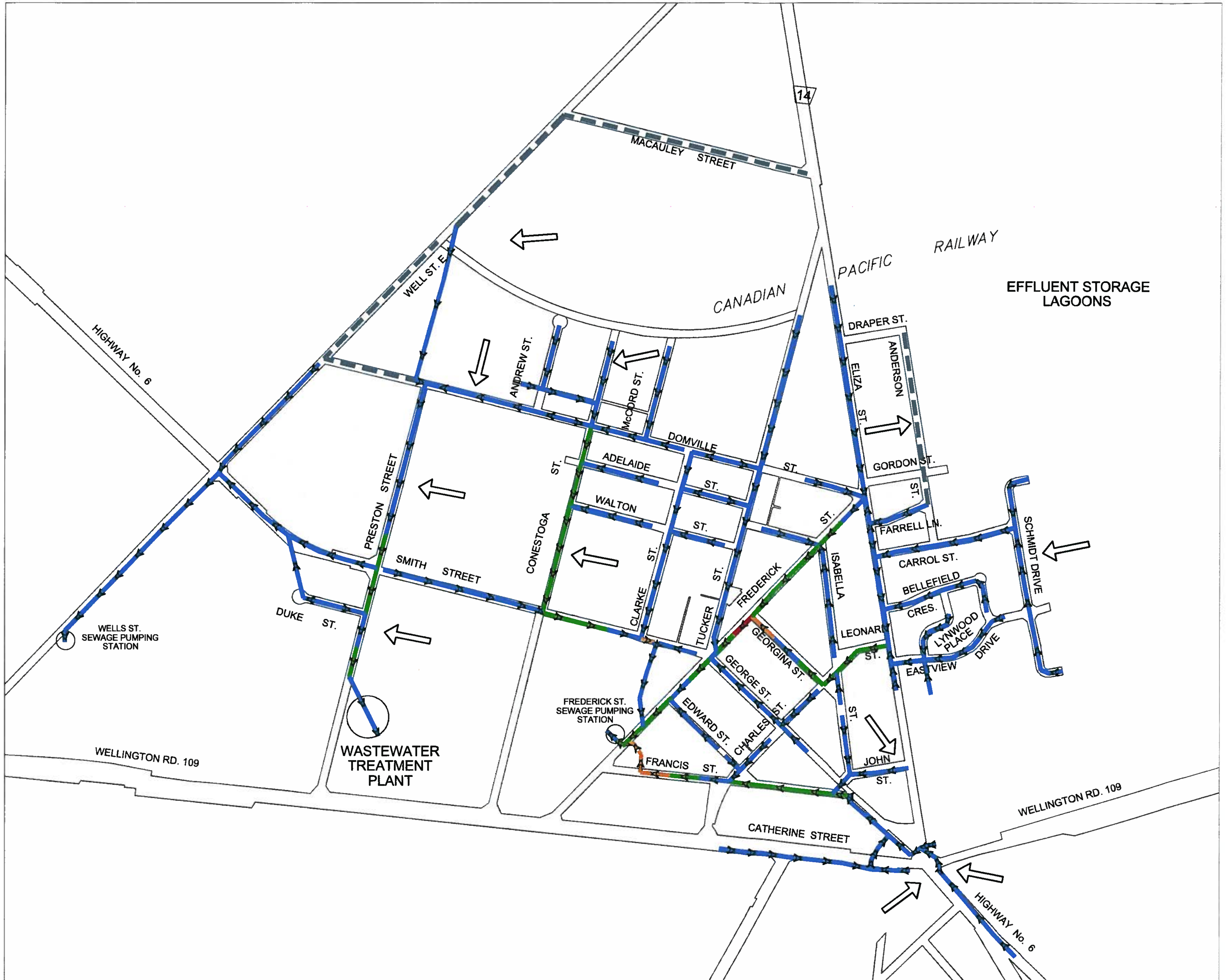

PEAK FLOW / CAPACITY (%)

- ≤25%
- ≤50%
- ≤75%
- ≤100%
- FUTURE SEWER EXTENSIONS
- FUTURE DEVELOPMENT CONTRIBUTION

FIGURE 4.4

PEAK SEWAGE FLOWS (2031 DEVELOPMENT)

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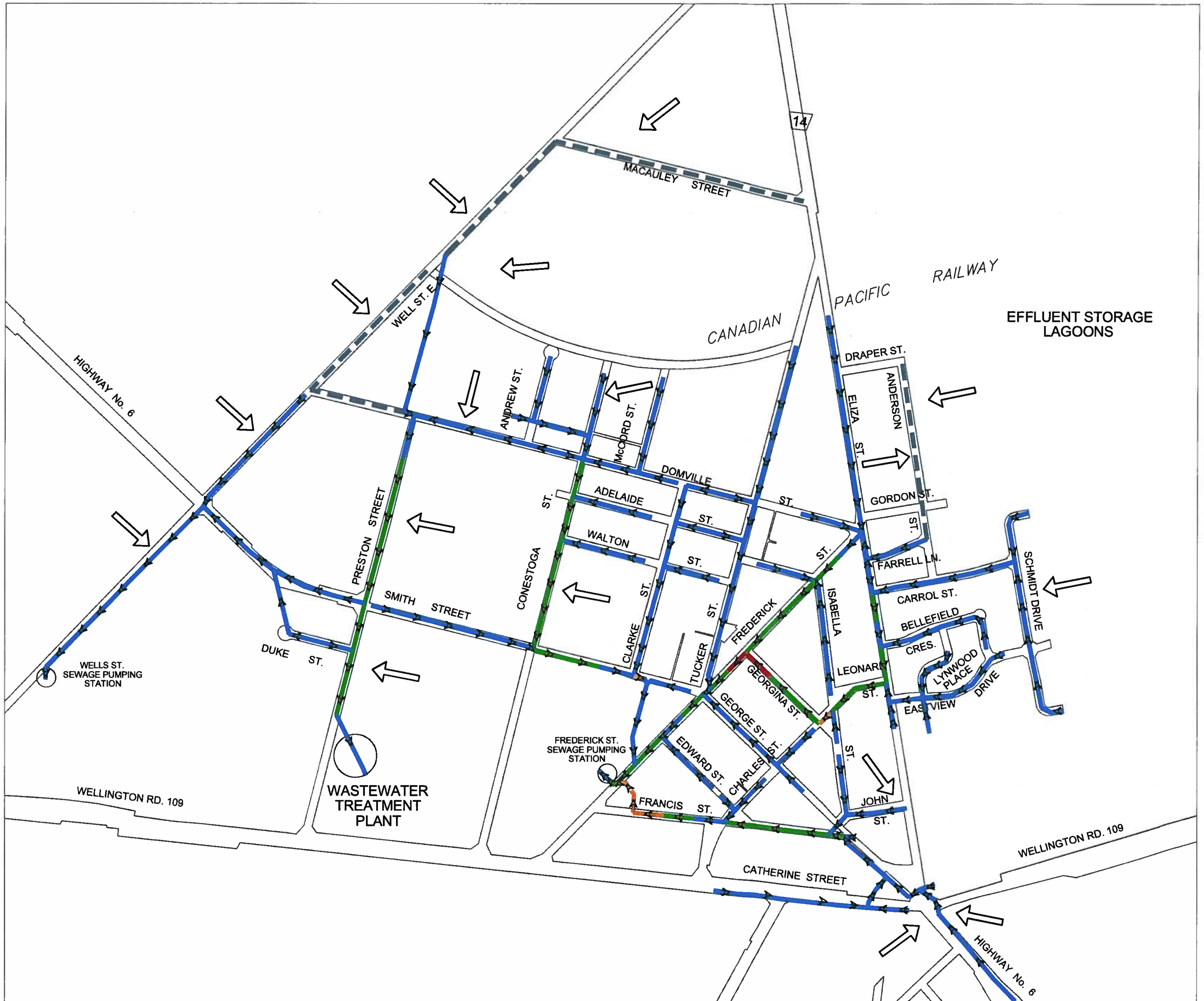

FIGURE 4.5

PEAK SEWAGE FLOWS (LONG-TERM DEVELOPMENT)

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ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

LEGEND

- EXISTING SEWER TO REMAIN
- UPGRADES
- EXTENSIONS

FIGURE 4.6

RECOMMENDED SANITARY UPGRADES AND EXTENSIONS

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recognizing that these inflows are difficult to reduce, the Township is considering providing equalization storage to contain bypass volumes until extreme peaks have subsided, at which time the stored volume would be conveyed to the WWTP at normal flow rates. Ideally, this equalization storage facility would be located next to the Frederick Street SPS so that the pumps and forcemain would not have to convey the extreme peak flows, rather they could pump the stored volume at their normal rate at off peak times. Unfortunately, the Frederick SPS site is constrained and accommodating such as facility will be difficult. Alternatively, the facility would be located at the WWTP site which has ample space to accommodate it. However, this would necessitate the upgrading of the SPS pumps and possibly the forcemain to accommodate the extreme peak flows. Detailed flow monitoring of inflow to the SPS and bypass volumes would be required in order that peak flows and storage requirements could be quantified, thereby, allowing for the preliminary design of an equalization facility, assessment of the Frederick SPS site to accommodate such a facility and costing of the system. Therefore, it is important that operators note the conditions under which bypassing occurs in order to support a preliminary design. Establishment of equalization storage would be considered as part of the Municipal Class EA for the WWTP.

4.4 Wastewater Treatment and Effluent Storage:

The Arthur Wastewater Treatment Plant (WWTP) is located at the south end of Preston Street near the Conestogo River. The plant provides tertiary treatment utilizing the extended aeration process. The treatment process components include:

- *Preliminary Treatment*: grit channels, comminutor, manual bar screen.
- *Secondary Treatment*: aeration, secondary clarifier.
- *Tertiary Treatment*: filtration, ultra-violet disinfection.
- *Biosolids Management*: aerobic digestion, biosolids storage.
- *Effluent Pumping Station*: two effluent pumps to transfer treated effluent from the WWTP to the lagoons during the non-discharge period.
- *Discharge*: to the Conestogo River during the discharge period, effluent is stored in the lagoons during the non-discharge period.

The rated average day flow (ADF) capacity of the plant is 1465 m^3/day. The Ministry of Environment Certificate of Approval limits the discharge period from September 16 to April 30 each year. The remainder of the year, effluent must be pumped to the effluent storage lagoons. During the discharge period, stored effluent is drained back to the WWTP for filtration and disinfection prior to discharge to the river.

The effluent storage lagoons are located in the northeast region of Arthur as shown on Figure 4.1. Effluent is pumped to storage from the WWTP through a 2.5 km long, 200/250 mm diameter forcemain. Flows are returned to the plant via gravity flow through the same pipe. Portions of the original 200 mm AC forcemain have been replaced with a 250 mm pipe in conjunction with road reconstruction and sewer upgrade work in the past. Although the original forcemain currently operates adequately, higher operating velocities within the smaller pipe result in increased head losses and resulting pumping costs. This issue, along with the advanced age of the pipe, warrants it’s continued replacement in conjunction with other works. The remaining 200 mm forcemain is generally located between George Street and the lagoons. It is estimated that there is approximately 1460 m of 200 mm forcemain that would need to be replaced at a cost of $780/m for a total cost of $1,138,800. *(Based on 2010 construction costs and assuming that forcemain is installed as part of road works. Allowances for engineering, contingency and HST have been included.)*. As indicated this replacement would be completed in sections in conjunction with future road reconstruction work. Therefore, an appropriate cost based on forcemain length should be included in future capital works budgets where applicable.

The lagoon facility has a storage volume of 340,000 m^3 which provides 232 days of effluent storage based on the design ADF of 1465 m^3/day. The original outfall sewer from the lagoons to the Conestogo River currently acts as an emergency overflow.

Reserve Capacity Calculations (RCC) completed over the past few years, have shown that the Arthur WWTP is nearing capacity (see Appendix F). In 2007, stress testing of the WWTP was completed. This testing indicated that the WWTP could be re-rated to a revised ADF of 1800 m^3/day provided there would be no significant changes to the Certificate of Approval discharge criteria.

Currently, the Township is completing technical studies to support a Municipal Engineers Class Environmental Assessment to review viable options to increase reserve sewage treatment capacity in Arthur. Options to be considered may include: water conservation measures, infiltration and inflow control and re-rating of the WWTP. The Class EA process may identify other viable options.

Successful re-rating will increase the hydraulic capacity of the WWTP, which will address short term capacity concerns. However, the capacity requirements for the 2031 Scenario are estimated to be 1950 m^3/day based on 2011 RCC per capita flow of 0.522 m^3/day. This figure exceeds the proposed re-rated capacity of the WWTP, therefore, in order to service the 2031 Scenario, additional hydraulic capacity will be required unless existing and future flows can be reduced. Therefore, it is recommended that the Township actively pursue a program to reduce the wastewater flows to the extent feasible. In addition, it will be important that the municipality consider the types of future industries that locate within the service area. If a number of “wet” industries are permitted to connect to the system, the available reserve capacity will continue to decrease.

5.0 SUMMARY

5.1 General

- The assessments and recommendations provided in this Master Plan are intended to assist the Township in future infrastructure and development planning. This document was prepared in accordance with the MEA Class Environment Assessment (Class EA) process and was based on available information regarding existing infrastructure, land-use, topography, water use, wastewater flows and future development planning. However, this information is continually changing, therefore, it is critical that as development proposals and/or infrastructure upgrades are considered, that the background information used in the assessment is reviewed to confirm that it is still applicable. The Class EA recommends that the Master Plan is to be reviewed every five years to determine the need for a detailed formal review and/or updating.

- This Master Plan considered two development planning periods, the 2031 Scenario and the Long-Term Scenario. The 2031 Scenario has been established using planning information provided by Wellington County Planning and Development Department from report entitled “Comprehensive Review of Residential and Employment Growth, Township of Wellington North” dated October 14, 2009 and has a 20 year planning horizon. This scenario is used to identify infrastructure requirements needed to support the specific development identified in this planning document. The Long-Term Scenario was included in the servicing review in order identify infrastructure servicing requirements beyond the 2031 Scenario, particularly in areas where servicing extensions are being considered. Although this scenario does not reflect a specific planning horizon, the additional areas included beyond the 2031 Scenario are considered as potential development areas that may affect design requirements of short-term infrastructure upgrades considered in this study.

5.2 Water System

- Source Capacity is a measure of the municipal water systems’ ability to supply water. Based on current information, the Source Capacity of the Arthur water system is sufficient to meet the estimated water demands of the 2031 Scenario. As indicated earlier, is it imperative that the information used to arrive at this conclusion be reviewed periodically to ensure that conditions (i.e., well production rates, water usage) have not changed from those used in the analysis. It is recommended that annual Water Supply Reserve Capacity Calculations be completed in order that current usage and future needs can be monitored regularly. Despite the current surplus of source capacity, it is crucial that the Township be proactive in securing future water sources since establishing these new sources, and the infrastructure required to deliver this water to the system, can be a lengthy, arduous process.

- Water storage within the system is provided for three purposes; fire storage to allow the system to achieve flow rates and volume necessary to effectively fight fires; equalization storage which provides water to the system during peak demand periods; and emergency storage intended to provide a safety factor for water supply. Based on the Ministry of Environment (MOE) guidelines, the existing storage is adequate for the current development and given the surplus in source capacity which can supplement storage requirements, the storage should be adequate for the immediate future. In considering the 2031 Scenario, the storage requirement is significantly above the current volume available and the surplus source capacity is diminished. Therefore, additional storage should be added to the system prior to the 2031. Given the age and limited volume of the existing facilities, and the fact that a tower with a higher operating level is needed to service outlying areas, consideration is to be given to constructing a new elevated tower that would replace the storage volume in the existing tower and provide additional storage for future development. This tower would also operate at a higher level in order to service outlying areas and improve fire flow capabilities of the existing system. Although the existing Freud Tower site may be suitable for the new tower, the Township should consider obtaining a site for this future tower as part of future development proposals in the industrial area. This would provide a higher base elevation (i.e., shorter tower) and allow for adequate system storage during construction of the new tower. Establishment of a new water storage facility and the decommission of the existing facilities are both considered Schedule B projects under the Class EA.

- A computer model of the water system was created as part of this study. Hydraulic analysis using this model indicates that system pressures and fire flow capabilities are adequate to meet existing and 2031 Scenario needs. However, the potential long term future development areas to the north, east and south of the existing urban area are located at a higher topographic elevations. Given this, it is recommended that consideration be given to increasing the operating range of a future elevated tower to adequately service these areas. This would have the added benefit of increasing system pressures and fire flows through the entire service area.

- Generally, the watermain within the existing system are in good condition, however, there is a moderate amount of cast iron pipe remaining is the system. This pipe is reaching the end of this service life and should be replaced as part of road reconstruction projects.

- Dead end mains are a concern from both a water quality perspective and service reliability standpoint. The Arthur distribution network is well looped with few dead-ends, those that do exist (i.e., Preston Street South, Duke Street) are situated such that looping options are not readily available. Servicing strategies for future developments should consider this issue and design watermain extensions to eliminate existing dead ends where possible and ensure adequate circulation in the system.

- Given the future expansion of the industrial area outlined in the development scenarios, it is recommended that watermain trunks be extended to service this area. It is recommended that these trunks would be extended on Wells Street East to Macauley Street, along Macauley Street to Eliza Street and on Eliza Street to complete the loop as illustrated in Figure 3.5. However, the optimum routing for these trunks may depend on the configuration of the developments to a certain extent and should be reviewed in the context of future development proposals.

5.3 Sanitary Collection System

- As part of this Master Plan, a computer simulation model of the sanitary collection system was prepared to estimate flows throughout the network and compare them to the hydraulic capacity of the various sewers and forcemains. Based on this analysis, the collection system has sufficient hydraulic capacity under existing development conditions. However, under the 2031 and Long-term Scenarios, hydraulic capacity in some sewers may be exceeded depending on the routing of future development. Specifically, servicing of the industrial development area 217 (between CPR and Macauley Street) and long-term areas to the north and west, should be directed to the Preston Street trunk sewer to the extent possible to ensure that available capacity is preserved in local sewers for other development areas which cannot access this trunk. This strategy will also reduce loading on both the Wells and Frederick Streets SPS. Figure 4.6 illustrates possible sewer extensions to service the future industrial development areas utilizing existing right-of-ways.

However, depending of the configuration of the developments, these sewers may be extended internal to the development areas. The optimum sewer configuration should be considered as development proposals are brought forward.

- Based on the modelling, the remainder of the existing collection system generally has sufficient available capacity to accommodate the assumed design flows generated by the 2031 and Long-term Scenarios. The model did identify a couple of areas where theoretical flows were approaching sewer capacity under future development conditions. Sewers recommended for upgrading include Frederick Street between Georgina Street and George Street, and on Georgina Street immediately upstream of Frederick Street to maintenance hole MH120. Actual flows at these locations should be confirmed with local flow monitoring in the future prior to build-out of the 2031 scenario or replacement of sewers.

- Using information sources currently available, it is estimated that the infiltration and inflow to the system is emanating primarily from services connected to the system, not the sewers themselves. In order to determine the location and extent of this extraneous flow, and to improve calibration of the sewer model, an extensive monitoring and investigation program would be required. It is recognized that such a program can be very expensive and therefore may not be feasible. However, as a minimum, local monitoring of specific areas of concern should be undertaken prior to committing to major developments, or to infrastructure upgrades to confirm necessity and design requirements.

- Based on the design flows and the recommended servicing strategy (i.e., utilization of the Preston Street Trunk Sewer), the Wells and Frederick Streets sewage pumping stations, and associated forcemains are adequate to service the future development scenarios considered in this study. However, it is noted that during extreme inflow events, there has been bypassing at the Frederick SPS bypass manhole. It is unclear if these bypasses are a result of insufficient pumping capacity at the SPS, or hydraulic limitations of the inlet sewer. Recent upgrades to the inlet sewer were completed in 2010 and no bypasses have occurred since, however, it is too early to conclude that this upgrade has addressed the issue. If in fact the bypasses continue, there are two options available to address this bypass issue; reduce the amount of inflow to a level that the SPS can accommodate, and/or provide equalization storage to contain extreme peak flows. Recognizing that it may be difficult to reduce inflows to a level that will eliminate bypasses, equalization storage could be to provided to reduce the incidence and severity of the bypasses. Detailed flow monitoring of the SPS and bypass volumes would be required in order that peak flows and storage requirements could be quantified. Thereby, allowing for the preliminary design of an equalization facility, assessment of the Frederick SPS site to accommodate such a facility and costing of the system. Establishment of equalization storage would be considered as part of the Municipal Class EA for the WWTP.

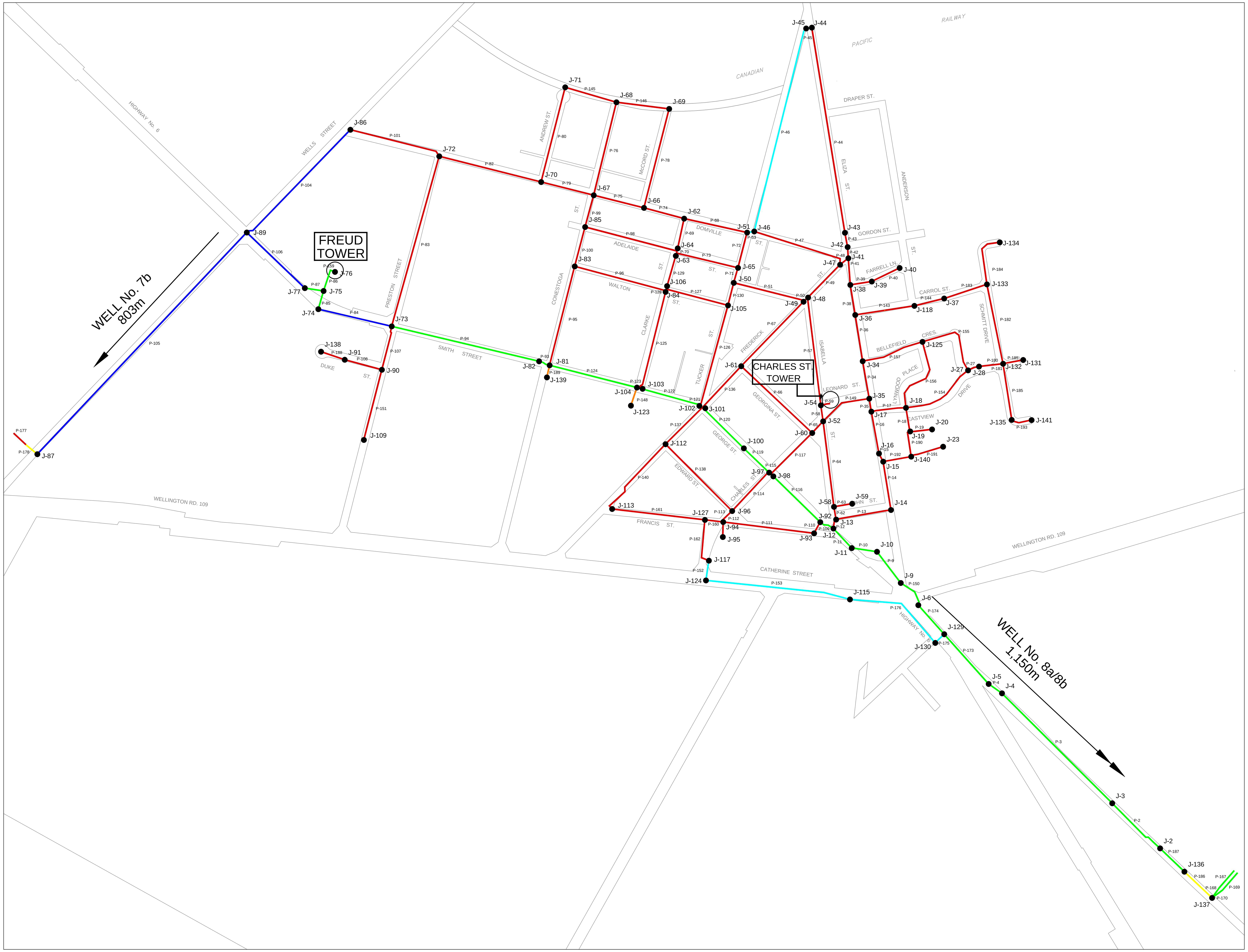
- Effluent is pumped to the effluent storage lagoons from the WWTP through a 2.5 km long, 200/250 mm diameter forcemain. Flows are returned to the plant via gravity flow through the same pipe. Portions of the original 200 mm AC forcemain have been replaced in conjunction with road reconstruction and sewer upgrade work in the past. Although the original forcemain does currently operate adequately, the smaller size and age to this portion of the forcemain warrant it's continued replacement in conjunction with road works along it's length.

- Based on 2011 Reserve Capacity Calculations (RCC), the ADF at the WWTP is 1338 m^3/day . With the committed development reflected in the RCC, there is a negative Uncommitted Reserve Capacity indicating the WWTP is nearing capacity. In response, the Township has undertaken an Optimization Study of the WWTP and concluded that the existing facility could be re-rated from its current capacity of 1465 m^3/day to approximately 1800 m^3/day. Although the re-rating will increase the capacity of the WWTP, which will address short term capacity concerns, the capacity requirements of the 2031 Scenario are estimated to be 1950 m^3/day based on 2011 RCC per capita flows. This figure exceeds the proposed re-rated capacity, therefore, it will likely be necessary to expand the WWTP to service the future development scenario unless existing and future flows can be reduced. In order to maximize the development serviced by the WWTP, it is recommended that the Township actively pursue a program to reduce the sewage flows to the extent feasible. In addition, it will be important that the municipality consider the types of future industries that locate within the service area. If a number of "wet" industries are permitted to connect to the system, the flows would be increased substantially.

This report is respectfully submitted.

TRITON ENGINEERING SERVICES LIMITED

R. D. Kirtz, P. Eng.



TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

WATERMAIN SIZING:

- ≤100mm
- 150mm
- 200mm
- 250mm
- 300mm
- ≤600mm

MODEL REFERENCE:

NODE ● J-100

PIPE ID — P-101

DRAWING - W1

WATER SYSTEM MODEL REFERENCE

SCALE 1:5,000
A5514

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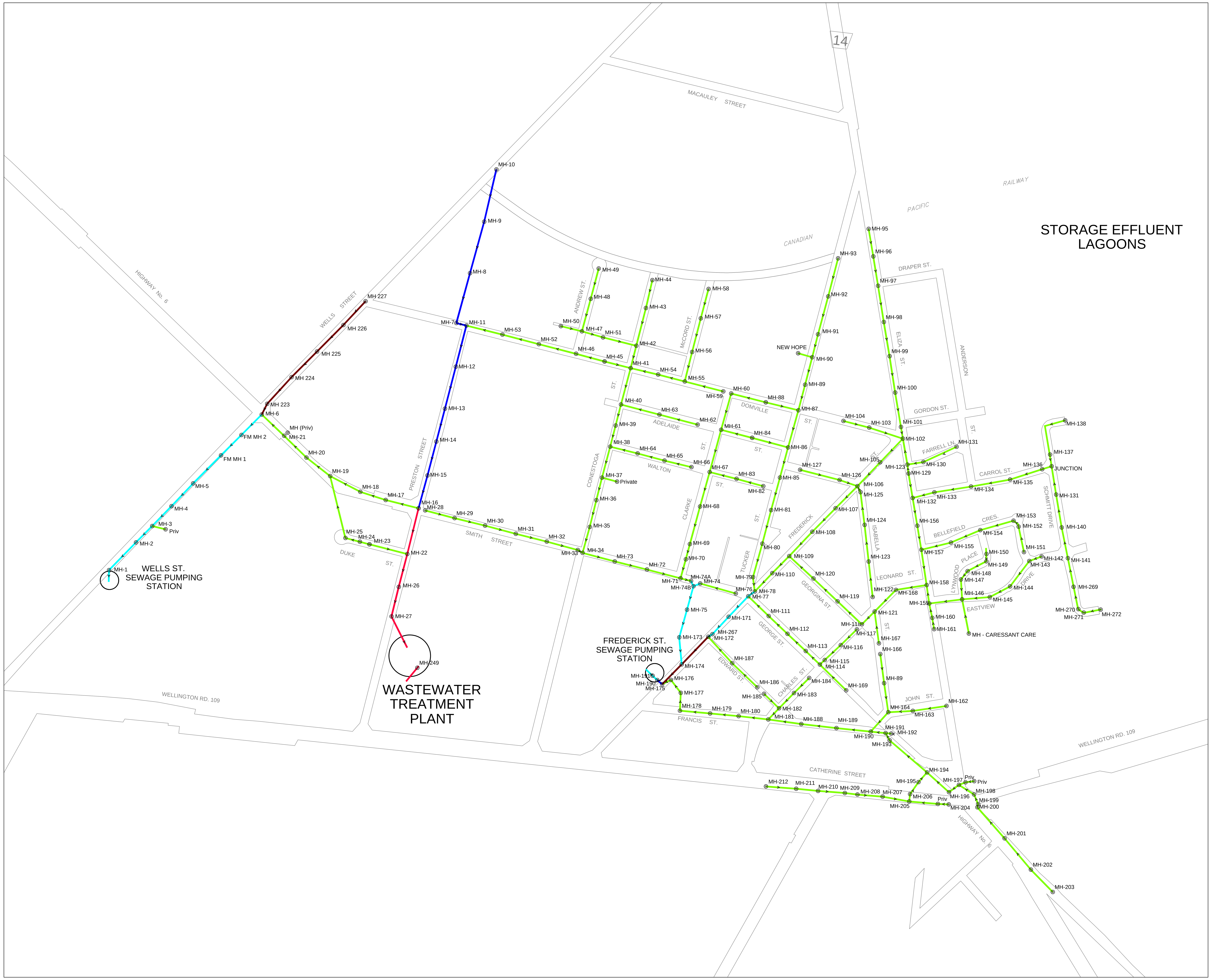
Consulting Engineers
Fergus, Ontario



TOWNSHIP OF WELLINGTON NORTH

ARTHUR WATER AND SANITARY SERVICING MASTER PLAN

SEWER SIZING:

200mm

250mm

300mm

375mm

450mm

MODEL REFERENCE:

MANHOLE ID MH-121

FLOW ARROW

DRAWING - S1

SEWER SYSTEM MODEL REFERENCE

SCALE 1:5,000
A5514

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Fergus, Ontario