FUNCTIONAL SERVICING REPORT
PORTA

July 18, 2025

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1 BACKGROUND

Harbour 25 LP (Porta) engaged Jewell Engineering Inc. (Jewell) to complete a servicing review in support of a rezoning application for the subject property at 25 Dundas Street West in Belleville, see Figure 1-1.

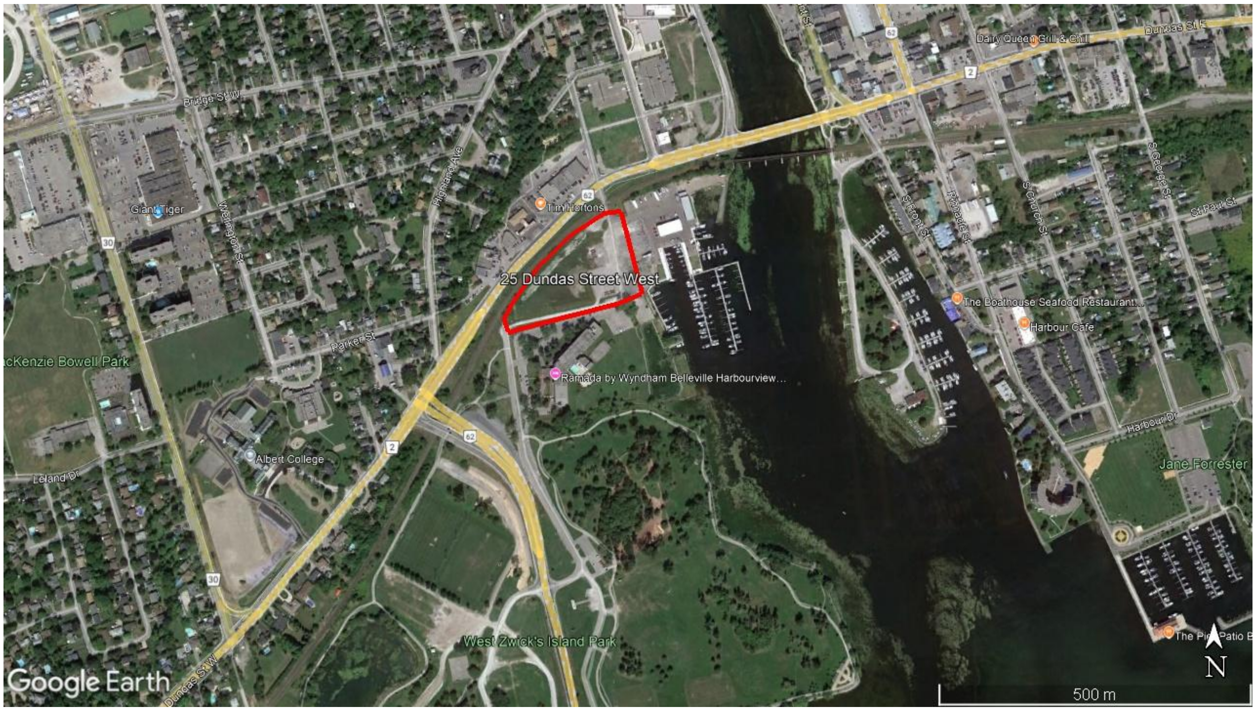

Figure 1-1: Property Location

The following services have been considered for the development:

- Water Distribution System
- Sanitary Sewer System
- Storm Sewer System
- Stormwater Management System (separate cover)
- Traffic Impact Study (separate cover)

1.1 SITE DESCRIPTION

The development area is approximately 2.94 ha. The surrounding land use is commercial to the south, marine commercial to the east and railway to the north.

The site is a former brownfield site and a risk assessment was completed by Blumetric. Blumetric found contamination within the soil but that the groundwater was not contaminated. This meant that the contaminants are not being mobilized into the water. The remediation plan includes both hard and soft caps consisting of a minimum of 75 mm asphalt over 150 mm of gravel or 1 m of clean fill, respectively. All piping will be bedded with clean material and will be separated from contaminated materials with a geotextile. What this means is that the pipe materials will not come into contact with contamination from either soil or groundwater; therefore, no special protective materials or installation practices are required.

The Ministry of the Environment has issued 2 Certificates of Property Use (CPU) for the site: RA1470-15-01 and RA1470-15-02. The certificates are based on the Risk Assessment RA170-15e prepared by Blumetric. The CPUs contains the full approval details for how the property is to be managed. In support of the CPUs, the Risk Assessors issued a Health and Safety Plan. The current version will be on file with the site owners. Blumetric, at the City's request, also provided a summary letter with direct answers to the City's environmental concerns. This letter is included in Appendix B.

1.2 Proposed Development

The proposed development will consist of 213 residential units and a few commercial units, see Figure 1-2. The development will include the construction of private roads with access from Old Bay Bridge Road. See Appendix A for the full plan.

Figure 1-2: Porta Concept Plan

1.3 SERVICING INFRASTRUCTURE

1.3.1 WATERMAIN

There is a 450 mm diameter watermain on Dundas Street West that supplies a 200 mm diameter watermain which services the subject lands and Crate Marine. The 200 mm main is situated along the Mary Street road allowance. There is also a 200 mm diameter watermain on Old Bay Bridge Road servicing the Ramada Inn. This main connects to Dundas Street West. Each of the two mains dead-ends presenting potential for water aging concerns.

The proposed 200 mm diameter watermain loop provides an opportunity to correct the dead-ends by connecting both dead-ends on Old Bay Bridge Road and Mary Street. The City of Belleville requested this loop and would take ownership of the main through a servicing easement.

1.3.2 SANITARY SEWER

The lands are low and cannot drain by gravity to the City sewer. It will require a lift station to discharge to the sanitary main along Dundas Street West. Currently, a lift station is located on the Crate Marine property that also served the subject lands. According to the 2014 Servicing Report by Towles and City drawings, the 100 mm forcemain from the lift station discharges to a 900mm sanitary main at the intersection of Dundas Street and James Street. The forcemain previously connected to the 600mm sewer at the intersection of Mary Street but was rerouted in the late 1980s.

Towles noted the most recent use of the lands by the Harbour Club and some residential dwellings would have generated a peak flow of some **10.45 L/s**. This would have represented 0.81% of the capacity of the 900 mm sewer.

The servicing strategy will be to install a new lift station and intercept this 100 mm forcemain to avoid any work within the CP Rail limits. The current lift station for Crate Marine will be routed to discharge to the new lift station.

A new 200 mm diameter gravity sewer will collect sewage and drain to the new lift station.

2 WATER DISTRIBUTION SYSTEM

A new watermain is proposed to connect from Mary Street through the site to Old Bay Bridge Road. The proposed watermain will be owned and maintained by the City of Belleville serving as a loop between the Mary Street and Old Bay Bridge Road mains. An easement over the main will be provided to the City of Belleville for future maintenance. Certificates of Property Use remain the responsibility of the Site Owners who are bound by the obligations within the certificates.

2.1 EXISTING CONDITIONS

There is an existing 200 mm water service connected to the existing marine buildings. There is also a 200 mm watermain on Old Bay Bridge Road. Both watermains are connected to the 450 mm watermain on Dundas Street.

2.2 DESIGN CRITERIA

The watermain design criteria used are based on the City and MECP guidelines are summarized below:

- Minimum Watermain Diameter Size: 200 mm
- Average Daily Demand:
  - Residential: 350 L/d*cap
  - Commercial: 2.5 L/d*m^2
  - Heavy Industry: 55,000 L/d*ha
- Maximum Day plus Fire Flow Demand Pressure Minimum: 20 psi
- Peak Hour Demand Pressure Minimum: 50 psi
- Peak Hour Demand Pressure Maximum: 80 psi
- Maximum Pressure: 100 psi
- Test Pressure: 200 psi
- Maximum Velocity: 3.0 m/s
- Friction Factor: 110
- Minimum Depth of Watermain: 1.8 m
- Maximum Depth of Watermain: 2.5 m
- Minimum Horizontal Separation: 3.0 m
- Minimum Vertical Separation: 0.5 m
- Fire Hydrant Spacing: 90 m

2.3 Fire Flow Requirements

The required fire flow for the proposed development was calculated using the method in the Water Supply for Public Fire Protection, 2020, published by the Fire Underwriters Survey (FUS).

The maximum required fire flow was determined to be 9,350 L/min based on the FUS calculation, see Table 2-1. The existing marine buildings are more than 45 m away and therefore, are not included in the fire flow calculations. The area used in the calculation is the largest area allowed without fire walls; however, since fire walls are expected to be between every 2-unit block, the fire flow calculated is very conservative.

Table 2-1: FUS Fire Flow Calculation Procedure

Fire Underwriters Survey Method*
Fire Flow - Porta
*Water Supply for Public Fire Protection - A Guide to Recommended Practice in Canada - 2020

A	Construction Type	Wood Frame		Type V	
	Constuction Coefficient		C =	1.5	
	Vertical Openings	use if C < 1.0		Unprotected	
B	Total Effective Floor Area	Ground Floor Area		600	sq.m
		Height (# of Storeys)		2	storeys
			A =	1,200	sq.m
C	Required Fire Flow	$RFF = 220C\sqrt{A}$		11,432	L/min
		Rounded to 1000 L/min		11,000	L/min
D	Occupancy and Contents Adjustment	Limited Combustible		-15%	
		Decrease by		-1,650	L/min
E	Automatic Sprinkler Adjustment	N/A		0%	
		Decrease by		0	L/min
F	Exposure Adjustment	Exposure Charge		0%	
		Decrease by		0	L/min
G	Adjusted Required Fire Flow	Calculated		9,350	L/min
		Converted		156	L/s
		Converted		2,470	USGPM

2.4 Population Demand

The daily average demand of the development was established by calculating the proposed domestic and commercial and the existing industrial demands, see Table 2-2.

Table 2-2: Daily Average Demands

Type	Residential			**Type**	Commercial	
# of Units	213			**Total Floor Area**	300	m^2
Population/unit	2.5			**Capita Usage**	2.5	$L/d*m^2$
Population	532.5			**Demand**	750	L/d
Capita Usage	350	L/d*cap				
Demand	186,375	L/d		**Type**	Industrial	
				Total Floor Area	0.1285	ha
				Capita Usage	55,000	L/d*ha
Total Average	**194,193**	**L/d**		**Demand**	7,068	L/d

Once the daily average demands were established, peaking factors were determined from Table 3-1 in the MECP design guidelines, as shown in Table 2-3.

Table 2-3:Peaking Factors, Table 3-1 MECP Guidelines

Peaking Factors	
Peak Hour	4.13
Maximum Day	2.75

2.5 Watermain Hydraulics

Jewell utilized Bentley WaterCAD water modelling software to analyse the proposed watermain loop. To create the model, Jewell simulated the pressure and flows available at the two proposed connection points with a reservoir, pump, and pump curve. The reservoir (set to the road elevation) provides an unlimited supply of water to the system. The pump pressurizes the proposed system within the operating range set by the pump curve. To create the pump curve, Jewell used the fire hydrant test completed by the City on March 26, 2007. The calculations for the pump curve can be found in Appendix E, along with the reservoir and pipe inputs.

To determine the available flows and pressures in proximity to the connection points on Old Bay Bridge Road and Mary Street, Jewell obtained the fire hydrant flow test for a hydrant on Dundas Street West directly across from the development. The City indicated the flow test on March 26, 2007, was the most current and remained applicable. Jewell was informed that for the purpose of this study, the flow test completed in 2007 is still relevant. A summary of the results is listed in Table 2-4 with the full results attached in Appendix D. The flow test results were used to calibrate the water model and simulate the existing network system.

Table 2-4: Fire Hydrant Test Results, March 26, 2007

Hydrant Location	Main Size (mm)	Static Pressure (psi)	Residual Pressure (psi)	Residual Flow (usgpm)
48 Dundas St W	450	84	78	1,898

Three scenarios were calculated: Static Pressure, Peak Hour, and Max Day + Fire Flow. These scenarios calculate the pressures available during no demand and standard high demand periods. The results are summarized in Table 2-5.

Table 2-5: Water Model Results

Water Model Results										
Label	Elevation	Static Pressure	Peak Hour		Max Day		Fire Flow	Max Day + Fire Flow		Fire Flow Available
		No Demand	Demand	Pressure	Demand	Pressure	Needed	Demand	Pressure	@ 20 psi
	(m)	(psi)	(L/min)	(psi)	(L/min)	(psi)	(L/min)	(L/min)	(psi)	(L/min)
J-1	76.7	84.4	0.0	84.3	0.0	84.4	12,000	12,000.0	69.8	27,008
J-2	76.7	84.4	10.0	84.3	6.7	84.4	12,000	12,006.7	68.3	25,659
J-3	76.0	85.4	50.2	85.3	33.4	85.4	12,000	12,033.4	68.5	25,217
J-4	76.0	85.4	25.1	85.3	16.7	85.3	12,000	12,016.7	67.9	24,766
J-5	76.0	85.4	50.2	85.3	33.4	85.3	12,000	12,033.4	67.6	24,582
J-6	76.0	85.4	65.3	85.3	43.5	85.3	12,000	12,043.5	67.3	24,360
J-7	76.0	85.4	40.2	85.3	26.7	85.3	12,000	12,026.7	67.0	24,109
J-8	76.0	85.4	55.2	85.3	36.8	85.3	12,000	12,036.8	65.7	23,224
J-9	76.5	84.7	19.7	84.6	13.1	84.6	12,000	12,013.1	68.6	25,811
J-10	76.5	84.7	60.2	84.6	40.1	84.6	12,000	12,040.1	67.7	25,066
J-11	76.5	84.7	55.2	84.6	36.8	84.6	12,000	12,036.8	67.1	24,577
J-12	76.5	84.7	70.3	84.6	46.8	84.6	12,000	12,046.8	66.9	24,414
J-13	76.5	84.7	35.1	84.6	23.4	84.6	12,000	12,023.4	67.1	24,527
J-14	76.8	84.2	0.0	84.2	0.0	84.2	12,000	12,000.0	69.5	26,905
J-15	76.2	85.1	20.3	85.1	13.5	85.1	12,000	12,013.5	41.2	14,866

Jewell Engineering Inc. Phone: 613-969-1111 Website: www.jewelleng.ca	**Designed:** Julie Humphries **Checked:** Bryon Keene, P.Eng. **Date:** June 18, 2025	**Project:** Porta

The pressures are lowest during the Max Day + Fire Flow scenario. This occurs when a fire flow demand has been applied to the system and maximum day domestic usage conditions are considered.

The MECP specifies that watermain pressures during Static and Peak Hour scenarios are to be between 50 psi to 80 psi; however, if they are greater than 80 psi, they cannot exceed 100 psi. The pressure during Static and Peak Hour scenarios do exceed 85 psi, but do not exceed 100 psi; therefore, the watermain meets the MECP specifications.

According to MECP specifications, a minimum system pressure of 20 psi is required to be maintained during fire fighting. The available fire flow along the looped main does not drop below 14,866 L/min. Therefore, the watermain provides sufficient fire flow.

The scenario results show that there is sufficient flow and pressure available in the existing system to accommodate the additional demand created by the proposed development.

Figure 2-1 shows the proposed layout of the watermain. For an enlarged map, see Appendix F.

Figure 2-1: Porta Watermain Layout

2.6 VELOCITY AND TRANSIENT FLOW

2.6.1 VELOCITY CHECK

Maximum acceptable velocity from one direction is 3.0 m/s. Velocity may be calculated using the continuity equation ($Q = VA$), given the proposed pipe size of 200 mm and peak flow assuming full fire flow and peak demand. Also, since the flow of water is sourced from two directions, the calculated velocity is divided in half. Jewell calculated the peak flow to be 557 L/min or 0.008 m^3/s. Velocity is 2.7 m/s. This is less than 3.0 m/s and is acceptable. See below for full calculations.

Flow Calculation

$$Q = (fire\ flow) + (average\ demand * peak\ hour)$$
$$Q = 0.16\, m^3/s + (0.002\, m^3/s * 4.13)$$
$$Q = 0.17\, m^3/s$$

Velocity Calculation

$$V = \frac{Q}{2A} = \frac{0.17\, m^3/s}{2(0.0314m^2)}$$
$$V = 2.7\, m/s$$

Area Calculation

$$A = \pi r^2$$
$$A = \pi(0.1m)^2$$
$$A = 0.0314m^2$$

Conclusion

$$0.6\, m/s < V < 3.0\, m/s$$
$$V = m/s < 3.0\, m/s$$
$$\therefore velocity\ is\ acceptable$$

2.6.2 Transient Flow Check

The transient flows are checked assuming a column of water flowing at 0.6 m/s is abruptly stopped. Transient flows are estimated using the water hammer equation:

$$P_{additional} = \frac{aV}{2.31g}$$

Where:

$a = speed\ of\ pressure\ wave = 4860\ ft/s$
$V = velocity\ in\ pipe\ (ft/s)$
$g = acceleration\ due\ to\ gravity = 32\ ft/s^2$

Given that velocity equals 0.6 m/s (1.97 ft/s), the additional pressure equals 129.5 psi. Total pressure is the additional pressure due to water hammer plus the static pressure, which is 213.5 psi. This is less than the maximum rated pressure of the DR18 pipe, which is 235 psi. Therefore, the 200 mm DR18 pipes are sufficient for the application. See below for full calculations.

Additional Pressure

$$P_{additional} = \frac{aV}{2.31g}$$
$$P_{additional} = \frac{(4860\ ft/s)(1.97\ ft/s)}{2.31(32\ ft/s^2)}$$
$$P_{additional} = 129.5psi$$

Total Pressure

$$P_{total} = P_{additional} + P_{static}$$
$$P_{total} = 129.5psi + 84psi$$
$$P_{total} = 213.5psi$$

Conclusion

$$P_{total} < 235psi\ (maximum\ rated\ pressure, DR18\ pipe)$$
$$P_{total} = 213.5psi < 235psi$$
$$\therefore 200mm\ DR18\ pipes\ are\ sufficient$$

Restraints must be provided per the manufacturers and the City's specifications.

2.7 Watermain Materials and Fittings

The City of Belleville standard watermain notes are included on the engineering drawings and will be carried through to the Approved for Construction drawings. A summary of item specifications is listed below.

2.7.1 Pipe Material

All watermain pipe 100 mm to 300 mm in diameter shall be DR18 PVC (or lower) and be manufactured in accordance with AWWA C900 and certified to NSF/ANSI 61 and CSA B137.3.

The pressure class of all pipes shall be a minimum of 235 psi.

2.7.2 Water Services

Water services should be installed at locations shown on the engineering drawings. They shall terminate 0.15 m outside the property line within the right-of-way. The service is to be controlled by a curb stop that shall be installed a minimum of 500 mm away from the driveway location. All water services conform to the City standards.

As per the City of Belleville standards, all 2-unit dwelling units will have a 25 mm service.

2.7.3 Fire Hydrants

Hydrants should be installed at locations agreed through consultation with the City during the review process. The City of Belleville standard for fire hydrant spacing requires no greater separation between hydrants than 90 m for towns and multi-units.

Hydrants shall conform to AWWA Standard C502: Dry Barrel Fire Hydrants.

If the drain hole is within or below the groundwater table, the hole is to be plugged. High groundwater table is expected at the development; therefore, the holes will be plugged.

Hydrant locations are adjusted where possible to be situated at high points in the main to help provide air release points.

2.7.4 Valves

Valves shall be installed at each intersection (2 at a ‘T’, 3 at a ‘X’) and at minimum separations as requested by the City during detailed design. This standard has been applied. Maximum valve spacing permitted by the City is 300 m. Valves do not exceed this standard.

All Valves conform to AWWA standards.

2.7.5 Chambers

There are no chambers proposed in this development.

2.7.6 Depth

All watermain shall be a minimum of 1.8 m in depth. Watermains will be placed 1.8 m below top of road.

2.7.7 Dead Ends

All locations where a watermain terminates (temporary or permanent) a plug and blow off shall be installed.

No watermain dead ends are proposed. All mains will be looped.

2.7.8 Restraints

All joints (at fittings, hydrants, valves, and bends greater than 11.25°) shall be mechanically restrained. The City Standard 1110.05.09 for specific instructions on placement of restraints is referenced. Since all pipes are 300 mm or smaller, mechanical restraints are required on all joints as follows:

Table 2-6: Restraint Distance Requirements at Pipe Fittings

Fitting	Restraint Distance (m)
Bend 11.25 degrees or larger	15
Cap	20
Plug	20
Valve	20
T (middle leg)	15
T (main legs)	Immediate joint (full length of pipe is required)
Hydrant Lead	Full Length

2.7.9 Nitrile Gasket Seals

Nitrile gasket seals shall be used for watermains buried in soil with or with the potential for hydrocarbon contamination. Nitrile gasket seals shall conform to AWWA standards.

There is known soil contamination on the subject lands and nitrile gasket seals are proposed.

3 SANITARY SEWER SYSTEM

The sanitary sewer system for the development will be constructed as a new private gravity system to a private pump station. The sewage will contribute to the Belleville Sewage Collection System via the existing forcemain. No new discharges to the natural environment are created by the installation.

Due to the high groundwater table conditions, care should be taken during construction such that all connections are well sealed. A product such as Riser Wrap will be placed around all manhole joints. Pipe connections to the manholes will be made using boot connections.

The project is not subject to Section 16 of the EAA.

Sanitary sewer installation must follow all applicable OPSS's, such as OPSS 410 and OPSS 407, OPSD's and City standards.

3.1 Existing Conditions

There is an existing 900 mm diameter sanitary main on Dundas Street West that receives a 100 mm diameter forcemain from Crates Marine.

3.2 Design Criteria

The sanitary design criteria used are based on the City of Belleville ECA design criteria, engineering standards, MECP guidelines, and MECP F-6-1, which are summarized below.

- Minimum Sewer Diameter: 200 mm
- Pipe Capacity Equation: Manning's
- Minimum Roughness Coefficient (Manning's n): 0.013
- Full Flow Velocity:
  - Minimum: 0.6 m/s
  - Maximum: 3.0 m/s
- Extraneous Flow Allowance: 0.28 L/s*ha
- Average Daily Residential Flow: 350 L/d*cap
- Population Factors:
  - Townhome: 2.5 persons/unit
  - Apartment: 2.5 persons/unit
- Peak Flows:
  - Commercial: 2.5 L/d*m^2
- Residential Peak Factor:

- Calculation: Harmon
- Minimum: 2.00

- Maximum Pipe Usage: 80%
- Horizontal Separation from Watermain: 2.5 m (minimum)
- Vertical Separation from Watermain: 0.5 m (minimum)

3.3 Sanitary Sewer Design

Pipes are sized using the standard Sanitary Sewer Design Sheet, see Table 3-1 or Appendix F. Peak flows of **10.06 L/s** are expected for both Crates and the subject lands. This is similar to the flows that were calculated by Towles (10.45 L/s) for the previous development.

Residential flows are determined by multiplying the number of residential units by number of residents per unit and the daily average per capita flow. Peak flows are found using the Harmon Peaking Formula (see Table 3-1). The Harmon Peaking Formula adjusts the peak flow factor based on the population served at each pipe length. Extraneous flows (I&I) are calculated by multiplying the City's standard rate of 0.28 L/s*ha by the contributing area.

Commercial flows that are determined use the standard flow rate of 2,500 L/d*m^2 including extraneous flows. The total peak flow is found by an arithmetic sum:

$$Peak\ Design\ Flow\ (Q_d) = Peak\ Population\ Flow\ \left(Q_p\right) + Peak\ Extraneous\ Flow\ (Q_i)$$

Pipe capacity is solved using Manning's Equation.

For pipes flowing partially full, flow depths and pipe capacities are resolved using MTO's Chart 2.30.

Greater detail for the calculations can be found directly on the Sanitary Sewer Design Sheet, Table 3-1.

The development will drain to a central pump station.

Sewers within the development have been designed to accommodate the proposed development; therefore, the sewer main will be 200 mm in diameter with a minimum slope of 0.4%.

3.4 Pump Station

A sewage lift station has been designed by John Brooks Appendix C. The lift station will include two submersible 3" pumps with 3 phase 4hp motors. The pump rates are 131L/s with an active storage volume of 1m^3. The station will be connected with SCATA controls and alarmed.

Figure 3-1: Porta Sanitary Layout

Table 3-1: Sanitary Sewer Design Sheet

SANITARY SEWER DESIGN SHEET

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

- q = Average daily per capita flow **350** L/d*cap
- I = Unit of peak extraneous flow **0.28** L/s*ha
- M = Harmon peaking factor (min = 2)
- P = Population in 1000's
- A = Area in hectares

Population Flows (Persons/Unit)

Townhomes	**2.5**
Apartment	**2.5**

Commercial Flows

Average Commercial Flows	**2.5** L/d*m^2
Peaking Factor	**2.0**

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

- A = area of pipe in m^2
- R = Hydraulic radius = A / P
- P = Wetted perimeter
- S = Slope (m/m)
- n = Manning's friction coef.

Check

$$Q_d \le 0.8 \cdot (Pipe\ Capacity)$$

$$0.6 \le V \le 3.0$$

use Actual V if d:D < **0.3**

LOCATION			PEAK FLOW CALCULATION													PROPOSED SEWER							
			Residential							Commercial		Population Flow	Commercial Flow	Peak Ex. Flow	Design Flow	Pipe Size	Grade (use m/m)	Capacity n = 0.013	Full Flow Velocity	Ratio d:D	Actual Velocity at Q_d	Check	q/Q
			Individual				Cumulative		Peaking Factor	Individual	Cumulative												
Catchment/ Street	Downstream Manhole	Upstream Manhole	Units		Pop.	Area (A) (ha)	Pop.	Area (A) (ha)	M	Area (A) (ha)	Area (A) (ha)	$Q_{(p)}$ (L/s)	$Q_{(c)}$ (L/s)	$Q_{(i)}$ (L/s)	$Q_{(d)}$ (L/s)	(mm)	(%)	(L/s)	(m/s)		(m/s)		
			Townhome	Apartment																			
	SA12-S	SA7	32		80	0.27	80.0	0.27	4.27		0.00	1.38	0.00	0.08	1.46	200	0.70%	27.4	0.87	0.15	0.00	Velocity	0.05
	SA13	SA11	12		30	0.27	30.0	0.27	4.35		0.00	0.53	0.00	0.08	0.60	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA10-W	SA11	12		30	0.18	30.0	0.18	4.35		0.00	0.53	0.00	0.05	0.58	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA11	SA5	4		10	0.1	70.0	0.55	4.28		0.00	1.21	0.00	0.15	1.37	200	0.40%	20.7	0.66	0.17	0.00	Velocity	0.07
	SA10-S	SA4	20		50	0.16	50.0	0.16	4.31		0.00	0.87	0.00	0.04	0.92	200	0.70%	27.4	0.87	0.12	0.00	Velocity	0.03
	SA12-W	SA9	12		30	0.25	30.0	0.25	4.35		0.00	0.53	0.00	0.07	0.60	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA10-E	SA9	12		30	0.18	30.0	0.18	4.35		0.00	0.53	0.00	0.05	0.58	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA9	SA3	32		80	0.23	140.0	0.66	4.20		0.00	2.38	0.00	0.18	2.57	200	0.40%	20.7	0.66	0.23	0.00	Velocity	0.12
Commercial & Crates	SA8	SA7	18	7	62.5	0.47	62.5	0.47	4.29	1.13	1.13	1.09	0.65	0.13	1.87	200	0.70%	27.4	0.87	0.17	0.00	Velocity	0.07
Amenity	SA7	SA2	4		10	0.06	152.5	0.80	4.19	0.16	1.29	2.59	0.75	0.22	3.56	200	0.40%	20.7	0.66	0.28	0.00	Velocity	0.17
	SA6	SA5	20		50	0.29	50.0	0.29	4.31		0.00	0.87	0.00	0.08	0.96	200	0.70%	27.4	0.87	0.13	0.00	Velocity	0.03
	SA5	SA4	12		30	0.11	150.0	0.95	4.19		0.00	2.55	0.00	0.27	2.81	200	0.40%	20.7	0.66	0.25	0.00	Velocity	0.14
	SA4	SA3	12		30	0.11	230.0	1.22	4.13		0.00	3.84	0.00	0.34	4.19	200	0.40%	20.7	0.66	0.31	0.66	OK	0.20
	SA3	SA2	4		10	0.07	380.0	1.95	4.03		0.00	6.21	0.00	0.55	6.75	200	0.40%	20.7	0.66	0.39	0.66	OK	0.33
	SA2	SA1			0		532.5	2.75	3.96		1.29	8.54	0.75	0.77	10.06	200	0.40%	20.7	0.66	0.49	0.66	OK	0.48
	SA1	PS			0		532.5	2.75	3.96		1.29	8.54	0.75	0.77	10.06	200	0.40%	20.7	0.66	0.49	0.66	OK	0.48

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Designed:	Julie Humphries, C.E.T.	**Project:** Porta
Checked:	Bryon Keene, P.Eng.	
Date:	June 18, 2025	

3.5 Sanitary Sewer Materials – Pipes

3.5.1 Pipe Materials

All sanitary sewer pips will be DR35 PVC. This conforms with OPSS 1841. Pipe joints will be bell and spigot style with a PVC compression gasket. No restraints are required.

PVC pipe is preferred for its ease of installation, fewer pipe joints and long service life in excess of 100 year (IPEX Sewer Piping Systems).

The Manning's n for PVC is published by manufacturers as 0.09 (IBID). As required by the Design Guidelines, a value of 0.013 was used that is representative of a rough surface, which adds conservatism to the calculations.

3.5.2 Bury Depth

3.5.2.1 PVC Deflection

The discussion below is focused on DR35 pipe. Where DR28 is used, it is understood that this grade of pipe is stronger than DR35, so when DR35 is permissible for used, DR28 would do as well.

The maximum bury depth for DR35 PVC is 10.8 m for trench installation and 6.5 m for embankment installation per OPSD 806.040 and all pipes will conform.

DR35 PVC is the industry standard and conforms to the pipe strength requirements and safety factors for OPSS 1841. Industry testing shows that PVC pipe joints can withstand deflections of 30% before leaking and given that PVC pipes are specified to have a minimum factor of safety of 4, the maximum allowable deflection of 7.5%. The OPSD 806.040 table specifies the maximum permissible burial depth to assure the 7.5% deflection limit will not be exceeded. OPSD 806.040 would permit burial depths of up to 10.8 m for DR35 pipe in Type 1, 2, and 3 soil conditions for pipe diameters up to 750 mm. The OPSD tables are based on the industry testing but have arbitrarily applied the 10.8 m limit. IPEX and Westlake publish their own tables for allowable bury depth that exceed the OPSD limits. The manufacturer's tables state they include the highway loading.

3.5.2.2 Concrete Cracking

Concrete pipes fail when subjected to excessive loading conditions which causes cracking. Provision of Class B bedding helps to protect against cracking and permits deeper burial depths per OPSD 807.010. The referenced OSPD table indicates the type of bedding, class of pipe, and maximum permitted burial depths as well as the minimum class of pipe for shallow depths between 0.3 m and 0.6 m. In all cases, the concrete pipes at the development will be within the permitted bury depths for the 65D class of pipe.

Reference was also made to the Concrete Pipe Design Manual, Ontario Concrete Pipe Association for factors of safety against cracking. The OPSD depths achieve the minimum required factor of safety of 1.1.

3.5.2.3 Buoyancy

Sanitary sewers are buried at greater depths and are typically well protected from buoyancy. Factor of safety against buoyancy for sanitary pipes for 1.5 m depth and assuming groundwater 0.5 m below ground is 11 (see <appendix> tables prepared for various pipe sizes).

3.5.2.4 Frost

Minimum depth of cover for frost protection would be 1.2 m for the Quinte Region (O.B.C.). OPSD 3090.010 indicates the frost depth for the Quinte Region is 1.4 m to 1.5 m. Pipes should then have a minimum cover of 1.5 m before requiring additional frost protection measures. All sanitary sewers will have sufficient cover and are protected from frost penetration.

Sewer mains are buried to a depth that allows for laterals to be positioned with inverts a minimum 2.2 m at the property line.

3.5.3 Syphon

No syphons are proposed.

3.5.4 Foundation Drainage

Foundation drains will not be connected to the sanitary system.

3.5.5 Pipe Size

The minimum allowable sewer size is 200 mm in diameter. All sewers will be 200 mm in diameter or greater. Sewers discharge into downstream sewer pipes that are equal or larger in size. At changes in pipe sizes, obverts were matched or, at a minimum, 80% of the diameters were matched.

3.5.6 Flow Velocity

Where pipes are flowing at 30% depth or greater, the pipes achieve target velocity range of 0.6 m/s to 3.0 m/s. Several pipes will be flowing partially full. These are often the first pipe in a branch and City standard requires 0.7% slope. Other pipes that are flowing partially full, but do not have many service connections, will also have velocities that do not achieve the target. The pipe slopes have been increased to help achieve target velocities where possible; however, some pipe slopes are limited by topography and cover, so velocities will fall below the target velocity.

Per MECP 2008, Section 4.4.1, conditions leading to higher risk of sulfide formation include:

- Pipe slope (low slopes)
- Transition structures
- Maintenance holes
- Proximity to receptors
- Inverted syphons and forcemains

Factors that tend to cause high residence time are not present in the design. Maintenance hole benching and pipe slopes and sizes were selected to maximize velocity where possible.

3.5.7 ALIGNMENT

Sewers will be located along the centreline of the roads per City standard. Sewers connect at maintenance hole with right or obtuse angles thereby satisfying the design requirements.

3.5.8 BY-PASS / SURCHARGE / OVERFLOW

The City notes there are no downstream overflow points (CSO or SSO) between the subject site and the WWTP. No by-pass is required for the current development. The system is designed using the maximum expected peak flows and retains a minimum 20% reserve capacity. Surcharge of the system is not likely and impacts to basements are not expected.

3.5.9 SEPARATION FROM DRINKING WATER

No sewers are proposed within 15 m of a drinking water facility. There are no drinking water facilities within 60 m of the development area.

Sewers are separated from watermains by 2.5 m horizontal separation of outside edge of pipes and 0.5 m separation vertically.

3.5.10 LATERALS

Laterals will be 150 mm DR28 PVC. Connections will be made using a manufactured tee. Risers for sewers greater than 4 m buried depth will use long sweep elbows connecting to the main at an angle no greater than 45 degrees. Lateral slopes are set to 2%.

3.6 SANITARY SEWER MATERIALS – MAINTENANCE HOLES

All maintenance holes are to be designed per the latest OPSDs and conform to all required guidelines, such as: Occupational Health and Safety Act, MOL Confined Space Guidelines, Fire Protection and Prevention Act.

3.6.1 SIZE

Sanitary maintenance holes are 1200 mm in diameter. This is the minimum size for the largest

3.6.2 ACCESS AND SAFETY

Maintenance holes access steps will conform to OPSD 405.010, which will facilitate safe access for operational maintenance.

No safety platforms are required since all structure heights are less than 5 m.

3.6.3 SEALING

Maintenance hole seals will conform to OPSS 1351. Additional seals are specified using Blue Skin or Riser Wrap.

3.6.4 FLOW ACCOMMODATION

No drop structures are needed at any of the maintenance holes for the development.

All sanitary maintenance holes are to be benched to ensure smooth flow transition to reduce sediment deposition. Benching will conform to OPSD 701.021.

Inverts are calculated such that all outgoing pipes are 3 cm lower than incoming pipes when pipes are 180 degrees apart and 6 cm lower than incoming pipes when pipes are 90 degrees apart.

3.6.5 SPACING

Maintenance hole spacing is specified by MECP 2008 Design Guidelines which require spacing to be no greater than 120 m for pipes sizes up to 375 mm, 150 m for pipes from 450 mm to 750 mm and up to 185 m spacing for pipes larger than 750 mm.

The City standard requires spacing between maintenance holes to be no greater than 120 m for pipes up to 400 mm in diameter and 140 m for pipes larger than 400 mm. All pipes are 200 mm in diameter; therefore, the maximum allowable spacing is 120 m.

All pipe lengths are measured centre-to-centre of maintenance holes and the pipe lengths are all below the 120 m maximum permissible.

3.6.6 Grading

Sanitary maintenance holes are typically placed at the centreline of the road, which is the high point of the cross-section. This will reduce surface infiltration into the maintenance holes.

3.6.7 Corrosion Protection

Section 5.9.9 of Design Guidelines states, "where corrosive conditions due to septicity or other causes are anticipated, corrosion protection on the interior of the manholes should provided". Concrete is not susceptible to corrosion due to hydrogen sulphide gas, but metals are. Hydrogen sulphide gas is produced from sewage that has a long residence time in sewers. By maintaining sewer velocities above 0.6 m/s, the risk of generating hydrogen sulphide gas is low. If hydrogen sulphide is converted to sulphuric acid in the presence of aerobic bacteria and moisture along the crown of the pipe, corrosion of the concrete surface may occur.

As much as possible, sanitary pipes will be placed at or above minimum slopes to reduce the risk of development of corrosive conditions. Similarly, high velocities can cause the development of corrosive conditions by entraining oxygen in the flows and causing the release of hydrogen sulphide and generation of sulphuric acid.

Factors such as temperature, pH, chemicals in wastewater can lead to corrosion of the metal components and the concrete. The City's Sewer Use By-law should be enforced to provide assurance that external factors do not increase the risk of corrosion.

Steps are fabricated of aluminum reduce the susceptibility to corrosion.

4 CONCLUSIONS

The Porta development can be developed on full municipal water and sanitary services. The proposed 200 mm watermain will complete a loop through the site to eliminate two dead-ends. This will improve overall water service in the area for water quality and flows.

A 200 mm gravity main will provide local sewage collection, but a pump station is needed to lift sewage to the 900 mm main on Dundas Street. The sewage lift station and sewers will remain in private ownership.

Prepared by:	Submitted by:
Julie Humphries	LICENSED PROFESSIONAL ENGINEER B. C. KEENE July 18/25 PROVINCE OF ONTARIO
Julie Humphries, C.E.T. Jewell Engineering Inc.	Bryon Keene, P.Eng. Jewell Engineering Inc.

5 REFERENCES

5.1 WATERMAIN

The information used to prepare this report is based on the following documents and information provided as noted below:

- City of Belleville Standard Specifications
  - 1010 Watermain Distribution – General
  - 1020 Watermain Distribution Design – General
  - 1030 Watermain Distribution Construction – General
  - 1110 Watermain Pipe
  - 1120 Watermain Flow Control Valves
  - 1130 Fire Hydrants
  - 1140 Service Pipes
  - 1150 Meters
  - 1160 Corrosion Protection
  - 1170 Temporary Watermains
  - 1190 Commissioning New Watermains
  - SD-WD-1001 Pipe Embedment
  - SD-WD-1002 Mechanical Joint Restraint
  - SD-WD-1010 Deflection of Watermain Under New Sewer
  - SD-WD-1011 Deflection of Watermain Under Existing Sewer
  - SD-WD-1020 Watermain Pipe Installed in Encasement (Trenchless)
  - SD-WD-1021 Watermain Pipe Installed in Encasement (Open Trench)
  - SD-WD-1030 Styrofoam Insulation for Existing Shallow Watermains
  - SD-WD-1031 Placement of Watermain Adjacent to Catch Basin
  - SD-WD-1040 Blow-off Assembly
  - SD-WD-1041 Temporary Bacteriological Test Sampling Assembly
  - SD-WD-1101 Fire Hydrant Installation
  - SD-WD-1201 Copper Water Service
  - SD-WD-1202 Polyethylene Water Service
  - SD-WD-1210 Styrofoam Insulation for Existing Shallow Water Services
  - SD-WD-1301 Valve Bypass Assembly
  - SD-WD-1901 Terminology Used for Drinking Water Systems Servicing Buildings
- Ontario Ministry of Environment
  - Design Guidelines for Drinking-Water Systems, 2008
- Fire Underwriters Survey
  - Water Supply for Public Fire Protection, 2020

5.2 Sanitary Sewer

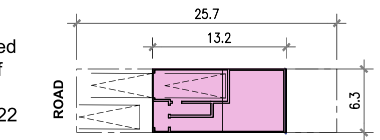
The information used to prepare this report is based on the following documents and information provided as noted below:

- Ontario Ministry of Environment
  - Design Guidelines for Sewage Works, 2008
  - Design Criteria for Sanitary Sewers, Storm Sewers, and Forcemains for Alterations Authorized under an Environmental Compliance Approval, v2.0, 2023

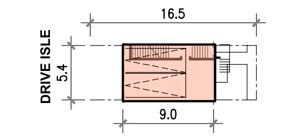
APPENDIX A:
PORTA CONCEPT PLAN

Total 213 Residential Units

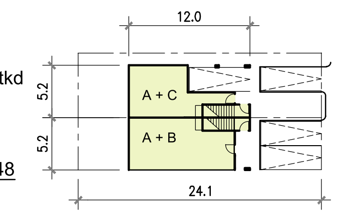
TYPE A
2 Units Stacked
Unit A: 2200sf
Unit B: 1430
Total Units = 22



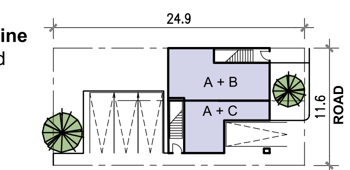
TYPE B
2 Units Stacked
Unit A: 1385sf
Unit B: 550sf
Total Units = 88



TYPE C
2 @ 2 Units Stkd
Unit A: 900sf
Unit B: 750sf
Unit C: 650sf
Total Units = 48



TYPE D - Rail Line
2 @ 2 Units Stkd
Unit A: 800sf
Unit B: 700sf
Unit C: 600sf
Total Units = 48



TYPE E
Condo
850sf
Total Units = 7

Statistics to come:

Per Unit type (A,B,C,D E):
1. Bedrooms / w/c's (estimate)
2. max height
3. lot area
4. frontage
5. lot coverage percentage
6. min. landscaped area
7. min front, side, rear yard setbacks
8. parking spaces per unit

General:
1. site dimensions
2. parking count visitor
3. commercial area
4. amenity area (building)
5. other...

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Porta - Townhome Study
25 Dundas Street West, Belleville Ontario

Client: MDM Developments

drawing: Low Rise Site Plan Sketch - V4,
Legal Boundaries Updated - 21M318

issued for: Consultant Coordination

scale:	1:750
job number:	21009
drawn by:	RD
dwg. ref. no:	N/A

plot date: 2025.06.06

drawing no: 1

APPENDIX B:
BLUMETRIC ENVIRONMENTAL RESPONSE

October 14, 2022
Project Number: 210367

Thomas Binczyk, P.Eng.
Belle Harbour LP
190 Hotchkiss Street
Gravenhurst, ON P1P 1H6

Re: Watermain Design Brief for 25 Dundas Street West, Belleville – Environmental Response to City of Belleville Comments dated July 29, 2022.

Dear Mr. Binczyk,

BluMetric Environmental Inc. (BluMetric ®) understands that on July 29, 2022 the City of Belleville provided comments and requested clarification to environmental conditions on the property at 25 Dundas Street West in Belleville (Subject Property) as they may pertain to the proposed new watermain installation. We have reviewed the Watermain Design Brief with revision date November 10, 2021 by Jewell Engineering as well as the comments from the City of Belleville date July 29, 2022.

BACKGROUND

BluMetric was first retained in 2012 to complete Phase One and Phase Two Environmental Site Assessments (ESA) on the Subject Property in order to characterize and quantify environmental impacts in soil and ground water. Between 2012 and February 2021 BluMetric conducted various site investigation activities and prepared a Phase One ESA Update, a Phase Two ESA and two Risk Assessments for the site. All environmental reporting is compliant with Ontario Regulation 153/04 and was intended to support a Record of Site Condition (RSC) for the Property.

For the purposes of the ESA work (i.e. Phase Two ESA and Risk Assessment), the 25 Dundas West property was divided into two parts: the Table 9 Parcel and the Table 7 Parcel. The Table 9 Parcel includes all the land within 30 metres (m) of the Belleville Harbour. It was assessed using the Ministry Table 9 Site Condition Standards (SCS) which are intended to be used to assess sites within 30 m of a surface water body in a non-potable ground water condition. The remainder of the 25 Dundas West property (The Table 7 Parcel) is greater than 30 m from surface water and is assessed

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using the Table 7 SCS which are intended to be used to assess sites with "shallow soils" in a non-potable ground water condition.

From the Watermain Design Brief, the watermain appears to be located along the north side of the property, i.e. in the Table 7 Parcel.

The Risk Assessment for the Table 7 Parcel was accepted by the MECP in February 2021 with a reference number (RA 1470-15). From the Risk assessment the MECP prepared a Certificate of Property use (CPU) which was finalized in October 2021 (CPU # RA1470-15-01). The MECP further acknowledged a RSC (#231113) for the Table 7 Parcel in May 2022.

The following sections provide excerpts and summaries of information from the various environmental reports and documents for the site to address the City's comments.

City request to identify Contaminant of Concern for the Site

From the Phase Two Conceptual Site Model for the Table 7 Parcel the following contaminants of concern (COC) were found to exceed the applicable generic standards in soil:

- **Metals:** Barium, Cadmium, Copper, Lead, Nickel, Thallium, Zinc
- **Hydride Forming Metals:** Antimony, Arsenic
- **Other Regulated Parameters (ORPs) in soil**: Mercury, SAR, HWS-boron, EC
- **PHCs:** F2 fraction, F3 fraction
- **PAHs:** Acenaphthylene, Anthracene, Benzo[a]anthracene, Benzo[a]pyrene, Benzo[b]fluoranthene, Benzo[k]fluoranthene, Dibenzo[a,h]anthracene, Fluoranthene, Indeno[1,2,3-cd]pyrene, Naphthalene, Phenanthrene
- **BTEX:** Benzene, Toluene, Xylene Mixture (Xylenes)

Initial groundwater sampling indicated that the following compounds may be present in the ground water at the site.

- **BTEX:** Benzene, Xylenes
- **VOCs:** Choroform, 1,4-Dichlorobenzene

Based on additional extensive testing BluMetric concluded, and the MECP agreed, that the initial groundwater samples do not represent the groundwater at the Site, as described below (see the Phase Two Conceptual Site Model):

- The Benzene and Xylenes detected in the initial ground water samples did not leach from the soil but are naturally occurring and are entirely attributable to the petroliferous / bituminous nature of the Verulam formation in Belleville.
- The single detection of Chloroform in one initial ground water sample was a laboratory anomaly, it was not detected in any subsequent ground water samples.
- The single detection of 1,4-Dichlorobenzene in one initial ground water sample was due to dilution and detection limit issues, it was not detected in any subsequent ground water samples.

City concerns regarding contaminant migration

The COCs in soil have been on the site for 80 years or more. No impacts to ground water arising from historical industrial activities on the site were identified and the majority of impacts found in soil were relatively insoluble parameters (metals, PAH, F2 & F3 PHC). More soluble contaminants in soil such as mercury, SAR, EC, B(HWS), benzene, toluene and xylenes were dispersed throughout the site with no clear trend of migration. This suggests that their presence is associated with small, isolated releases or placement and grading of poor quality fill throughout Table 7 Parcel. Fluctuations in ground water level due to climatic or meteorological conditions are likely to have occurred over time, but these fluctuations do not appear to have significantly affected the distribution or migration of contaminants and there is no evidence thus far that COCs are partitioning into ground water on the site. This is not likely to occur in the future.

There are underground utility trenches, conduits, and structures currently on the adjacent Table 9 Parcel. These could influence ground water movement on the Table 7 Parcel; however, since no impacts in ground water have been identified this does not need to be addressed further.

City comment on ground water management and dewatering

If ground water management is necessary during construction a dewatering contractor should be retained. The water should be tested for the CoCs listed above as a minimum as well as total suspended solids and any other parameters required by the licensed receiving facility. Depending on the volume of ground water to be removed from trenches permits may be required through the MECP.

City comment on watermain bedding and backfill

Any parties undertaking excavation works now or in the future should review the details in the CPU and consult the owners to obtain the site-specific health and safety plan and other necessary details on site management, in advance of the commencement of work. BluMetric has prepared an Environmental Management Manual for the site, which summarizes actions and precautions to be taken to ensure safe work, to manage excavated soil, to minimize disruption of barriers once in place and to ensure compliance with the CPU for the site.

From the watermain design brief, utility trenches are planned to be excavated to remove any contaminated soil and will be lined with geotextile to prevent contact of backfill with any contaminants in the base and sidewalls of the excavation. It is understood that the backfill will consist of clean material. As detailed above there is little likelihood of contaminant migration via ground water into the clean material. Further, this new utility trench is not expected to act as a conduit for contaminant migration. Neither the backfill nor the watermain pipe are expected to be in contact with appreciable concentrations of the COCs, and in particular organic COCs, in soil at the site, and the risk of pipe wall or gasket permeation by the COC on the site is negligible.

Future excavation for repairs of the watermain should maintain the geotextile liner intact or replace sections that are disrupted.

Any hard of fill caps that are disturbed for watermain work must be reinstated as per section 4.2 of the CPU and as summarised in Section 1 of the Watermain Design Brief.

City comment on health and safety for trench workers

As part of the Environmental Management Manual BluMetric prepared a health and safety plan (HASP) for the site. This is a requirement of the CPU as detailed in Section 4.5 of that document. The HASP will be a living document and may be updated from time to time. Excavation workers including utility works should get in touch with the site owner and obtain a copy of the HASP prior to commencing work to ensure that risks are understood and that proper PPE is used and management measures are followed.

If you have any questions or require clarification please do not hesitate to contact the undersigned.

Regards,

BluMetric Environmental Inc.

Paul Bandler, M.Sc.
Senior Scientist

BluMetric Environmental Inc.

Keith Metzger, M.A.Sc., P.Eng., QP_{RA}, QP_{ESA}
Senior Engineer, Team Lead – Risk Assessment

APPENDIX C:
FUNCTIONAL SERVICING REPORT, JOHN TOWLES ASSOCIATES LIMITED

JOHN TOWLE ASSOCIATES LIMITED
CONSULTING ENGINEERS

City of Belleville
City Hall
169 Front Street
Belleville, ON
K8N 2Y8

September 26, 2014

Project No: 13100

Attention: **Mr. Phil Cantelo, P. Eng.**
Development Engineer
Engineering & Development Services

Reference: East Marina Investments Ltd.
Proposed Mixed Use Condominium Development
25 Dundas Street West
City of Belleville
FUNCTIONAL SERVICING REPORT

Dear Sir:

In response to the City of Belleville's latest comments, we have completed revised calculations related to the sanitary discharge from the marina lands for both the original site development and for the proposed site development. Also, included is the Water Demand analysis for the proposed site development.

A. SANITARY SEWER FLOW ANALYSIS

(i) Original Site Development

The original site was composed of five (5) buildings all itemized in excerpts from the Real Estate Overview Prospectus found in the Appendix. This Prospectus outlines the building uses, associated building GFA, etc.

Building 3 (Harbour Fitness Club) represents the building that would generate the most sewage flow as it contained a fitness club with showers, washrooms, swimming pool, etc., as well as large capacity restaurant with a large patio area. The Project Planner, Ruth Ferguson Aulthouse, has been a long time Belleville resident and confirms that weddings were also hosted here as it was the main "meeting" location for several years. The Prospectus notes that, including staff, approximately 382 people could be in the restaurant and outdoor patio areas at one time. Estimated sewage flow was calculated using MOE Guidelines for a shopping centre, i.e. Peak Flow at 5.0 $l/day/m^2$.

Ms. Aulthouse interviewed the Manager of the former Harbour Fitness Club to arrive at the 165 people that could attend the various fitness programs, classes, clubs at any one time. Estimated sewage flow for the Club was calculated using the City of Belleville's existing peak residential flow rate of 1,815 l/cap/day.

The estimated peak sewage flow for the commercial/retail component (Buildings 1, 2 and 4) was calculated using the MOE Guideline for a shopping centre, as well.

The estimated site sewage flow including the marine buildings is calculated to be 5.09 l/s. This sewage was collected on site in a sewage pump manhole and pumped through a 100mm dia. forcemain on Dundas Street at Mary Street. Original plans retrieved from City archives showed this forcemain to be connected to a 600mm dia. sanitary sewer running east on Dundas Street. However, a more recent investigation revealed that this was disconnected when Building 3 was designed around 1988. The 100mm dia. forcemain was re-routed along Dundas Street to James Street, to the east, to discharge to a 900mm dia. sanitary sewer. These engineering plans are included in the Appendix. It can only be assumed that the sewage flow calculations provided in 1988 for the 10 Unit commercial complex (Building 3) indicated that the capacity of the existing 600mm dia. sanitary sewer would be exceeded thus requiring a connection to the larger 900mm dia. sanitary sewer to the east.

(ii) Proposed Site Development

The proposed development of the marina lands calls for the construction of three (3) apartment buildings totaling approximately 303 units. (See Appendix) This area would be severed from the marina property, where Buildings 1 and 2, would remain.

The proposed development includes ground floor retail.

The total proposed site discharge, including that from Buildings 1 and 2, is estimated to be approximately 13.13 l/s.

B. WATER DEMAND ANALYSIS

The City of Belleville's Water Distribution Department performed hydrant flow tests on two (2) public fire hydrants fronting #48 Dundas Street West and #68 Dundas Street West in March 2007. These hydrants are connected to an existing 450 mm dia. trunk watermain on Dundas Street West. A 200 mm dia. watermain currently services the marina lands off this watermain.

The proposed development intends to utilize the existing 200 mm dia. public watermain that extends under the CP Railway corridor into the site. There is a 150 mm dia. watermain on Mary Street that originally serviced the site but this is not connected to the trunk watermain and has been abandoned within the site.

Even though the test results are from March 2007, we have been informed that water demand in the area has not changed significantly the results could be used for this water demand analysis. See Appendix.

Estimated fire flow calculations were based on design criteria and assumptions found in the Fire Underwriters Survey – Water Supply for Public Fire Protection (1999).

This design criteria and assumptions are noted in the Water Demand Analysis found in the Appendix.

In particular, the floor area used was obtained from the site statistics provided by Sweeny & Co Architects Inc. as found in the Appendix. We are informed that they represent the maximum allowable for the site, hence worst case scenario.

Three (3) apartment buildings are proposed for the site, all of which are constructed on grade. All parking has been designed at grade level or in adjacent above grade parking garages.

Consequently, ground level for each building is primarily parking with minimal non-residential use. The apartments are located on Level 3 and above.The current architectural plans with building sections, individual floor level layouts, etc. are included in the Appendix.

The Architect has confirmed that fire restrictive construction design will be utilized resulting in the C = 0.6 factor being used. They also confirm the use of non-combustible building materials and that the buildings will be designed with sprinklers.

All of the associated flow reductions and/or additions using appropriate factors and coefficients have been used in the Water Demand Analysis.

The existing marina repair building to the east is over 45 m from the east apartment building line of Building A and consequently was not considered in the fire flow calculations. It was, however, included in the water demand calculation.

The total water demand for the site is calculated to be 1425 gpm and based on the City's hydrant flow test on the Dundas Street West hydrants, 80 psi residual pressure results which easily exceeds the City of Belleville requirements. Expected pressure drops from the Dundas Street West hydrants into the site are not anticipated to result in a substantial reduction in residual pressure.

A more detailed analysis would be performed at the design stage.

C. STORMWATER MANAGEMENT

The 100 year Floodline on site was determined by Quinte Conservation to occur at elevation 75.9 m GSC.

This flooding is a result of frequent ice jamming (frazzle ice) at the mouth of the river. See attached Quinte Conservation schematic in the Appendix that shows the location of this floodline. Flood proofing will be provided above this elevation.

Basements or underground parking are not proposed and consequently all building openings at ground level (Level 1) will be designed above elevation 75.9 m GSC.

A topographic survey was undertaken to establish more relevant site elevations on site due to demolition activities. The Draft Plan of Subdivision showing the 100 year floodline is found in the Appendix and more accurately locates the 75.9 m GSC floodline location. It extends only into the Block 3 (Building B) area. The floodline will be adjusted at the site grading design stage such that the southerly condominium road elevation will be established at an elevation above elevation 75.9 m so that floodline will not extend north onto the building areas. The landscape plaza area at the southeast corner will be designed to provide the equivalent volume lost to this southerly shift of the 100 year floodline.

Previous discussions with Quinte Conservation and City Engineering staff confirmed that stormwater quantity control will not be required due to adjacent Lake Ontario. Only stormwater quality control would be required in the form of a Stormceptor manhole or equivalent system.

D. CONCLUSIONS

The site sewage flow calculations for the original development and the proposed development indicate an 8.04 l/s increase is generated by the current proposal.

It is recommended that the existing 100mm dia. forcemain connection to the 900mm dia. sanitary sewer on Dundas Street be maintained and utilized for the proposed apartment development and marine buildings. Our preliminary review indicates that the proposed peak discharge rate of 13.13 l/s can be adequately accommodated in the existing 100 mm dia. forcemain.

The final design of the pumping station will establish the optimum pumping rate using friction and head losses such that minimum flow velocity is achieved, all in accordance with the latest MOE Guidelines.

The sewage from the proposed development will still need to be collected and pumped from the site, as before, the rate of which will be determined by the sewage pump capacity. The proposed site sewage flow represents only 1.02% of the capacity of the 900mm dia. sanitary sewer, which is insignificant.

The Owner's acknowledge that the pumping station will be privately owned and operated by the Condominium Corporation and the marine lands by way of a private Servicing Agreement. The laterals, pumping station and forcemain to the Dundas Street West streetline will be maintained under the terms of this Agreement with no recourse to the City of Belleville.

Residual water pressure is calculated to be approximately 80 psi using hydrant flow test results on hydrants located at #48 Dundas Street West and #68 Dundas Street West. This easily exceeds the 20 psi minimum static pressure required by the City of Belleville.

The site can be graded such that all building openings will be above the established 100 year floodline elevation of 75.9m GSC and said floodline location can be adjusted to be contained mainly in the landscape plaza area without compromising the 100 year storage volume.

Stormwater quality control of the site's runoff will be designed to City and Quinte Conservation standards.

In conclusion, it is felt that the development, as proposed, will not have any significant impact on the municipal servicing infrastructure The site area represents a very small percentage of the contributing area and associated sewage flow to the existing sanitary system on Dundas Street.

It has also been demonstrated through recent hydrant flow testing that adequate residual static pressures will be maintained on the existing municipal watermain system for the water demand generated by the proposed development.

Stormwater management issues can be easily met at the detailed design stage.

Yours truly

JOHN TOWLE ASSOCIATES LIMITED

REGISTERED PROFESSIONAL ENGINEER
T. TOWLE
PROVINCE OF ONTARIO

John Towle, P. Eng.
President

APPENDIX

- Site Overview from Prospectus
- Surveyor's Real Property Plan
- Belle Harbour Site Description
- Schedule A – Original Site Plan
- Schedule B – Proposed Site Plan and Architectural Plans
- Schedule C – Proposed Draft Plan of Subdivision
- Sanitary Sewer Flow Calculations and Supporting Documents
  - Original Site Development
  - Proposed Site Development
- Sanitary Sewer Area Plan
- Ex. 100 mm dia. Sanitary Forcemain Plans Dundas Street West
- Water Demand Calculations
- Stormwater Management

Site Overview
25 Dundas Street West, Belleville, Ontario

Lot Area:	Part 1	5.310 acres ±		
	Part 2	0.527 acres ±		
	Part 3	0.167 acres ±		
	Part 4	0.043 acres ±		
	Part 5	3.140 acres ±		
	Part 6	0.074 acres ±		
	Part 7	0.055 acres ±		
	Total	9.316 acres ±	(3.77 ha. ±)	
Building Areas:				
Marine	Building 1			6,510 s/f ±
	Building 2:			
	Ground Floor		5,520	
	Second Floor		1,800	7,320
Fitness Centre/Restaurant	Building 3:			
	Ground Floor		22,637	
	Second Floor		13,020	35,657
Three story Building	Building 4:			
	Ground Floor - Retail		9,828	
	Second Floor - Office		9,492	
	Third Floor - Residential		5,000	24,320
Single Family Home	Building 5			2,500
Number of Boat Slips				225
Zoning:	C6 – Water Oriented Commercial Zone; C6-2 – Water Oriented Commercial Zone, Special Provision E – Environmental Control Zone			

Please note: This information is for discussion purposes only.

Belle Harbour Site Description

Parcel 1 was known as Morch Marine/Belle Harbour property which contains 5.31 acres. It is designed as Part 1 on 21R-7426.

Parcel 2 is the site on which the 3 storey Residential/ Office/Retail building sits. It contains 0.527 acres and also uses some parking areas on Parcel 1. It is designated as Part 2 on 21R-7426.

Parcel 3 is the site of a single-family residence. It contains 0.167 acres and is designated as Part 4 on 21R-7426.

Parcel 4 is three peripheral parcels that are located along the north boundary of the site. They contain a total of 0.172 acres. Parcel 5 is located to the west of Parcel 1 and fronts also on Bay Bridge Road. It contains a total of 3.140 acres and is designed as Part 5 on 21R-7426.

Total area of the entire site is 9.316 acres.

Parcel 1 is the major component of the site and is an irregular shaped property that does not have any actual road frontage along Dundas Street West. It is accessed via Mary Street, a very short distance from Dundas Street West, over the CPR line which forms the north boundary of this site; area of this site is 5.31 ± acres. The site has 964 feet ± total frontage along the Bay of Quinte, which is all improved with sea walls. It is described as Part 1 on 21R-7426.

The main site entrance slopes upwards and then slopes downward after crossing the CPR line, with the main body of the site being mostly level throughout. The north boundary backs onto the CPR line, and has an older privacy fence along a portion for some buffer.

The site is improved with three buildings, utilized as a marina complex and health club. The areas of the site not covered by improvements consist of paved and graveled parking areas for boats, etc., as well as vehicle access throughout the site and the waterfront.

There is a gravel surfaced parking area just south of the Health Club (Building 3) with some grassed area as well. A gravel pathway commencing at the subject site leads to Zwick Island Park which is situated just south of the subject property.

Shoreline improvements consist of concrete, steel and wood seawalls, and interlocking brick and paved walkways. There are a total of 225 slips situated on two water lots.

Most of the slips have hydro, but exact numbers must be confirmed. Approximately 30 slips are for transient use.

Most slips are a series of steel finger docks from longer steel docks from the mainland. The slips at the north end to the rear of the main building are wood. There is a wharf which forms a U-shaped bay here.

Belle Harbour Site Description

Other shore improvements include fuel facilities (gas and diesel), these tanks are now empty and may require repairs to comply with TSA standards.

The site is serviced with municipal water and sewer, hydro, telephone and natural gas. The sewer is pumped to the main city sewer lines from a central pumping station.

As part of the marina operation, there is one leased and one managed waterlot.

Leased Areas:
Waterlot # 1 – This area is leased from the Federal Government and covers an area of 2.415 acres lying directly in front of the subject marina, on the Bay of Quinte. It is irregular in shape and also includes two rectangular shaped floating tire breakwater units lying directly south of the leased water lot area. This is a Federal Government lease and the buyer cannot use the area for any other purpose other than in connection with the marina operation.

Waterlot #2 - The Managed Waterlot lies directly south of the marina complex, and technically, does not have direct access from the main body of the subject's upland parcel. This managed waterlot was originally a marina operated by the former Four Seasons Hotel (now Ramada Inn) under a waterlot lease to the Federal Government. The buyer will require access over the Ramada property to access the marina slips

Along the shoreline in front of these two parcels has been improved with wood, concrete and steel retaining walls.

This managed area has an area of 2.9 acres . A portion has been filled and a reserved L-shaped piece jutting out into the water is utilized for docking purposes.

Parcel 2 is a separately deeded parcel improved with a two-storey frame office/apartment building that is accessed over an informal right-of-way over the subject site.

Parcel 3 is Part 3 on 21R-7426, a near square shaped parcel adjacent to Part 2, and is only accessible over an information right-of-way over the subject site.

Parcel 5 is an undeveloped, irregular shaped parcel located immediately adjacent to parcel 1. It is a level and generally clear site and is accessed via a right-of-way over adjoining lands from Bay Bridge Road. It has frontage on Bay Bridge Road of 135.59 ft. but the frontage is located where access is not permitted.

Parcel 1 is improved with a marina complex consisting of 3-buildings, two of which are utilized as part of the marina, and one which is utilized as a health club with a two-level restaurant. There is ample on-site parking, and 225 boat slips. A tennis court is also on site, part of the former health club. This site is the former site of a planning mill (circa 1874) and boat building operation (1966 to 1972).

Information contained herein has been obtained from other sources deemed reliable. We have no reason to doubt its accuracy but regret we cannot guarantee it. It is recommended the Buyer completes their own due diligence.

Coldwell Banker Ekort Realty

Belle Harbour Site Description

Gross areas Marina site buildings are as follows:

Building 1		6,510 sq.ft. ±	
Building 2	Main	5,520 sq.ft. ±	
	Second	1,800 sq.ft. ±	
	Sub-Total		7,320 sq.ft. ±
Building 3	Main	22,637 sq.ft.±	
	Second	13,020 sq.ft. ±	
	Sub-Total		35,657 sq.ft. ±
Total		49,487 sq.ft. ±	

Building 1
This is a rectangular building of steel and concrete block construction that is utilized for sales and service of boats, motors, ATV's, etc. The building has an estimated clear ceiling height of 18 ft. which allows for a storage mezzanine area at the north end of the building. The floor is concrete, the roof is flat with tar and gravel surface. The sales area for ATVs has painted drywall walls and ceilings (mezzanine above). The building is sprinklered, and is heated via suspended, gas fired heaters, and a forced air gas furnace. There is one overhead door along the west side of the building into the sales area, and at the south end for the service area.

The building is older (exact age unknown) and since that time, it has been renovated and upgraded with new windows, exterior siding, overhead doors, lights, etc. for its present use.

Building 2
This building is one and partial two-storey building that is divided into two sections occupied by two different tenants.

This building was renovated in 1988, and now has colorlok exterior siding (2-storey section) with some chipboard at the east end, and a steel roof. The building is on a slab foundation, is steel and concrete block framing with woodframe on the second level.

C. Keeble for Sails Ltd. occupy the west end of the building, with 4,500 sq.ft. of ground floor space and 1,800 sq.ft. on the second floor. The ground floor space has painted concrete floor, pegboard walls, painted drywall and T-bar ceilings with fluorescent light fixtures. The upper level is open concept and has a finished hardwood floor.

There is a boater's washroom accessed along the south side of the subject building with a sink, toilet and a shower stall. A ladies' washroom contains two stalls and three sinks. There are common washrooms, with combination lock access.

The northeast end of the building is utilized as a service shop for ATV's, etc. Floors are concrete, ceiling is open and walls are painted drywall. An overhead door provides for vehicle access.

Belle Harbour Site Description

Building 3
This is the largest of the three buildings and it, too, is an older building that has been renovated and added to and converted for use as a health and fitness club, with restaurant facilities. Please note the entire facility is now vacant and all sports equipment has been removed.

The building was completely renovated in 1984 to contain a restaurant, an indoor pool, men's and women's washrooms and locker rooms, squash courts, racquetball courts, weight room and a daycare centre on the ground floor. The second floor contained a lounge, tanning room, men's and women's washrooms, and an aerobics gym. Within the past few years, the building has had a small one-storey addition and additional renovations and upgrades completed.

The former nautilus gym has been divided into two separate areas, one being a spinning studio, the remaining area an expansion of the daycare area. One of the squash courts now contains the fitness equipment and weights. A second floor was built over one of the squash courts as well as the racquetball courts and now on this second level, there is a second new aerobics studio, and a cardio and equipment room. The former tanning room is utilized for storage and a junior lounge is now an office. Approximately 1,472 sq.ft. of the second floor area is open area for the three remaining squash courts on the main floor.

The restaurant area had a seating capacity of 60 persons on the main floor, 160 on the second floor and 150± on the outdoor patio.

The building is of concrete block and steel construction with metal exterior siding and asphalt shingle roofing. The main floor of the building is sprinklered, thought the building was not winterized and both domestic water and sprinklers will require repairs for broken pipes.

The upper aerobics studios and fitness rooms have open ceilings that have been sprayed with insulating foam. Remaining ceilings are a mix of suspended tiles in T-Bar frames, or painted drywall. Floor coverings are also a mix of carpet, ceramic tile, hardwood in fitness areas and squash courts, and carpet squares. Stairwells are primarily painted concrete block, and brick, with T-bar ceilings.

The men's and women's washrooms on the main floor were in relatively good condition. The women's washroom has pedestal sinks, four separate stalls, ceramic floors and partial walls, spacious locker and change area.

The pool area has an open ceiling above a portion only, with sloped double glazed windows along the side providing ample natural lighting. The remainder of ceiling is open web steel joists and steel pan. Approximately 170 sq.ft. of the second floor area is open space to the pool below.

Belle Harbour Site Description

Main Building : The building located on Parcel 2 is a three storey, woodframe, mixed use structure containing a total are of 19,658 sq.ft. ± on the first and second floors. It now vacant save for the residential tenants on the third floor and one small commercial tenant on main level.

Floors 1 and 2 were utilized as professional offices, while the third floor houses four rental apartments. The third floor is approximately 5,000 sq.ft ±, for a total building area of 24,658 sq.ft. ±.

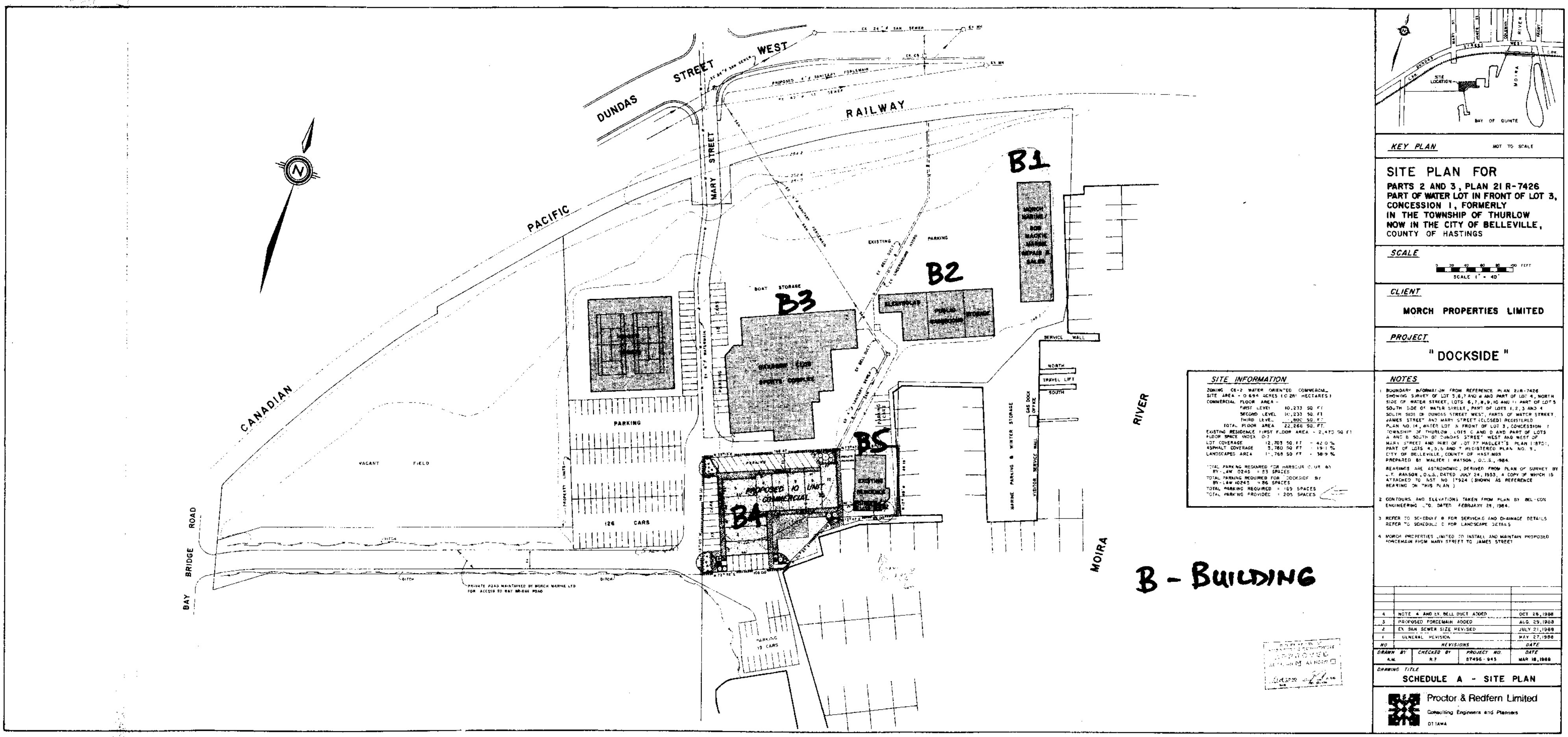
Of the ground floor, 6,612 sq.ft. ± was occupied by Pathways to Independence. The entire second floor is approx 9,492 sq.ft. ±. This would exclude stairwells.

The building is heated by a series of gas furnaces covering different areas and there are a number of roof and balcony mounted air conditioning units.

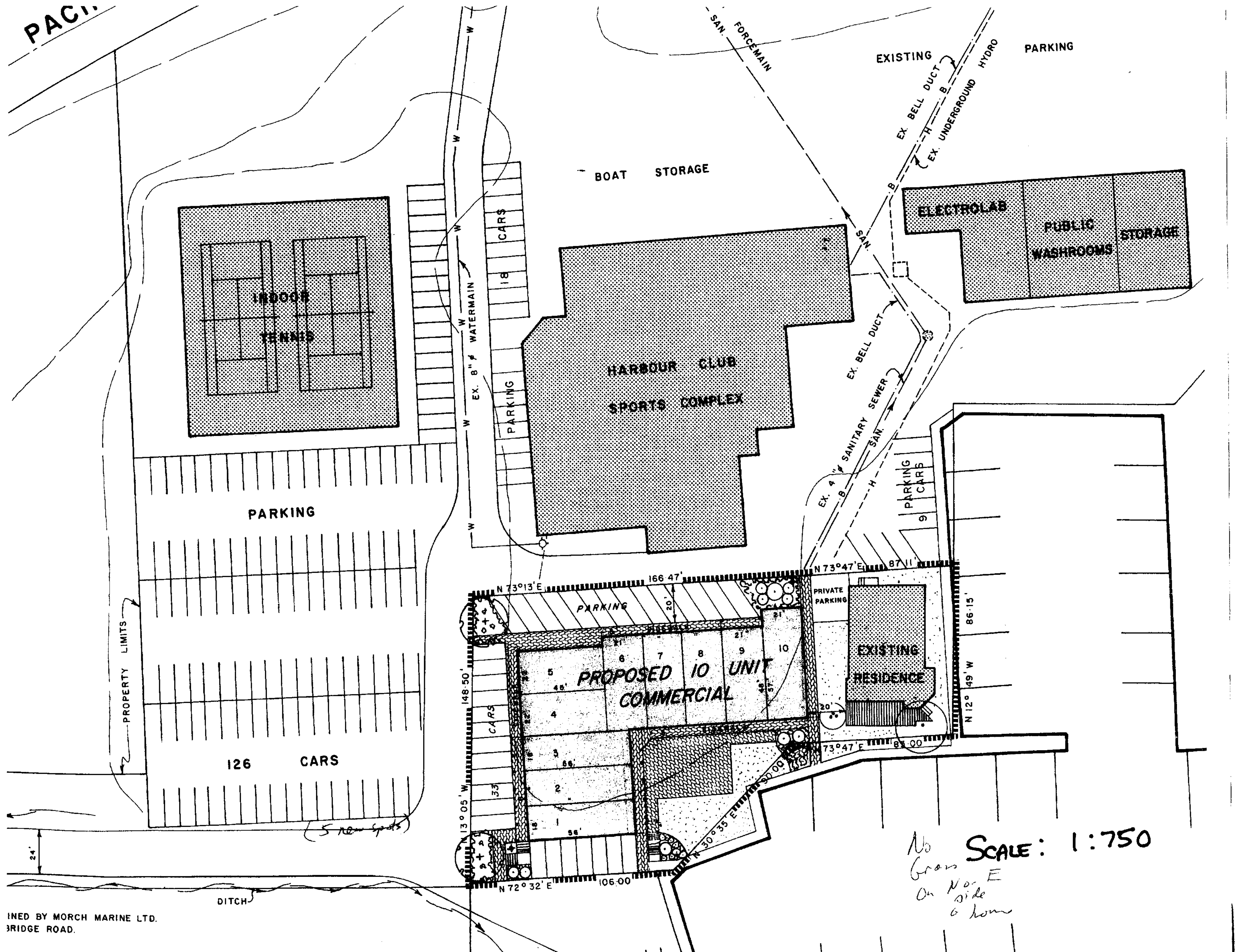
The ground floor is concrete slab and the upper floors are woodframe.

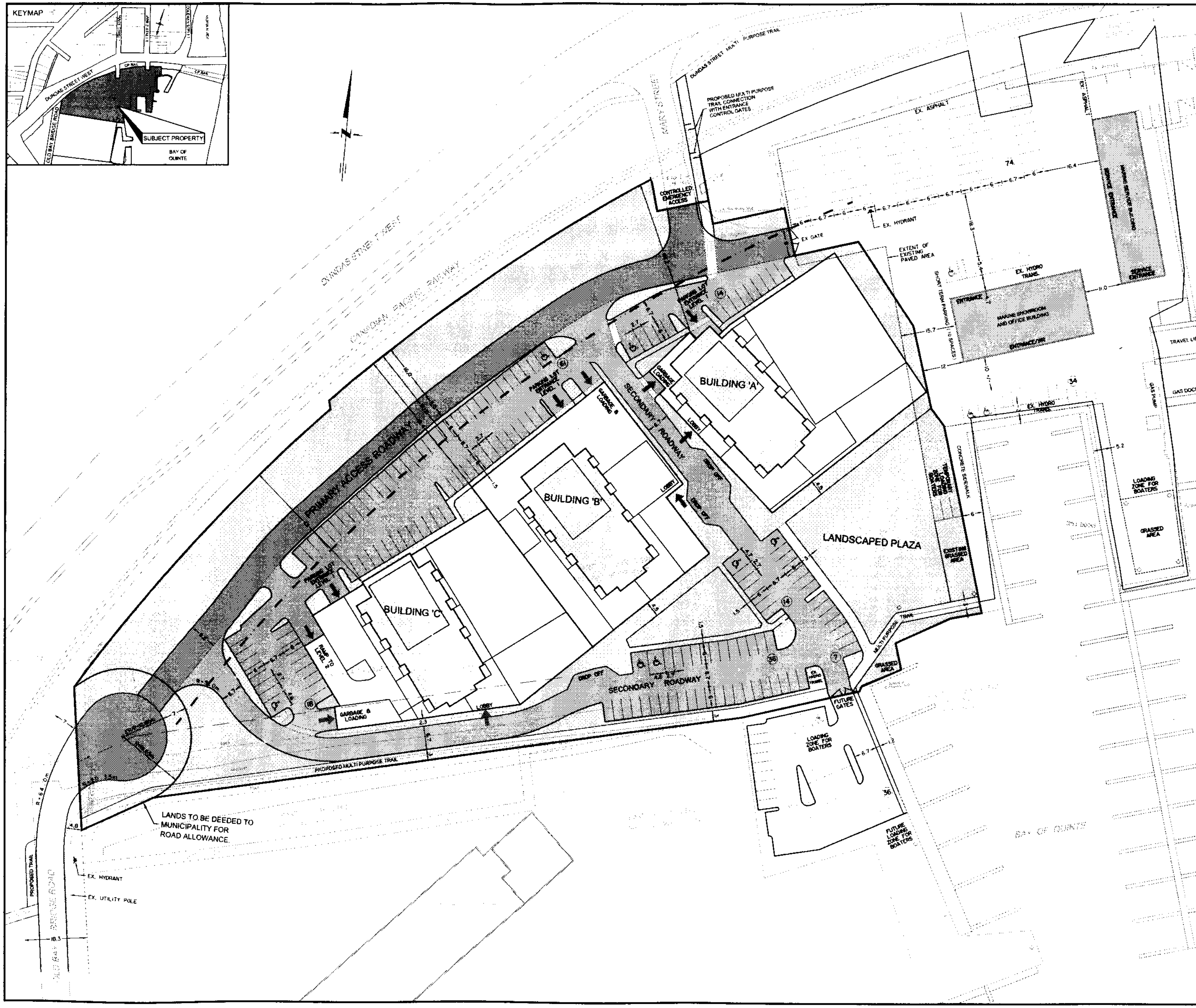
Access to the second and third floors is by stairwells along the perimeter of the building. A balcony along the second floor also provides access, primarily for emergencies.

PLEASE NOTE THE SELLER HAS NO FINANCIAL INFORMATION OR HISTORIAL DATA RELATING TO THE OPERTATIONS OF THE MARINA , FITNESS CENTRE, RESTAURANT OR RENTAL BUILDINGS.



SCHEDULE A - ORIGINAL SITE PLAN


PACI
EXISTING
PARKING
EX. BELL DUCT
EX. UNDERGROUND HYDRO
SAN. FORCEMAIN
BOAT STORAGE
ELECTROLAB
PUBLIC WASHROOMS
STORAGE
INDOOR TENNIS
18 CARS
EX. 8" ø WATERMAIN
PARKING
HARBOUR CLUB SPORTS COMPLEX
EX. BELL DUCT
SAN.
EX. 4" ø SANITARY SEWER
PARKING 9 CARS
PARKING
126 CARS
PROPERTY LIMITS
(5 new spots)
N 73°13'E
166.47'
N 73°47'E
87.11'
PRIVATE PARKING
EXISTING RESIDENCE
86.15'
N 12° 49' W
PROPOSED 10 UNIT COMMERCIAL
148.50'
CARS
33
N 13°05' W
73°47'E
85.00
30.00
N 30°35 E
N 72°32' E
106.00
24'
DITCH
No Grass on No. E side of home
SCALE: 1:750
INED BY MORCH MARINE LTD.
BRIDGE ROAD.



SITE PLAN
EAST MARINA INVESTMENTS LTD.

PARTS 1, 2, 3, 5, PLAN 21R-7426
PART 1, PLAN 21R-10783
LOTS 4, 5, 6, 7 & 8, NORTH SIDE OF WATER STREET
LOTS 5, 6, 7, 8, 9, 10 & 11, SOUTH SIDE WATER STREET
LOTS 1, 2, 3 & 4, SOUTH SIDE DUNDAS STREET
PART OF ANN STREET, WATER STREET AND
JAMES STREET (CLOSED)
REGISTERED PLAN No 14,
PART OF LOTS 4, 5, 6 & 7
REGISTERED PLAN 9
PART OF LOTS A & B, LOTS C & D, SOUTH SIDE DUNDAS
STREET, WEST SIDE ANN STREET
PART OF LOT 77
HASLETT'S PLAN
PART OF WATER LOT IN FRONT OF LOT 3, CONCESSION 1
TOWNSHIP OF THURLOW
NOW IN THE CITY OF BELLEVILLE
COUNT OF HASTINGS

SCALE = 1:500 METRIC

LAND USE SUMMARY LAND USE	AREA(m^2) WITHIN DEVELOPMENT LANDS
CRATES MARINA BUILDINGS AND LAND	10996.9
CONDOMINIUM BUILDINGS AND PARKING PODIUMS	8978.2
PLAZA AND LANDSCAPED AREAS	7985.3
MULTI PURPOSE TRAIL AND PEDESTRIAN WALKWAYS	2641.8
PROPOSED INTERNAL ROADS AND SURFACE PARKING	9308.8
SITE TOTAL	39911.0m^2

DEVELOPMENT SUMMARY

	BLDG A	BLDG B	BLDG C	TOTAL
RESIDENTIAL UNITS (3-15 STOREY BUILDINGS)	102	98	102	302
RETAIL - GROUND FLOOR (SQUARE METERS)	704.7	855.1	495	2054.8
MARINA BERTHS/SLIPS	EXISTING 175		FUTURE 25	200

PARKING SUMMARY

	PARKING SPACES
CONDOMINIUM DECK PARKING	314
CONDOMINIUM VISITOR SURFACE PARKING	75
RETAIL SURFACE PARKING - 1 PER 28 sq. m.	75
CRATES MARINA SURFACE PARKING	144 (+30 FUTURE)
TOTAL PARKING	638

LEGEND

RESIDENTIAL DEVELOPMENT BOUNDARY
30.0m SETBACK FROM C.P.R. PROPERTY
TEMPORARY LOADING AREA ASPHALT TO BE REMOVED AND LANDSCAPED WITH GRASS

DRAWN BY: L.B. | CHECKED BY: R.F.A. | DATE: AUGUST 25, 2014 | SCALE: 1:500

METRIC NOTE: DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

TOPOGRAPHIC SURVEY PROVIDED BY WATSON LAND SURVEYORS LTD.

SEPTEMBER 8 2014

JOB No. 370-SP

RFA
211 Dundas Street East, Suite 202,
Belleville, Ontario, K8N 1E2

SCHEDULE B - PROPOSED SITE PLAN

AREA ANALYSIS - BY BUILDING

	BUILDING A			BUILDING B			BUILDING C			TOTAL
	2BD	3BD	TOTAL	2BD	3BD	TOTAL	2BD	3BD	TOTAL	
Level 1										
Level 2										
Level 3	5	1	6		2	2	5	1	6	14
Level 4	6	2	8	5	3	8	6	2	8	24
Level 5	6	2	8	5	3	8	6	2	8	24
Level 6	6	2	8	5	3	8	6	2	8	24
Level 7	6	2	8	5	3	8	6	2	8	24
Level 8	6	2	8	5	3	8	6	2	8	24
Level 9	6	2	8	5	3	8	6	2	8	24
Level 10	6	2	8	5	3	8	6	2	8	24
Level 11	6	2	8	5	3	8	6	2	8	24
Level 12	6	2	8	5	3	8	6	2	8	24
Level 13	6	2	8	5	3	8	6	2	8	24
Level 14	6	2	8	5	3	8	6	2	8	24
Level 15	6	2	8	5	3	8	6	2	8	24
TOTAL RES.	**77**	**25**	**102**	**60**	**38**	**98**	**77**	**25**	**102**	**302**
TOTAL NON RES.										
GRAND TOTAL										

	BUILDING A				BUILDING B				BUILDING C				TOTAL	
	NON-RESIDENTIAL GROSS FLOOR AREA		RESIDENTIAL GROSS FLOOR AREA		NON-RESIDENTIAL GROSS FLOOR AREA		RESIDENTIAL GROSS FLOOR AREA		NON-RESIDENTIAL GROSS FLOOR AREA		RESIDENTIAL GROSS FLOOR AREA			
Level 1	704.7 m²	7,585 SF	427.7 m²	4,604 SF	855.1 m²	9,204 SF	596.9 m²	6,425 SF	495 m²	5,328 SF	695.7 m²	7,489 SF		
Level 2			60 m²	646 SF			73.3 m²	789 SF			92 m²	990 SF		
Level 3			878.9 m²	9,461 SF			622 m²	6,695 SF			875.9 m²	9,428 SF		
Level 4			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 5			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 6			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 7			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 8			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 9			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 10			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 11			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 12			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 13			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 14			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
Level 15			878.8 m²	9,459 SF			898.1 m²	9,667 SF			878.4 m²	9,456 SF		
TOTAL RES.			**11,912.3 m²**	**128,223 SF**			**12,068.7 m²**	**129,[illegible]07 SF**			**12,204.9 m²**	**131,373 SF**	**36,186 m²**	**389,502 S**
TOTAL NON RES.	**704.7 m²**	**7,585 SF**			**855.1 m²**	**9,204 SF**			**495 m²**	**5,328 SF**			**2,054.8 m²**	**22,118 SF**
GRAND TOTAL													**38,240.8 m²**	**411,620 SF**

NOTE 1: GROSS FLOOR AREA EXCLUDES PARKING GARAGE, PORCH, VERANDA, OR SUN ROOM PER BELLEVILLE ZONING BY-LAW.

AVERAGE RESIDENTIAL UNIT SIZE

99.2 m²	1,068 SF

PARKING PROVIDED

Building	Formulae	Parking QTY.
Building A Residents'	1.03 per unit	105
Building B Residents'	1.00 per unit	98
Building C Residents'	1.09 per unit	111
Total Residential		**314**
Visitor		75
Retail		75
TOTAL PARKING PROVIDED		**464**

PROJECT STATISTICS

&Co Sweeny &Co Architects Inc.

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16		AG LL	CR	**A101**

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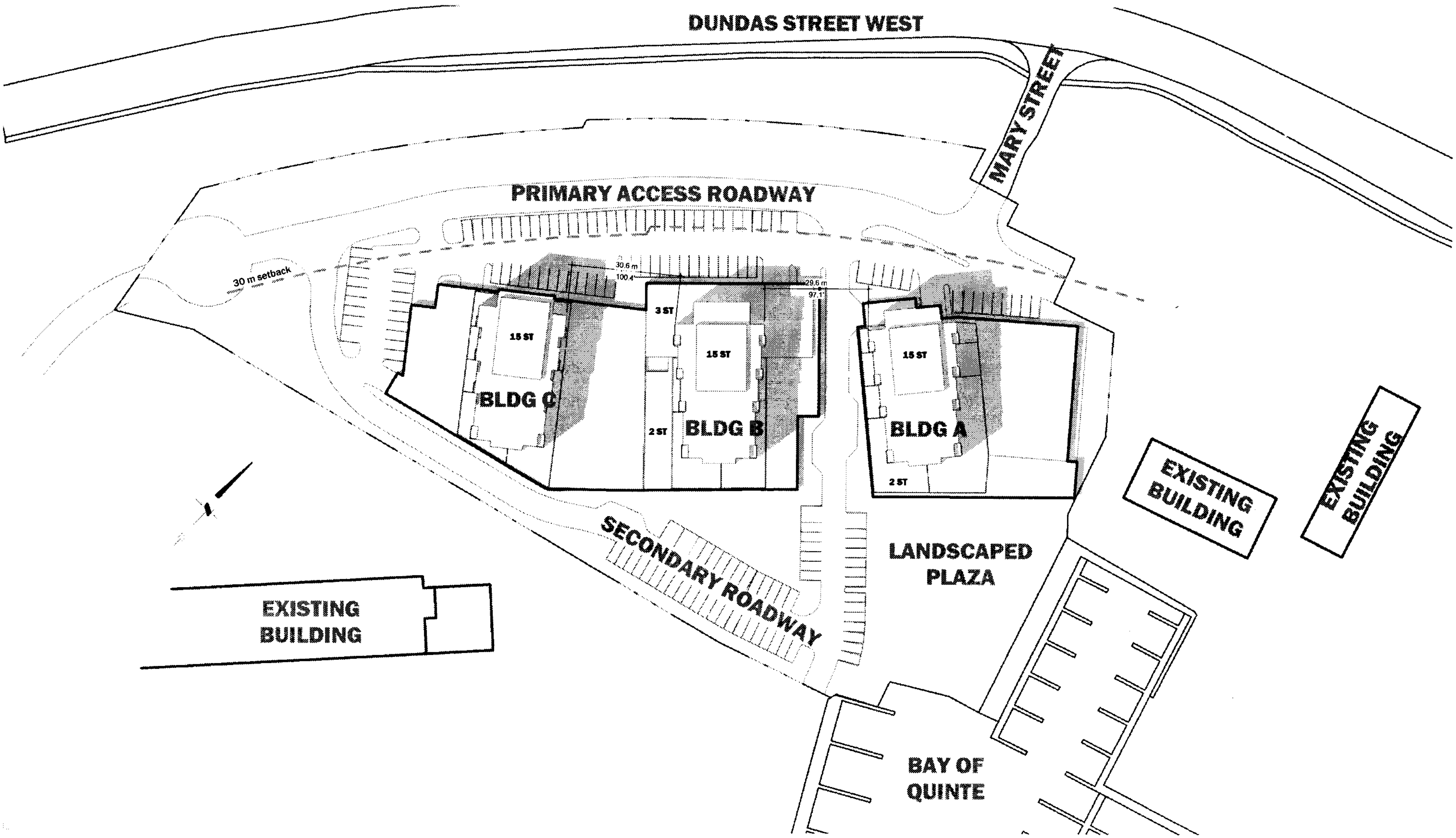

SITE PLAN

&Co Sweeny &Co Architects Inc.

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16	1 : 1000	AG	CR	A102

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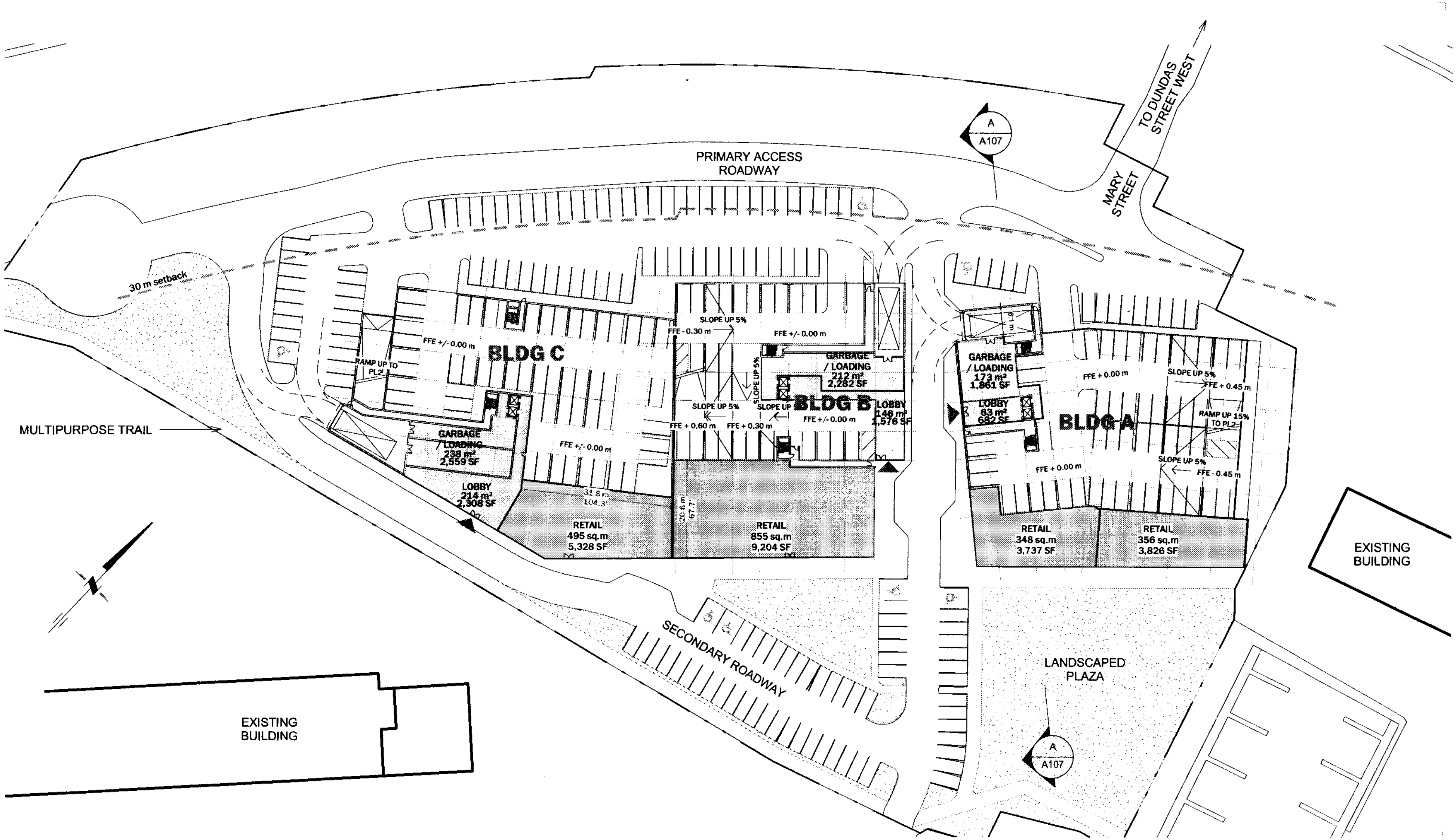

LEVEL 1

&Co **Sweeny &Co Architects Inc.**

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16	1 : 750	AG	CR	**A103**

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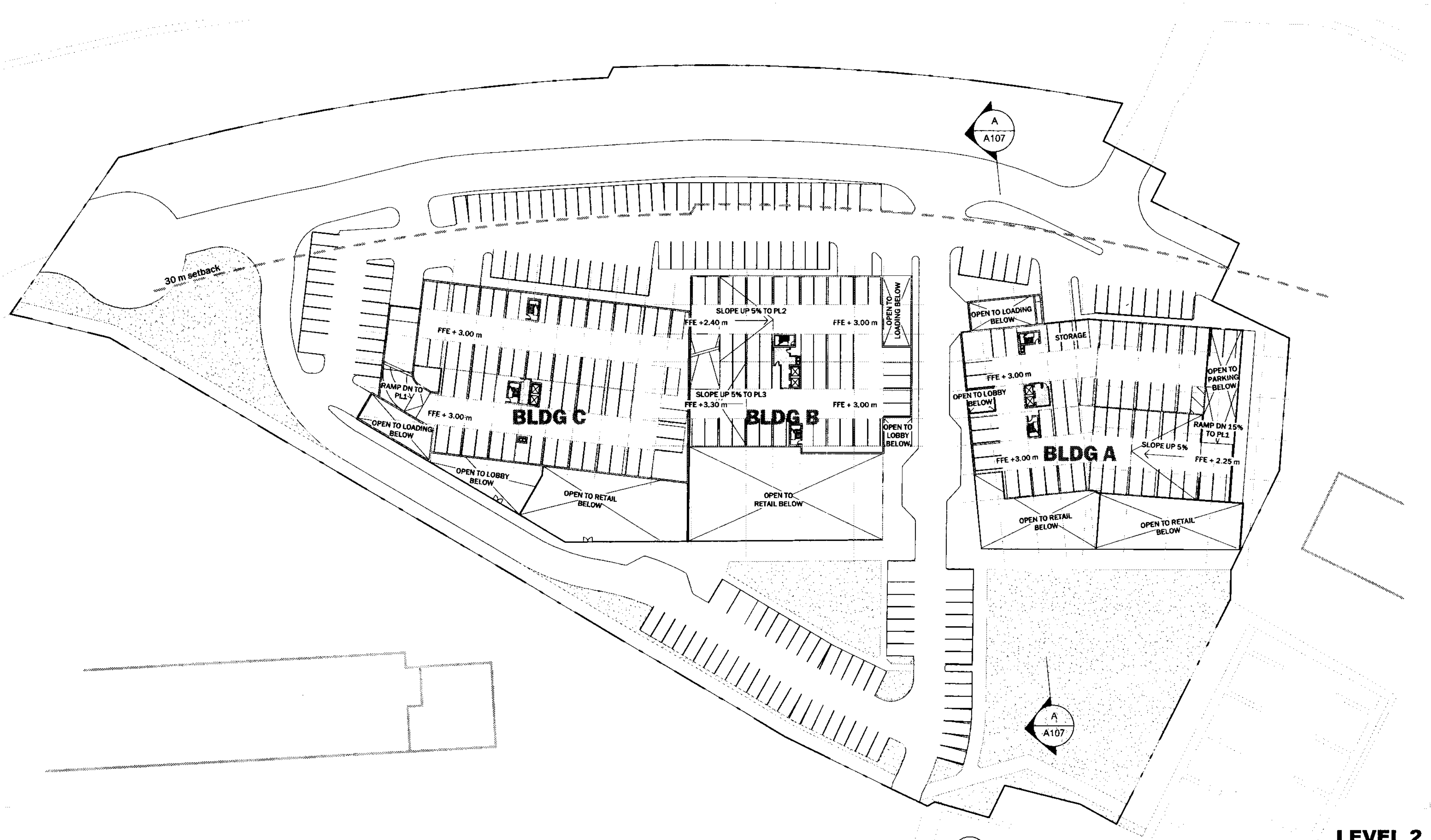

LEVEL 2

EAST MARINA PROJECT - BELLEVILLE

&Co Sweeny &Co Architects Inc.

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16	1 : 750	AG	CR	A104

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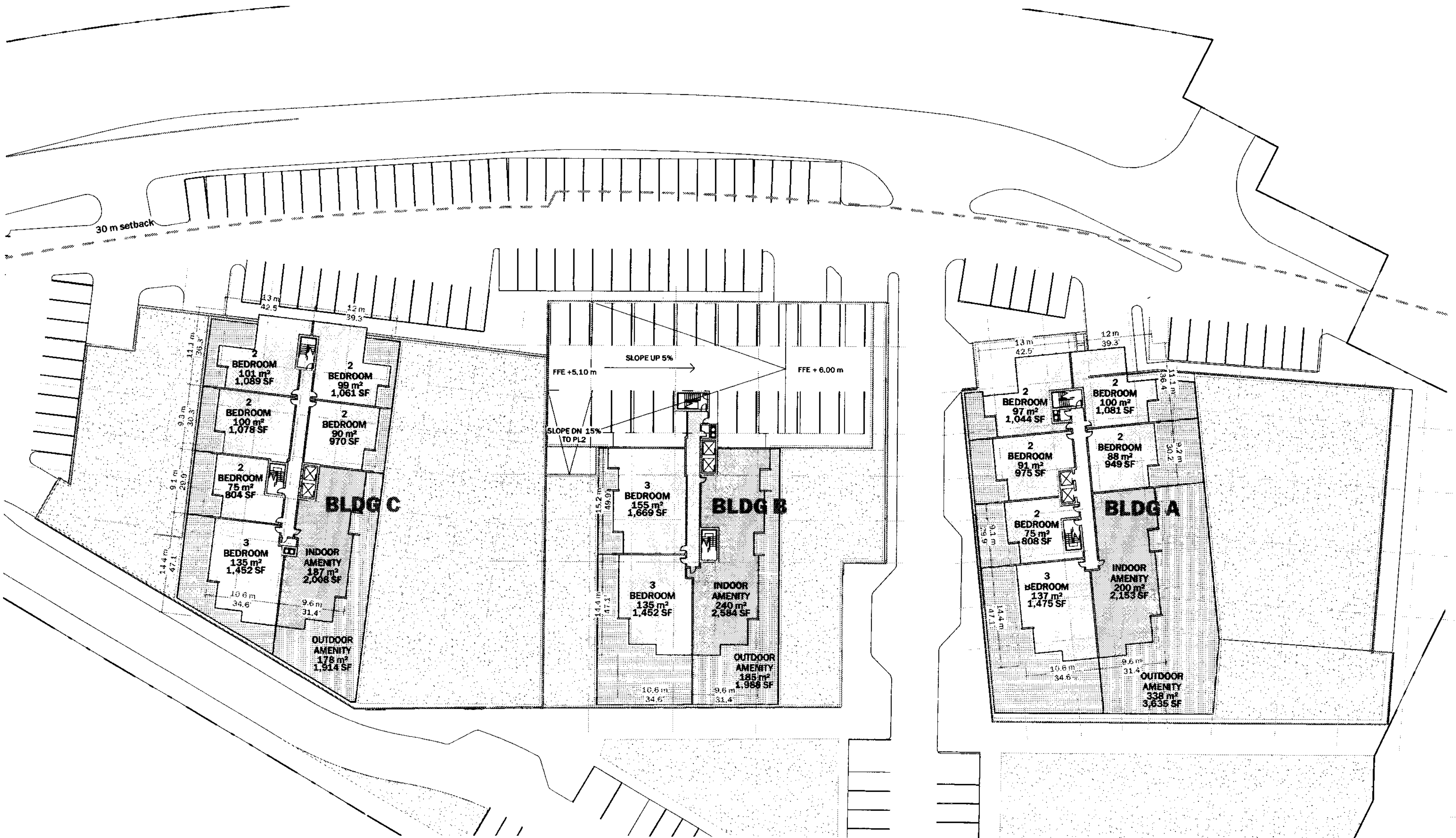

LEVEL 3

EAST MARINA PROJECT - BELLEVILLE

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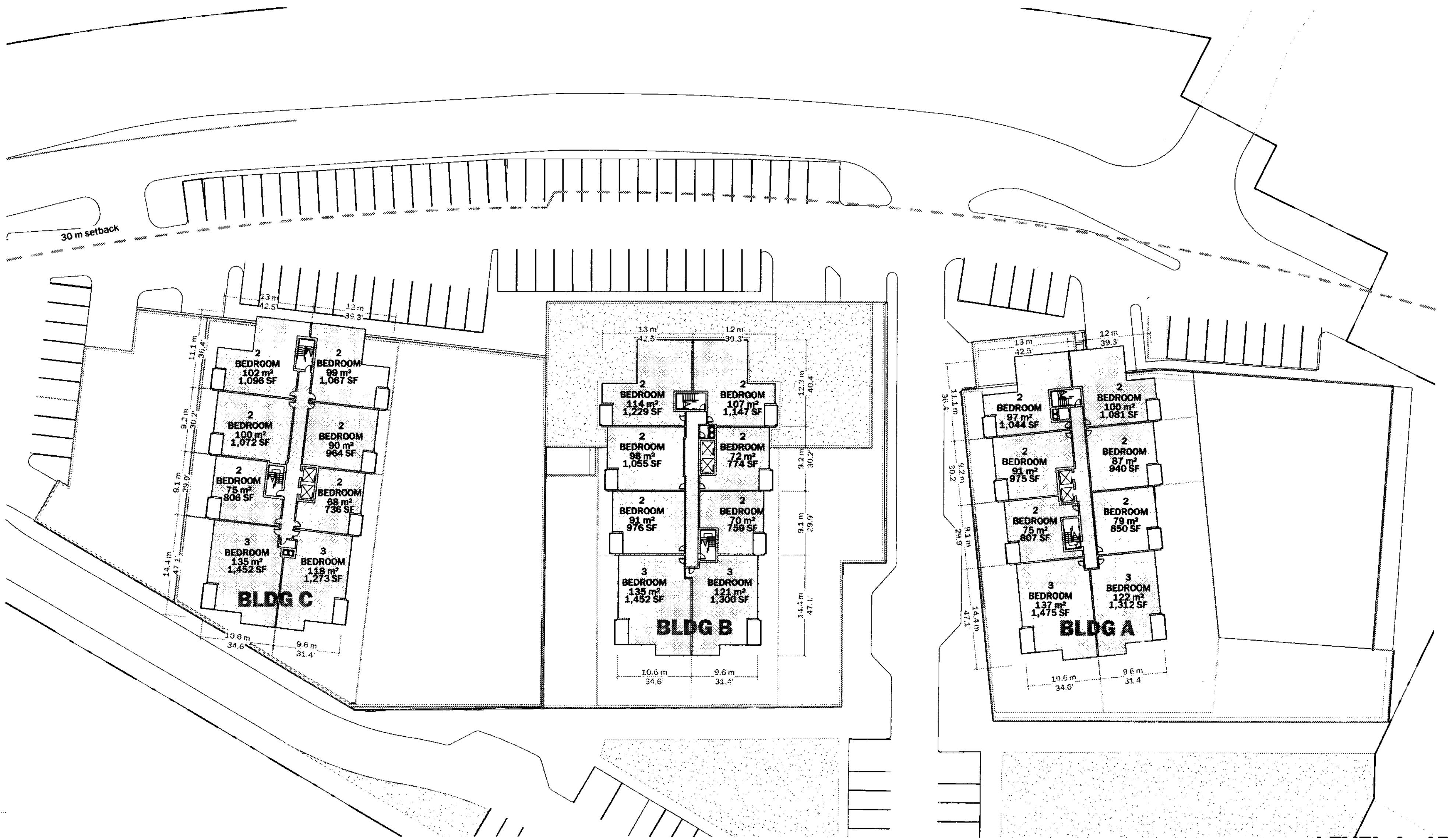
PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16	1 : 500	AG	CR	A105



LEVEL 4 - 15

&Co Sweeny &Co Architects Inc.

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16	1 : 500	AG	CR	A106

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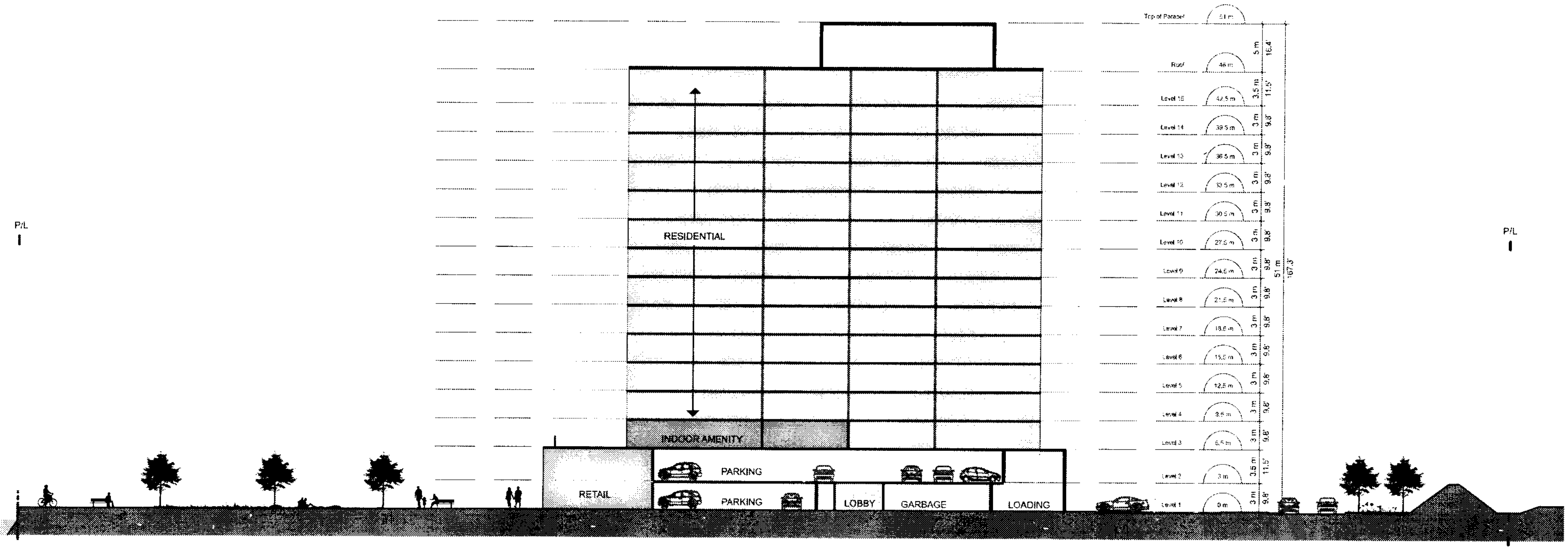

BUILDING A - SECTION A - A

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.08.25	1 : 400	AG	CR	A107

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VIEW 1

&Co Sweeny &Co Architects Inc.	**EAST MARINA PROJECT - BELLEVILLE**					
PROJ. No.	DATE	SCALE	DRAWN	CHECKED		DWG No.
1326	2014.09.16		HH	CR		**A108**

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VIEW 2

&Co Sweeny &Co Architects Inc.	**EAST MARINA PROJECT - BELLEVILLE**					
	PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
	1326	2014.09.16		HH	CR	**A109**

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VIEW 3

&Co **Sweeny &Co Architects Inc.**

EAST MARINA PROJECT - BELLEVILLE

PROJ. No.	DATE	SCALE	DRAWN	CHECKED	DWG No.
1326	2014.09.16		HH	CR	**A110**

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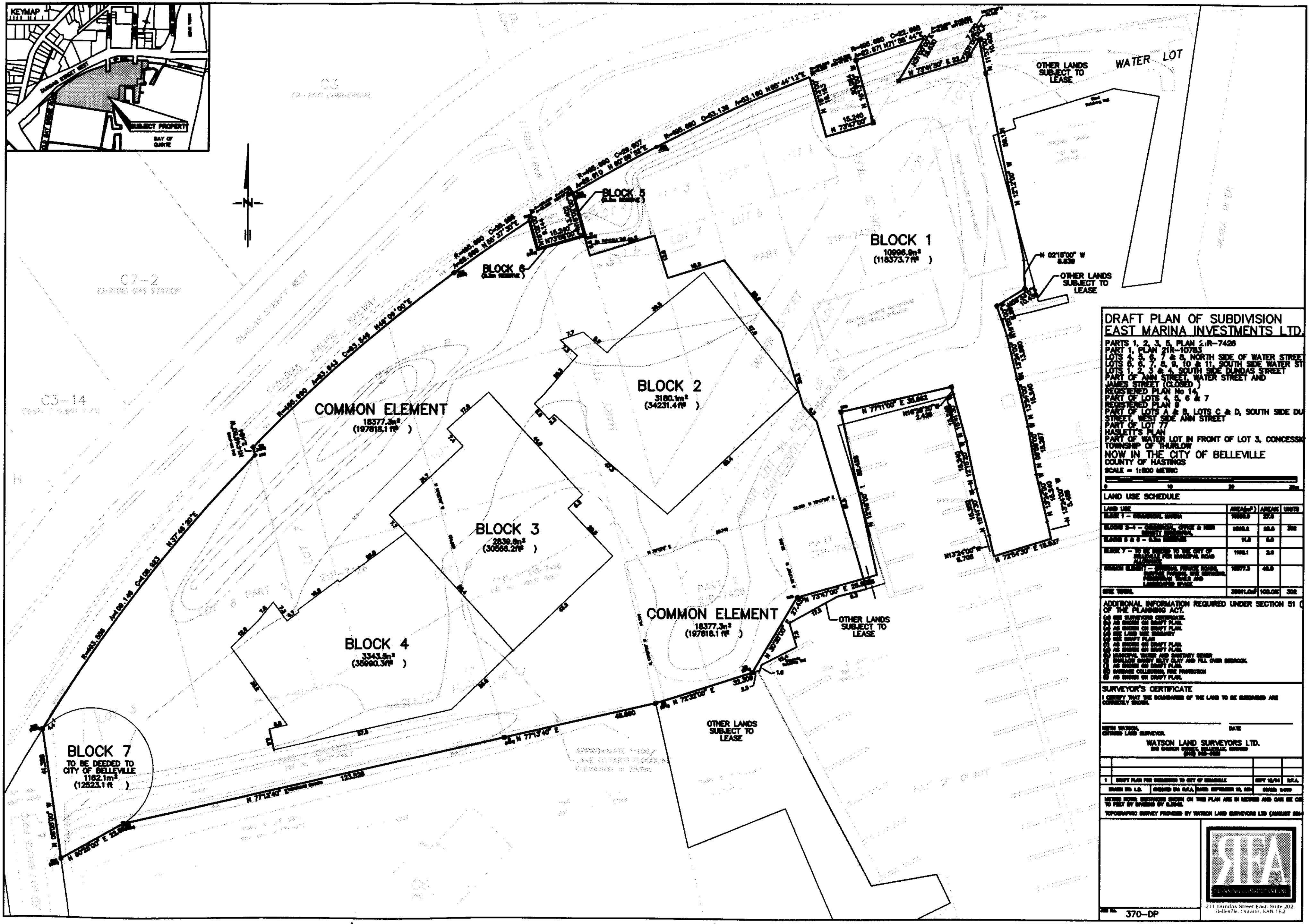


SCHEDULE C - PROPOSED PLAN OF SUBDIVISION

Sanitary Sewer Flow Calculations

Project No.: 13100

East Marina Investments Ltd.

25 Dundas Street West
City of Belleville

A. Estimated Sanitary Sewer Flow Calculations
(Original Site Development Area = 3.77 ha.)

(i) Commercial/Retail

Building Areas (GFA): (See Prospectus)	Building 1 (Marina)	- 0.0605 ha. (6,510 sf)
	Building 2 (Marina)	- 0.0680 ha. (7,320 sf)
	Building 4 (3 Storey)	- 0.1795 ha. (19,320 sf)
	Total GFA:	0.3080 ha. (33,150 sf)

Peak Sewage Flow Q @ 5.0 $l/day/m^2$ (MOE Guideline)

Q = (5.0) (3080) l/day
= 15,400 l/day
= 0.18 l/s

(ii) Restaurant (Portion of Building)

Total GFA = 0.3313 ha. (35,657 sf)

Assumed Restaurant Area @ 60% = 0.1987 ha.

Peak Sewage Flow Q @ 5.0 $l/day/m^2$ (MOE Guideline)

Q = (5.0) (1987) l/day
= 9,935 l/day
= 0.11 l/s

(iii) Fitness Club (Portion of Building 3)

Program classes, staff daycare @ 165 people
Peak Per Capita Sewage Flow = (165) (1815 l/cap/day)
= 299,475 l/day
= 3.47 l/s

Estimated Sanitary Sewer Flow Calculations (Original Site) – Cont'd

(iv) Residential

Building 4	4 apts @ 2.5 people	=	10 people
Building 5		=	3 people
	Sub-Total:		13 people
Peak per Capita Sewage Flow		=	(13) (1815 l/cap/day)
		=	23,595 l/day
		=	0.27 l/s

(v) Infiltration Flow Q = (3.77) (0.28)

= 1.06 l/s

(vi) Total Estimated Site Flow

Total = 0.18 + 0.11 + 3.47 + 0.27 + 1.06

= 5.09 l/s

= 440 cu. m/day

July 29, 2014

Project No.: 13100

East Marina Investments Ltd.

25 Dundas Street West
City of Belleville

B. Estimated Sanitary Sewer Flow Calculations
(Proposed Site Development A=3.77 ha.)

(1) Retail

Retail Area A	= 0.2052 ha. (22,085 sf)
Peak Sewage Flow Q	= (2,052) (5.0 l/day/m^2)
	= 10,260 l/day
	= 0.12 l/s

(ii) High Density Residential (50 U.P.A.)

Sites A, B & C Area A	= 2.10 ha. (303 Units)
Population (P) @ 2.5 persons/unit	
P	= (2.5) (303)
	= 758 persons
Per Capita Sewage Flow	= (758) (350 l/cap/day)
	= 265,300 l/day
	= 3.07 l/s
Harmon Peaking Factor (M)	$= 1 + \dfrac{14}{4+ (P/1000)^{1/2}}$
	$= 1 + \dfrac{14}{4+ (0.75)^{1/2}}$
	= 1 + 2.87
	= 3.87
Peak Residential Sewage Flow Q	= (3.07) (3.87)
	= 11.88 l/s

Estimated Sanitary Sewer Flow Calculations (Proposed Site) – cont'd

25 Dundas Street West, Belleville

(iii) Infiltration Flow Q = (3.77) (0.28) Assuming Site Area = 3.77 ha.

= 1.06 l/s

(iv) Existing Sewage Flow (Buildings 1 & 2)

Building Areas (GFA) (See Prospectus)	= Building 1 – 0.0605 ha. (6,510 sf) (Marina)
	= Building 2 – 0.0680 ha. (7,320 sf) (Marina)
Total GFA:	0.1285 ha. (13,830 sf)

Peak Sewage Flow Q @ 5.0 l/day/m^2 (MOE Guideline)

= (5.0) (1,285) l/day

= 6,425 l/day

= 0.07 l/s

(v) Total Estimated Site Flow = 0.12 + 11.88 + 1.06 + 0.07 l/s

= 13.13 l/s

= 1,135 cu. m/day

July 29, 2014

Project No.: 13100

East Marina Investments Ltd.

25 Dundas Street West
City of Belleville

C. Existing Sanitary Sewer Capacity

(i) Ex. sanitary sewer 900 mm dia. @ 0.20% on Dundas Street

Q_{cap} = 1,283 l/s

(ii) Estimated Site Flow (Original Site Development)

% capacity = 5.09/1,283

= 0.40%

(iii) Estimated Site Flow (Proposed Site Development)

% capacity = 13.13/1,283

= 1.02%

D. Estimated Additional Flow to Treatment Plant

Estimate for Original Conditions = 440 cu.m/day

Estimate for Proposed Development = 1,135 cu.m/day

Net Increase = 695 cu.m/day

July 29, 2014

SANITARY SEWER DESIGN CRITERIA

EFFECTIVE JUNE 1, 2000

1. Population Density - 20 persons/acre - Residential (50 persons/hectare)
   2.5 persons/unit - Apartment - 50 U.P.A.
   3.0 persons/lot - Single Family Residential

2. Existing Residential

   A) Low Density Residential (Less Than 50 U.P.A.)

      (i) Per Capita Sewage Flow - Average 150 gal/day (680 l/day)
      - Peak 400 gal/day (1815 l/day)

      or Design Flow Rate - 0.015 cfs/acre (1.05 l/s / ha)

      (ii) Infiltration Rate - 400 gal/acre/day (.052 l/ha.s)

   B) High Density Residential (greater than 50 U.P.A.)

      Apartment - 50 U.P.A. - 0.094 cfs/acre (6.58 l/s/ha) includes infiltration

3. New Residential

   A) Low Density Residential (less than 50 U.P.A.)

      (i) Per Capita Sewage Flow - use MOE criteria of 350 l/cap.day
      and Harmon Formula for Peaking Factor

      (ii) Infiltration Rate - 0.28 l/ha.s

   B) High Density Residential (greater than 50 U.P.A.)

      Apartment - 50 U.P.A. - 0.094 cfs/acre (6.58 l/s/ha) includes infiltration

4. New and Existing Industrial and Commercial Flows - 0.015 cfs/acre (1.05 l/s/ha) includes infiltration

5. New Trunk Sewer Systems (to be used for design areas in excess of 50 ha)

   Design Flow Rate - 0.012 cfs/acre (0.84 l/s/ha) includes infiltration

6. Private Drain Connections - footing drainage and roof water leaders are not permitted to be connected.

   NOTE: Gallons are in Imperial Gallons

$$Q\,(d) = \frac{PqM}{86.4} + IA$$

where

Q (d)	=	Peak domestic sewage flow (including extraneous flows) in L/s
P	=	Design population, in thousands
q	=	Average daily per capita domestic flow in L/cap·d (exclusive of extraneous flows)
I	=	Unit of peak extraneous flow, in L/(ha·s); applicable references should be consulted for values
A	=	Gross tributary area in hectares
M	=	Peaking factor (as determined from Harmon or Babbitt Formula)

Harmon Formula

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

Babbitt Formula

$$M = \frac{5}{P^{0.2}}$$

Note that the minimum permissible peaking factor M is 2.0.

5.5.2.2 Commercial and Institutional Sewage Flows

The sewage flows from commercial and institutional establishments vary greatly with the type of water-using facilities present in the development, the population at the facilities, the presence of water metering and the extent of extraneous flows entering the sewers.

Institutional flows should be computed for each individual case based on historical records, when available. Where no records are available, the unit values in Table 5-3 should be used. For commercial and tourist areas, a minimum allowance of 28 m^3/(ha·d) [2,993 US gal/(ac·d)] average flow should be used in the absence of reliable flow data.

For individual commercial and institutional uses the sewage flow rates shown in Table 5-3 are commonly used for design.

The peaking factors applicable for sewage flows from individual establishments will be similar to the relative peak water usage rates. The designer should refer to the ministry *Design Guidelines for Drinking Water Systems* for details.

Table 5-3 - Common Sewage Flow Rates for Commercial and Institutional Uses

Use	Unit Sewage Flow[1] SI (L/d)	Unit Sewage Flow[1] US (US gal/d)	Flow Unit per
Shopping Centre	2.5-5.0	0.7-1.3	based on total floor area (m^2 and ft^2)
Hospitals	900-1800	237-475	bed
Schools	70-140	18.5-37	student
Travel Trailer Parks	340 800	90 211	space (min. without water hook-ups) space (min. with indiv. water hook-ups)
Campgrounds	225-570	59-150	campsite
Mobile Home Parks	1000	264	parking space
Motels	150-200	40-53	bed space
Hotels	225	59	bed space

Note:
1. Unit sewage flow rates exclusive of extraneous flows.

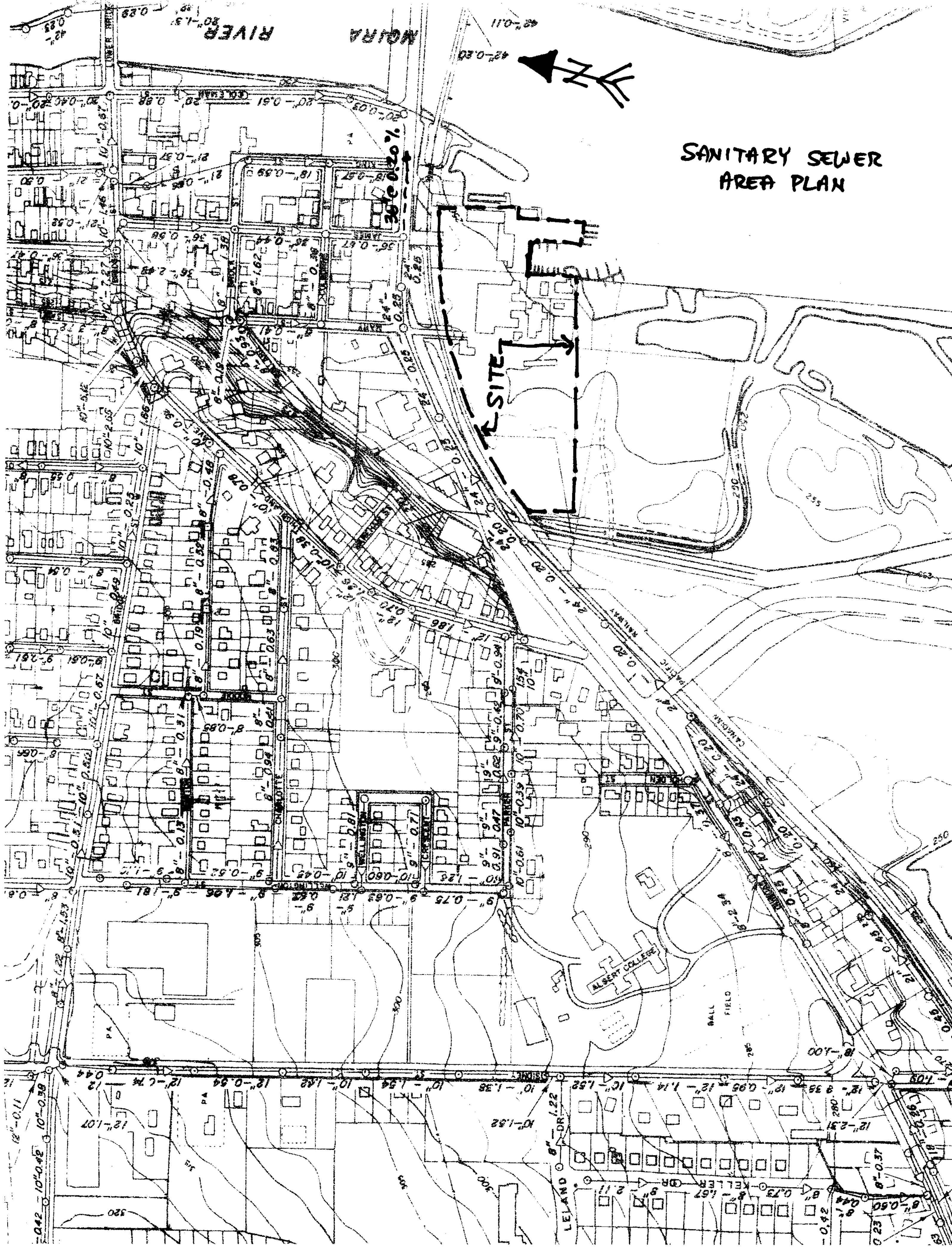
5.5.2.3 Industrial Sewage Flows

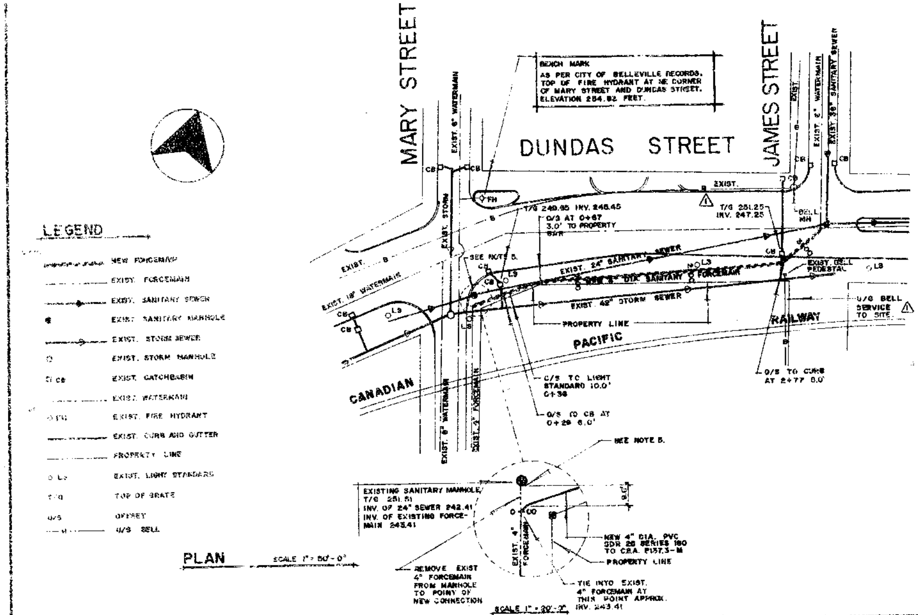
Peak sewage flow rates from industrial areas vary greatly with the extent of area development, the types of industries present, the provision of in-plant effluent treatment and recycle/re-use or rate of flow controls, the presence of cooling waters in the discharge and other factors. Due to the occasional presence of individual industrial water supplies, the rates of water supply from municipal systems into industrial areas will not always be indicative of the sanitary sewage flows to be expected. The discharge of cooling water from municipal supplies into storm sewers or *surface water* courses may result in lower flows in sanitary sewers than would be expected based on municipal water usage.

The calculation of design average and peak sewage flow rates for industrial areas is industry/process specific and may be difficult to predict accurately. Improving flow prediction may include better monitoring of industries present in industrial areas. In this way, a reasonable allowance can be made for peak industrial sewage flows for an area and by flow monitoring allowances can be better maintained. Industries with the potential to discharge sewage at higher than the accepted rates may be required to provide flow equalization with discharge at off-peak discharge periods.

5.5.2.4 Foundation Drainage

It is essential that foundation drainage be directed to storm sewer systems or in accordance with local municipal best management practices taking into account on-site drainage/infiltration conditions. Since sanitary sewers are not designed to accept these flows (i.e., rainwater leaders and/or foundation drains), serious damage/problems may result, such as cracking of basement

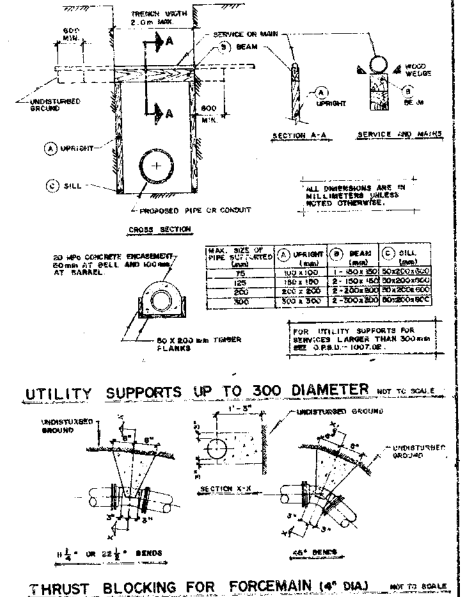

SANITARY SEWER
AREA PLAN
SITE
MOIRA RIVER
CANADIAN PACIFIC RAILWAY
ALBERT COLLEGE
BALL FIELD
KELLER DR
LELAND DR
36" @ 0.20%



GENERAL NOTES

1. EXISTING UTILITIES HAVE BEEN DRAWN ACCORDING TO CITY OF BELLEVILLE PLAN NO. N-19-65 AS DRAWN BY DAMAS AND SMITH LIMITED CONSULTING ENGINEERS (STA 115+00 TO STA. 130+00). LOCATIONS ARE SHOWN FOR CONTRACTORS INFORMATION ONLY AND THE ENGINEER DOES NOT GUARANTEE THEIR ACCURACY OR COMPLETENESS.
2. ALL UNDERGROUND AND ABOVEGROUND WORK IS TO BE DONE IN ACCORDANCE WITH CURRENT CITY PLANS, STANDARDS AND SPECIFICATIONS INCLUDING THE FOLLOWING: STANDARD TRENCH DETAIL - SPEC. NO. M-16, 600 mm CURB AND GUTTER - SPEC. NO. M-43, ETC.
3. THE OWNER IS TO INSTALL AND MAINTAIN THE NEW FORCEMAIN FROM MARY STREET TO JAMES STREET. THE OUTLET OF THE FORCEMAIN AT JAMES STREET IS TO POINT TOWARDS THE OUTLET SEWER TO FACILITATE THE FLOW.
4. NO BLASTING IS PERMITTED ON THE CITY RIGHT-OF-WAY.
5. THE EXISTING SEWER CONNECTION IS TO BE SECURELY CAPPED OFF WITH A MECHANICAL CAP AT THE MANHOLE TO PREVENT INFILTRATION INTO THE MAIN SEWER.
6. THE REINSTATEMENT OF ASPHALT ROADWAYS AND CURBS ON THE CITY ROAD ALLOWANCE IS TO BE DONE BY THE CITY OF BELLEVILLE AT THE OWNER'S EXPENSE.
7. EXISTING SUBDRAINS ALONG THE CURB WHICH ARE DISTURBED ARE TO BE RESTORED TO THEIR ORIGINAL CONDITION.

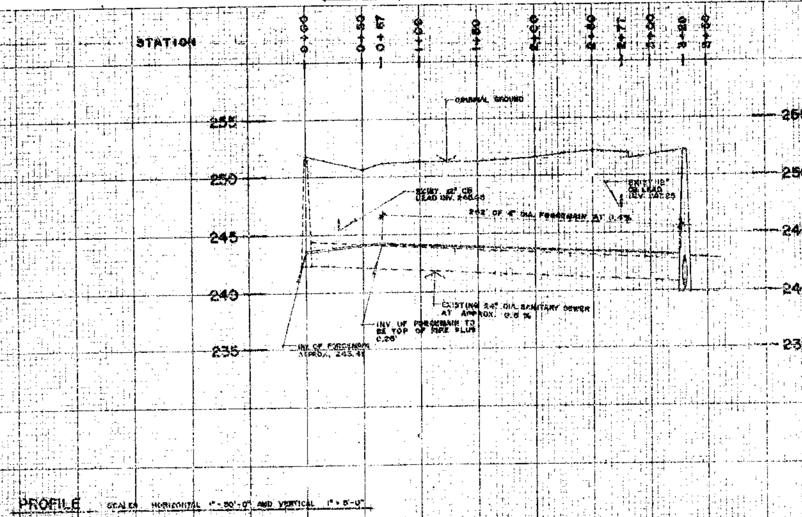
CITY OF BELLEVILLE ENGINEERING DEPARTMENT
APPROVED



ALL DIMENSIONS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.

MAX. SIZE OF PIPE SUPPORTED (mm)	(A) UPRIGHT (mm)	(B) BEAM (mm)	(C) SILL (mm)
75	100 x 100	1 - 150 x 150	50x200x600
125	150 x 150	2 - 150 x 150	50x200x600
200	200 x 200	2 - 200 x 200	50x200x600
300	300 x 300	2 - 300 x 300	50x200x600

FOR UTILITY SUPPORTS FOR SERVICES LARGER THAN 300 mm SEE O.P.S.D. - 1007.02.



NOTE:
EX. 100mm Ø SANITARY FORCEMAIN
FROM MARINA LANDS CONNECTS TO
EX. SAN. MH. @ JAMES ST. TO OUTLET
TO EX. 900mm Ø SAN.

Baxter-Langman Engineering Ltd.
Phone 1 (613) 966-7155
223 COLEMAN STREET
BELLEVILLE, ONTARIO
K8P 3H8

NOTE:
This drawing and the design herein is the property of Baxter-Langman Engineering Ltd. and must be returned to them upon completion of work.
Contractor, construction manager, project manager or the owner, whichever is applicable is solely responsible for the verification of all dimensions. Any discrepancies must be reported to the engineers before construction commences.
All plans, specifications construction documents and the actual construction must adhere to the Ministry of Labour, Fire Department, The Building Code, the Ministry of the Environment and all other governing authorities guidelines.
All work by contractor and subtrades must meet requirements of all authorities noted above.

ENGINEER

D. E. B.

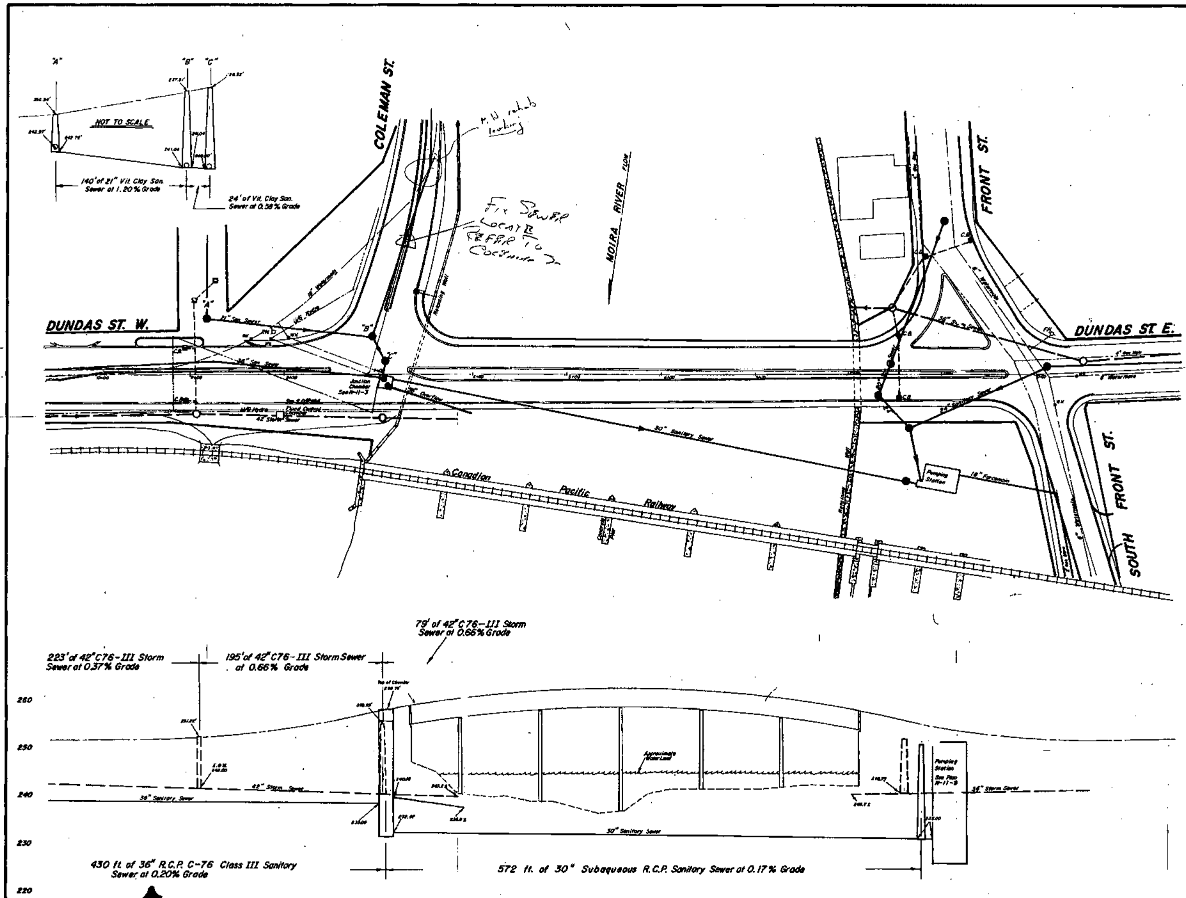
DATE	REVISION	BY
OCT 29 '88	1 AS PER CITY OF BELLEVILLE COMMENTS LETTER DATED OCTOBER 25, 1988.	TJG

CLIENT: MORCH PROPERTIES LIMITED

DOCKSIDE

TITLE: SITE SERVICING FOR 10 UNIT COMMERCIAL COMPLEX - FORCEMAIN

DATE: OCTOBER 1988
SCALE: AS NOTED IMPERIAL
DRAWN BY: TJG
CHECKED BY:
PROJECT NUMBER: 88-14002
DRAWING: C1



NOTE: ↑ OUTLET SAN. 900mmØ @ 0.20%

ROAD GRADE	
SANITARY SEWER INVERT	
STORM SEWER INVERT	
CHAINAGE	

CITY OF BELLEVILLE
DEPT. OF PUBLIC WORKS

DUNDAS ST. WEST

DRAWN BY: S.A. STORMS	DATE:
DESIGN ENG:	SCALES: HOR. 1" = 50' VERT. 1" = 10'
CITY ENG: J. ANGELO P. ENG.	PLAN No. N-19-65-3

Water Demand Calculations

Project No.: #13100

Water Demand Analysis

25 Dundas Street, Belleville

The following calculations estimate the Fire Flow required to provide satisfactory protection to the building based on information found in the Fire Underwriters Survey - Water Supply for Public Fire Protection (1999). This added with standard domestic flow will provide the total water demand.

a) Proposed Development

1. Fire Flow Calculation

$F = 220\ C\ \sqrt{A}$ Where:

F = Fire flow in L/min
C = Construction type coefficient
0.6 Fire-resistive
A = Total floor area in m²
(excluding basements, includes garages)

Floor	Area m²	%	Net m²
Level 1	3775.1	100	3775
Level 2	225.3	25	56
Level 3	2376.8	25	594

Net A =	4425	m²		
F =	220 *	0.6	* √	4425
F =	8781	L/min		
F =	9000	L/min	(Rounded to nearest 1000 L/min)	

2. Occupancy Reduction

25% Reduction for non-combustible

Reduction =	2250	L/min
F=	6750	L/min

3. Sprinkler Reduction

50% Reduction for fully sprinklered

Reduction =	3375	L/min
F=	3375	L/min

4. Separation Charge

Side	Separation	%	
North	45+	0	
South	45+	0	
East	20.1-30m	10	
West	20.1-30m	10	
		20	(To a max. 75%)

20% Separation Charge = 1350 L/min

Project No.: #13100

Water Demand Analysis

25 Dundas Street, Belleville

5. Result Flow

F =	3375	+	1350
F =	4725	L/min	
F =	1248	gpm	

6. Domestic Water Demand

Average Daily Domestic Demand = 450 Lpcd
(=1 L/s per 192 people)

Estimated people P = 755 (302 Units @ 2.5 ppu)

∴ Ave. Daily Domestic Demand = 755 / 192
= 3.93 L/s
= 236 L/min

& Maximum Daily Demand Factor = 2.75 (As per 2006 MOE Design Guidelines for Drinking Water Systems - See Appendix)

∴ Total Maximum Daily Demand = 2.75 x 236
649 L/min

b) Existing Marine Buildings 1 & 2

Total G.F.A. = 0.1285 ha.

∴ Average Daily Demand (ADD) = 55.00 m3/(ha·day)
(as per 2008 MOE Water Demand Guidelines - Heavy Industry)
= 55 m^3/(0.1285 ha day)
= 7.07 m^3/day
= 7070 l/day
= 4.91 l/min.

Use Peak Demand Factor (PF) = 4 (as per 2008 MOE Water Demand Guidelines - Heavy Industry)
∴ Peak Demand = ADD x PF
= (4.91) (4)
= 19.64 l/min.
Use Peak Demand = 20.0 l/min.

7. Total Water Demand

Total = Fire Flow + Domestic Flows (prop. dev. & ex. marina)
= 4725 + 649 + 20
= 5394 L/min
= 1425 gpm

Project No.: #13100

Water Demand Analysis

25 Dundas Street, Belleville

8. Resultant Static Water Pressure

Using the graph of a hydrant flow test performed by the City of Belleville in March 2007 at 48 amd 68 Dundas St. West (see attached), a total water demand of 1425 US gpm. results in a residual water pressure in the municipal watermain of approximately 80 psi at 20 psi (140 kPa) which meets the City of Belleville requirements.

Gmail by Google

John Towle <john.towle.associates@gmail.com>

Re: Marina lands - 25 Dundas Street West, Belleville #13100

Lucciola, Darrin <dlucciola@city.belleville.on.ca> Wed, Sep 3, 2014 at 3:01 PM
To: "John.Towle.Associates@gmail.com" <John.Towle.Associates@gmail.com>

Hi John

Dan is away on holidays this week and has asked me to follow up with you regarding servicing and flows into and near this site. According to previous emails, I believe it has been determined that the 150mm water main no longer services the site. That leaves just the 200mm main as the only service entering the site. I have attached a flow test for the hydrant on the north side of Dundas that does connect to the 450mm water main. On the flow test that particular hydrant is referred to as 48 Dundas St. W. The hydrant referred to as 68 Dundas St. W. is also off of the 450mm main and is the adjacent hydrant to the west. The flow test is from 2007 but the results would be no different today.

Darrin Lucciola

Supervisor Water Distribution

City of Belleville

Environmental and Operational Services

613-966-3651 ext. 2278

Cell: 613-961-9826

dlucciola@city.belleville.on.ca

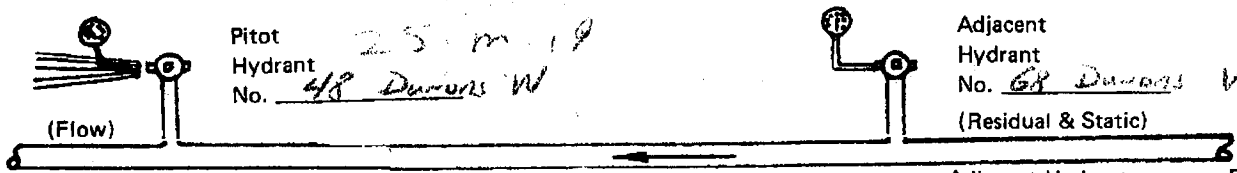
48 Dundas Flow Test.pdf
34K

Routing
White - 1. Op. Mgr. 2. Draft. 3. FF bk.
Pink - File 842
Canary - Originator

Belleville Utilities Commission
459 SIDNEY STREET
P.O. BOX 939
BELLEVILLE, ONT., K8N 5B6
(613) 966-3651

Date: Mar 26/07
Time: 0920
Performed by [illegible]
File: 842

FIRE HYDRANT FLOW TEST



Pitot Hydrant No. 48 Dundas W   25 m 19
(Flow)

Adjacent Hydrant No. 68 Dundas W
(Residual & Static)

Adjacent Hydrant _______ Ft.
Above or Below Pitot Hydrant

Street Name Dundas W

Location on St.
or name of Bldg. ____________

Provide Four Pressure Readings:
Select outlets to give 10 psi
drop at adjacent hydrant if possible

		OUTLETS					
		one - 1"	one - 1 1/8"	one - 1½"	one - 2½"	two - 2½"	
Step One -	Adjacent Hydrant				84	84	psi (static)
Step Two -	Pitot Hydrant				70	46	psi (flow)
Step Three -	Adjacent Hydrant				81	78	psi (residual)
Step Four -	Adjacent Hydrant				84	84	psi (static check)

low with 20 psi residual at adjacent hydrant

$$= \text{measured flow} \left(\frac{\text{available drop}}{\text{test drop}}\right)^{.54}$$

Available drop is static less 20
Test drop is static less residual

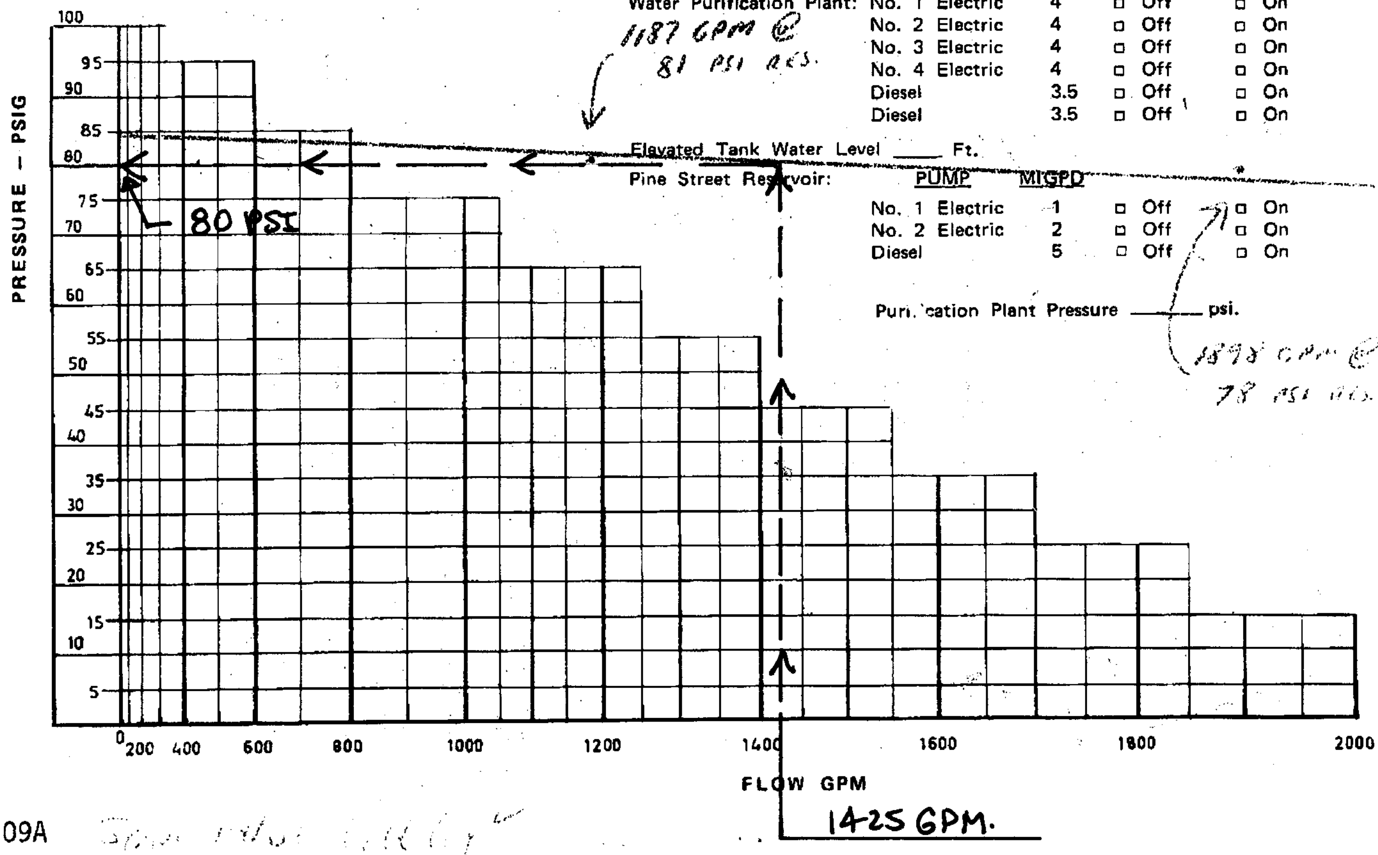
Information below can be obtained at a later date from records at Water Purification Plant.

Water Purification Plant:	PUMP	MIGPD		
	No. 1 Electric	4	□ Off	□ On
	No. 2 Electric	4	□ Off	□ On
	No. 3 Electric	4	□ Off	□ On
	No. 4 Electric	4	□ Off	□ On
	Diesel	3.5	□ Off	□ On
	Diesel	3.5	□ Off	□ On

Elevated Tank Water Level _____ Ft.

Pine Street Reservoir:	PUMP	MIGPD		
	No. 1 Electric	1	□ Off	□ On
	No. 2 Electric	2	□ Off	□ On
	Diesel	5	□ Off	□ On

Purification Plant Pressure _______ psi.



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requirements (*Section 8.4 – Sizing of Storage Facilities* and *Section 10.1.2 – Fire Protection*).

The capacity of the treatment processes should be greater than the highest demand (typically maximum day demand) since allowance is needed for water required for in-plant use and process losses. Depending on the processes in the treatment plant, water may be lost as clarifier ***blowdown*** or ***membrane reject*** streams and treated water may be used for practices such as filter washing, service water, and chlorine injectors. Allowance is also needed for filter downtime during a wash cycle. The designer should be particularly careful in designing small treatment plants since in-plant water use can be a significant portion of total production.

The designer should consider the capacity of the plant to ensure that it is possible to produce sufficient water to satisfy the most onerous regularly occurring combination of water demand and raw water quality. This may occur in the spring when raw water quality from surface sources is often worse than average and raw water temperatures are low (reaction times are longer and the efficiency of sedimentation tanks and filters is reduced under peak solids loading). The design should be evaluated against the expected water demand at that time of the year. A most onerous condition also may occur at any time as a result of algal blooms. The designer should review the records for such challenging occurrences (*Section 3.6 – Plant Capacity Rating*).

3.4.2 Domestic Water Demands

Domestic water demands vary greatly from one water system to another. Depending upon such factors as the presence of service metering, lawn-watering practices, use of bleeders to prevent freezing, water quality, water conservation programs and leakage (*Section 3.5 – Water Conservation*), daily per capita consumption can vary from less than 180 L (48 USgal) to more than 1,500 L (396 USgal). For design purposes, existing reliable records should be used wherever possible. Domestic water demand used in design historically has ranged from 270 to 450 L/(cap·d) [70 to 120 USgal/(cap·d)]. With increased use of water metering and increased water conservation, the designer may find values at the low end of this range.

Minimum rate, maximum day and peak rate factors for the system should be based on existing flow data, where available. Table 3.1 provides peaking factors for use with average day demand when actual data are not available or are unreliable.

Table 3-1: Peaking Factors

POPULATION	MINIMUM RATE FACTOR (MINIMUM HOUR)	MAXIMUM DAY FACTOR	PEAK RATE FACTOR (PEAK HOUR)
500 - 1 000	0.40	2.75	4.13
1 001 - 2 000	0.45	2.50	3.75
2 001 - 3 000	0.45	2.25	3.38
3 001 - 10 000	0.50	2.00	3.00
10 001 - 25 000	0.60	1.90	2.85
25 001 - 50 000	0.65	1.80	2.70
50 001 - 75 000	0.65	1.75	2.62
75 001 -150 000	0.70	1.65	2.48
greater than 150 000	0.80	1.50	2.25

3.4.3 Commercial and Institutional Water Demands

Institutional and commercial flows should be determined by using historical records, where available. Where no records are available, the values in Table 3.2 should be used. For other commercial and tourist-commercial areas, an allowance of 28 m^3/(ha·d) [3000 USgal/(acre·d)] average flow should be used in the absence of reliable flow data.

When using the above unit demands, maximum day and peak rate factors should be developed. For establishments in operation for only a portion of the day such as schools and shopping plazas, the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water use rate would be at an average rate of 70 L/(student·day) [19 USgal/(student·day)] x 24/8 or 210 L/student (55 USgal/student) over the 8-hour period of operation. The water use will drop to a residual amount during the remainder of the day. Schools generally do not exhibit large maximum day to average day ratios and a factor of 1.5 will generally cover this variation. For estimation of ***peak demand*** rates, an assessment of the water-using fixtures is generally necessary and a fixture-unit approach should be used.

Table 3-2: Typical Water Demands for Selected Commercial and Institutional Users

COMMERCIAL AND INSTITUTIONAL USE	WATER USE (DAILY AVERAGE)
Shopping Centres (based on total floor area)	2500-5000 L/(m^2·day) [60-120 USgal/(ft^2·day)]
Hospitals	900-1800 L/(bed·day) [240-480 USgal/(bed·day)]
Schools	70-140 L/(student·day) [20-40 USgal/(student·day)]
Travel Trailer Parks (min.with separate hook-ups)	340 L/(space·day) [90 USgal/(space·day)] 800 L/(space·day) [210 USgal/(space·day)]
Campgrounds	225-570 L/(campsite·day) [60-150 USgal/(campsite·day)]
Mobile Home Parks	1000 L/(space·day) [260 USgal/(space·day)]
Motels	150-200 L/(bed-space·day) [40-50 USgal/(bed-space·day)]
Hotels	225 L/(bed-space·day) [60 USgal/(bed-space·day)]

3.4.4 Industrial Water Demands

Industrial water demands are often expressed in terms of water requirements per gross hectare of industrial development when the type of industry is unknown (e.g., new industrial parks). These demands will vary greatly with the type of industry, but common allowances for industrial areas range from 35 m^3/(ha·d) [3740 USgal/(acre·d)] for light industry to 55 m^3/(ha·d) [5880 USgal/(acre·d)] for heavy industry. These are average daily demands. Peak usage rates will generally be 2 to 4 times the average rate depending on factors such as the type of industry and production schedule.

When the type of industry is known, discussions should be held with representatives of the industry to determine water requirements.

3.4.5 Demand Considerations for Systems Serving Fewer than 500 People

3.4.5.1 Household (Interior) Water Demands & Peaking Factors

As a minimum, the water supply/treatment facility should be designed to meet the projected maximum daily flow requirement of the service area with peak hourly, outdoor use and fire demands met from storage. Where it is possible to develop the source of supply to meet more than the projected maximum daily flow, the storage volume can be reduced accordingly.

Average daily domestic consumption rates can vary from less than 180 L/(cap·d) [48 USgal/(cap·d)] to more than 1,500 L/(cap·d) [396 USgal/(cap·d)]. These values represent the average flow over a 24 hour period and do not reflect the fact that there are maximum day and peak hour/instantaneous demands in the system each day which will exceed the average value by a significant amount. It is essential that the source of supply and the distribution system be capable of meeting these maximum and peak demand rates without overtaxing the source or resulting in excessive pressure loss in the distribution system.

In general, small systems have higher peaking factors for maximum day and peak hour demand than large systems. The minimum rate, maximum day and peak rate factors for the system should be based on existing flow data or data from a similar nearby system where available. Table 3.3 provides peaking factors for use with average day demand when actual data are not available.

Table 3-3: Peaking Factors for Drinking-Water Systems Serving Fewer than 500 People

DWELLING UNITS SERVICED	EQUIVALENT POPULATION	NIGHT MINIMUM HOUR FACTOR	MAXIMUM DAY FACTOR	PEAK HOUR FACTOR
10	30	0.1	9.5	14.3
50	150	0.1	4.9	7.4
100	300	0.2	3.6	5.4
150	450	0.3	3.0	4.5
167	500	0.4	2.9	4.3

3.4.5.2 Outdoor Water Use

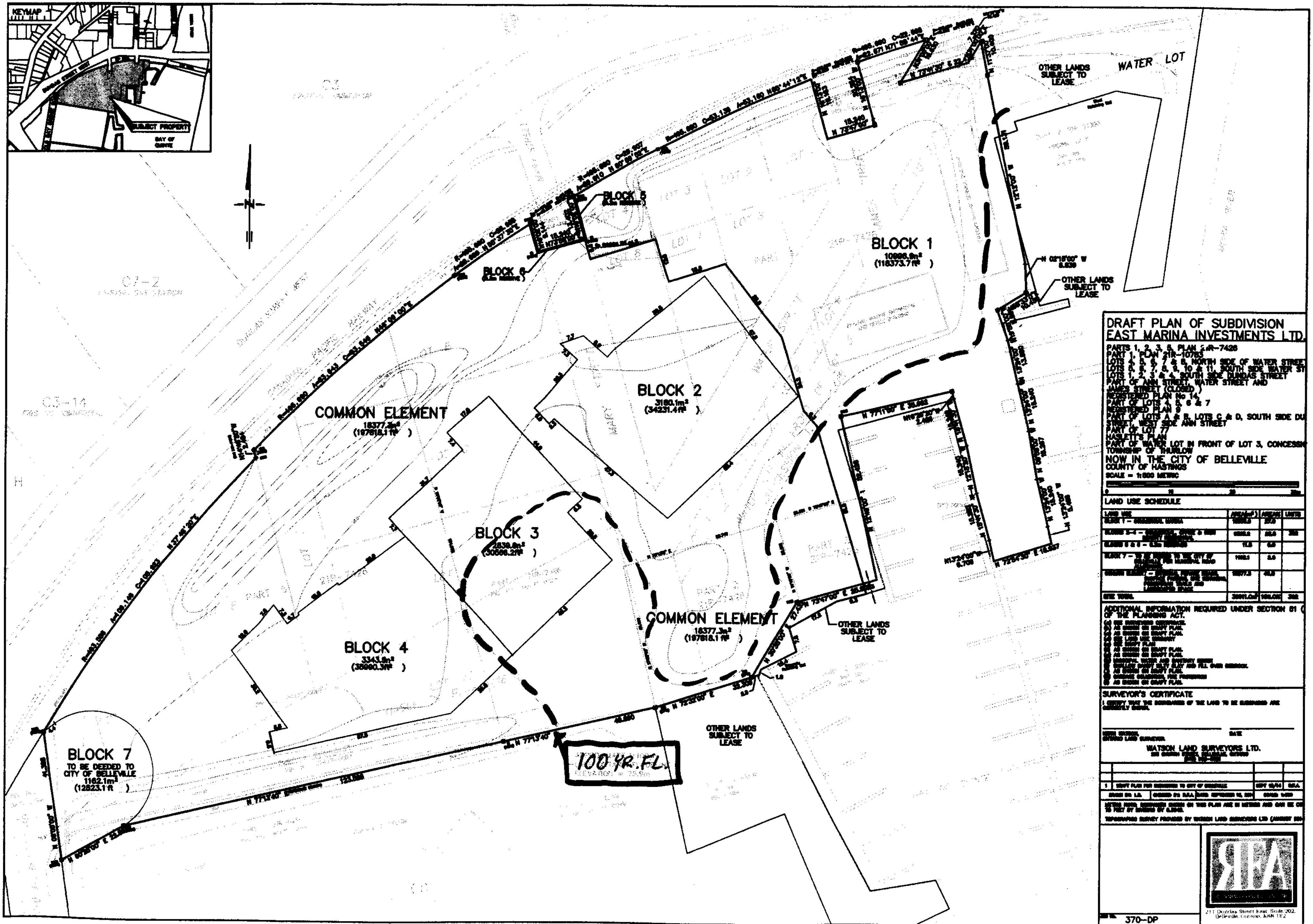
For outdoor water use, it should be assumed that a maximum of 25% of the homeowners could be using an outdoor tap at any one time at a rate of 20 L/min (5.3 USgpm) for one hour per day. Where fire protection is provided, then this outdoor use need not be considered.

3.4.5.3 Fire Protection

The decision as to whether or not fire protection will be provided via the communal water supply system is a municipal responsibility. In deciding upon

Stormwater Management


Legend
Floodline
Floodline
Assessment Parcels
Assessment Parcels
Lots
Lots
Township Boundary
Township Boundary
Main Creeks
Main Creeks
Watercourse
Watercourse
DUNDAS STREET WEST
MARY STREET
CON 1 THURLOW
LOT 2
LOT 3
1:1,000
Quinte CONSERVATION


KEYMAP
SUBJECT PROPERTY
BAY OF QUINTE
WATER LOT
OTHER LANDS SUBJECT TO LEASE
BLOCK 1
10996.6m²
(118373.7ft²)
BLOCK 2
3180.1m²
(34231.4ft²)
BLOCK 3
2839.6m²
(30566.2ft²)
BLOCK 4
3343.8m²
(35990.3ft²)
BLOCK 5
BLOCK 6
BLOCK 7
TO BE DEEDED TO CITY OF BELLEVILLE
1162.1m²
(12523.1ft²)
COMMON ELEMENT
18377.3m²
(197818.1ft²)
100 YR. FL.
DRAFT PLAN OF SUBDIVISION
EAST MARINA INVESTMENTS LTD.
NOW IN THE CITY OF BELLEVILLE
COUNTY OF HASTINGS
LAND USE SCHEDULE
SURVEYOR'S CERTIFICATE
WATSON LAND SURVEYORS LTD.
370-DP

APPENDIX D:
FIRE HYDRANT FLOW TEST RESULTS

Routing

White - 1. Op. Mgr. 2. Draft. 3. FF bk.
Pink - File 842
Canary - Originator

Belleville Utilities Commission
459 SIDNEY STREET
P.O. BOX 939
BELLEVILLE, ONT., K8N 5B6
(613) 966-3651

FIRE HYDRANT FLOW TEST

Date: Mar 26/07
Time: 0920
Performed by MAB/JC
File: 842

(Flow)

Pitot Hydrant No. 48 Dundas W 25 m 19

Adjacent Hydrant No. 68 Dundas W
(Residual & Static)

Adjacent Hydrant _____ Ft. Above or Below Pitot Hydrant

Street Name Dundas W
Location on St. or name of Bldg. __________

Provide Four Pressure Readings:
Select outlets to give 10 psi drop at adjacent hydrant if possible

		OUTLETS					
		one - 1"	one - 1 1/8"	one - 1½"	one - 2½"	two - 2½"	
Step One -	Adjacent Hydrant				84	84	psi (static)
Step Two -	Pitot Hydrant				70	46	psi (flow)
Step Three -	Adjacent Hydrant				81	78	psi (residual)
Step Four -	Adjacent Hydrant				84	84	psi (static check)

low with 20 psi residual at adjacent hydrant

$$= \text{measured flow} \left(\frac{\text{available drop}}{\text{test drop}} \right)^{.54}$$

Available drop is static less 20
Test drop is static less residual

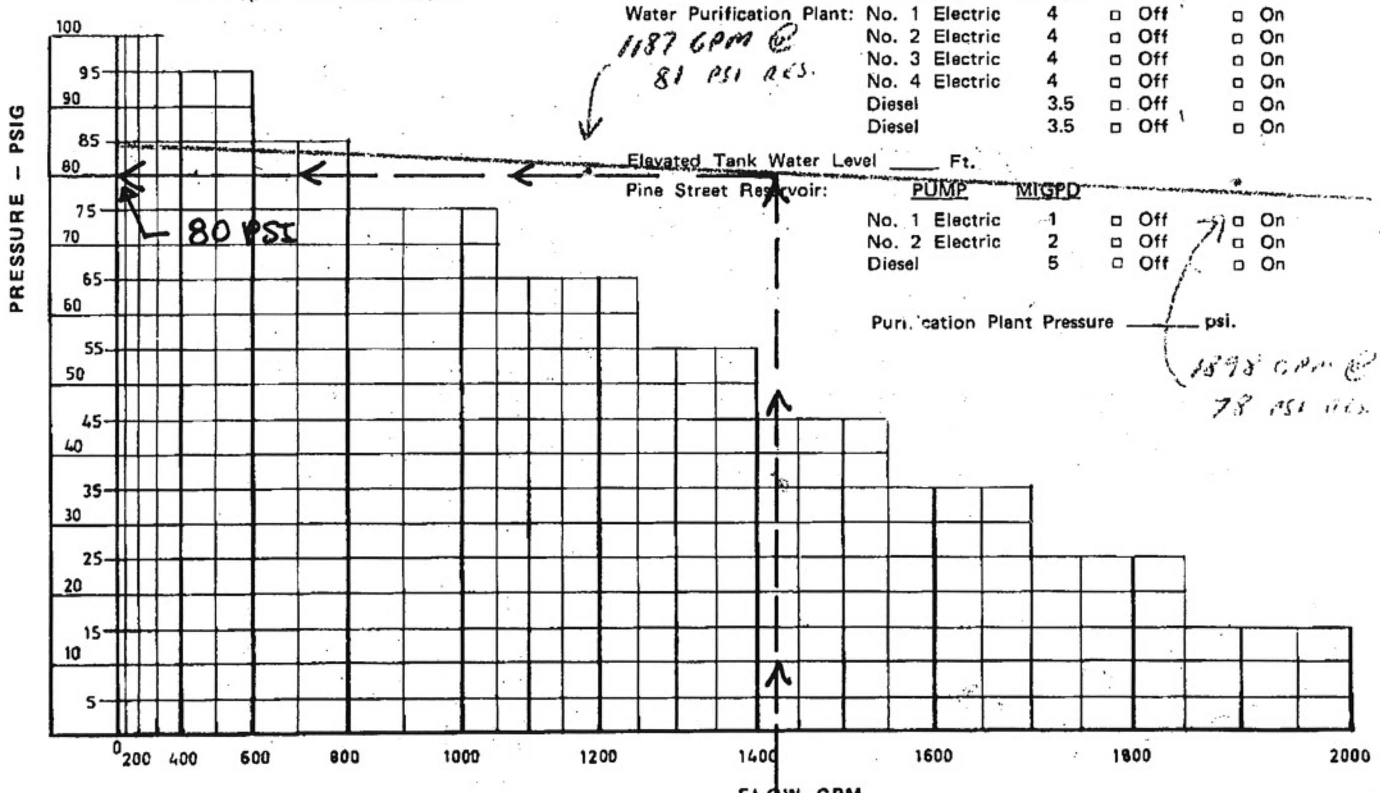
Information below can be obtained at a later date from records at Water Purification Plant.

	PUMP	MIGPD		
Water Purification Plant:	No. 1 Electric	4	☐ Off	☐ On
	No. 2 Electric	4	☐ Off	☐ On
	No. 3 Electric	4	☐ Off	☐ On
	No. 4 Electric	4	☐ Off	☐ On
	Diesel	3.5	☐ Off	☐ On
	Diesel	3.5	☐ Off	☐ On

Elevated Tank Water Level _____ Ft.

	PUMP	MIGPD		
Pine Street Reservoir:	No. 1 Electric	1	☐ Off	☐ On
	No. 2 Electric	2	☐ Off	☐ On
	Diesel	5	☐ Off	☐ On

Purification Plant Pressure _____ psi.



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APPENDIX E:
WATERCAD PUMP CURVE CALCULATION

Pitot Hydrant:	48 Dundas St W	Performed by:	
Adjacent Hydrant:	68 Dundas St W	City of Belleville	
Date:	March 26, 2007	Time:	09:20

Static Pressure (P_s):	84	psi
Residual Pressure (P_t):	78	psi
Residual Flow (Q_t):	1898	gpm
Design Pressure (P_0):	80	psi
Design Flow (Q_0):	1525	gpm

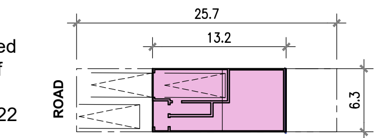
$$Q_0 = Q_t\left(\frac{P_s - P_0}{P_s - P_t}\right)^{0.54}$$

	Flow (gpm)	Pressure (ft)	Flow (L/s)	Pressure (m)
Shutoff Point	0	194.04	0.00	59.14
Design Point	1525	184.80	96.20	56.33
Max Operating Point	1898	180.18	119.74	54.92

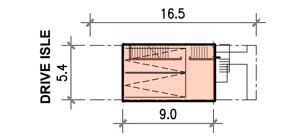
APPENDIX F:
ENLARGED SERVICING FIGURES AND TABLES

Total 213 Residential Units

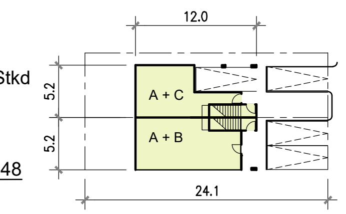
TYPE A
2 Units Stacked
Unit A: 2200sf
Unit B: 1430
Total Units = 22



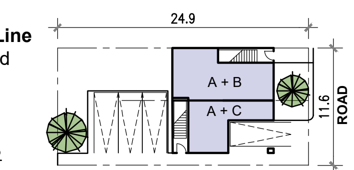
TYPE B
2 Units Stacked
Unit A: 1385sf
Unit B: 550sf
Total Units = 88



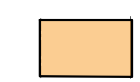
TYPE C
2 @ 2 Units Stkd
Unit A: 900sf
Unit B: 750sf
Unit C: 650sf
Total Units = 48



TYPE D - Rail Line
2 @ 2 Units Stkd
Unit A: 800sf
Unit B: 700sf
Unit C: 600sf
Total Units = 48



TYPE E
Condo
850sf
Total Units = 7



Statistics to come:

Per Unit type (A,B,C,D E):
1. Bedrooms / w/c's (estimate)
2. max height
3. lot area
4. frontage
5. lot coverage percentage
6. min. landscaped area
7. min front, side, rear yard setbacks
8. parking spaces per unit

General:
1. site dimensions
2. parking count visitor
3. commercial area
4. amenity area (building)
5. other...

ROSALIE DAWSON ARCHITECT INC.
416.209.4424
rosalie@RDArchitect.ca
RDArchitect.ca
169 Shoreline Drive,
Oro-Medonte, ON L0L 1T0

Porta - Townhome Study
25 Dundas Street West, Belleville Ontario

Client: MDM Developments

drawing: Low Rise Site Plan Sketch - V4, Legal Boundaries Updated - 21M318

issued for: Consultant Coordination

scale:	1:750
job number:	21009
drawn by:	RD
dwg. ref. no:	N/A

plot date: 2025.06.06

drawing no: 1

Total 213 Residential Units

TYPE A
2 Units Stacked
Unit A: 2200sf
Unit B: 1430
Total Units = 22

TYPE B
2 Units Stacked
Unit A: 1385sf
Unit B: 550sf
Total Units = 88

TYPE C
2 @ 2 Units Stkd
Unit A: 900sf
Unit B: 750sf
Unit C: 650sf
Total Units = 48

TYPE D - Rail Line
2 @ 2 Units Stkd
Unit A: 800sf
Unit B: 700sf
Unit C: 600sf
Total Units = 48

TYPE E
Condo
850sf
Total Units = 7

Statistics to come:

Per Unit type (A,B,C,D E):
1. Bedrooms / w/c's (estimate)
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8. parking spaces per unit

General:
1. site dimensions
2. parking count visitor
3. commercial area
4. amenity area (building)
5. other...

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issued for: Consultant Coordination

scale: 1:750
job number: 21009
drawn by: RD
dwg. ref. no: N/A

plot date: 2025.06.06

drawing no: 1

SANITARY SEWER DESIGN SHEET

Peak Design Flow Calculation

(Q_d) Peak Design Flow = (Q_p) Peak population flow + (Q_i) Peak extraneous flow + (Q_c) Commercial Flow

$$Q_d = Q_p + Q_i + Q_c$$

$$Q_p = \frac{PqM}{86.4}$$

$$Q_i = IA$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

Where:

- q = Average daily per capita flow — 350 L/d*cap
- I = Unit of peak extraneous flow — 0.28 L/s*ha
- M = Harmon peaking factor (min = 2)
- P = Population in 1000's
- A = Area in hectares

Population Flows (Persons/Unit)

Townhomes	2.5
Apartment	2.5

Commercial Flows

Average Commercial Flows	2.5 L/d*m^2
Peaking Factor	2.0

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

- A = area of pipe in m^2
- R = Hydraulic radius = A / P
- P = Wetted perimeter
- S = Slope (m/m)
- n = Manning's friction coef.

Check

$Q_d \leq 0.8 \cdot (Pipe\ Capacity)$

$0.6 \leq V \leq 3.0$

use Actual V if d:D < 0.3

LOCATION			PEAK FLOW CALCULATION													PROPOSED SEWER							
			Residential							Commercial													
			Individual				Cumulative			Individual	Cumulative												
			Units																				
Catchment/ Street	Downstream Manhole	Upstream Manhole	Townhome	Apartment	Pop.	Area (A) (ha)	Pop.	Area (A) (ha)	Peaking Factor M	Area (A) (ha)	Area (A) (ha)	Population Flow $Q_{(p)}$ (L/s)	Commercial Flow $Q_{(c)}$ (L/s)	Peak Ex. Flow $Q_{(i)}$ (L/s)	Design Flow $Q_{(d)}$ (L/s)	Pipe Size (mm)	Grade *(use m/m)* (%)	Capacity n = 0.013 (L/s)	Full Flow Velocity (m/s)	Ratio d:D	Actual Velocity at Q_d (m/s)	Check	q/Q
	SA12-S	SA7	32		80	0.27	80.0	0.27	4.27		0.00	1.38	0.00	0.08	1.46	200	0.70%	27.4	0.87	0.15	0.00	Velocity	0.05
	SA13	SA11	12		30	0.27	30.0	0.27	4.35		0.00	0.53	0.00	0.08	0.60	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA10-W	SA11	12		30	0.18	30.0	0.18	4.35		0.00	0.53	0.00	0.05	0.58	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA11	SA5	4		10	0.1	70.0	0.55	4.28		0.00	1.21	0.00	0.15	1.37	200	0.40%	20.7	0.66	0.17	0.00	Velocity	0.07
	SA10-S	SA4	20		50	0.16	50.0	0.16	4.31		0.00	0.87	0.00	0.04	0.92	200	0.70%	27.4	0.87	0.12	0.00	Velocity	0.03
	SA12-W	SA9	12		30	0.25	30.0	0.25	4.35		0.00	0.53	0.00	0.07	0.60	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA10-E	SA9	12		30	0.18	30.0	0.18	4.35		0.00	0.53	0.00	0.05	0.58	200	0.70%	27.4	0.87	0.10	0.00	Velocity	0.02
	SA9	SA3	32		80	0.23	140.0	0.66	4.20		0.00	2.38	0.00	0.18	2.57	200	0.40%	20.7	0.66	0.23	0.00	Velocity	0.12
Commercial & Crates	SA8	SA7	18	7	62.5	0.47	62.5	0.47	4.29	1.13	1.13	1.09	0.65	0.13	1.87	200	0.70%	27.4	0.87	0.17	0.00	Velocity	0.07
Amenity	SA7	SA2	4		10	0.06	152.5	0.80	4.19	0.16	1.29	2.59	0.75	0.22	3.56	200	0.40%	20.7	0.66	0.28	0.00	Velocity	0.17
	SA6	SA5	20		50	0.29	50.0	0.29	4.31		0.00	0.87	0.00	0.08	0.96	200	0.70%	27.4	0.87	0.13	0.00	Velocity	0.03
	SA5	SA4	12		30	0.11	150.0	0.95	4.19		0.00	2.55	0.00	0.27	2.81	200	0.40%	20.7	0.66	0.25	0.00	Velocity	0.14
	SA4	SA3	12		30	0.11	230.0	1.22	4.13		0.00	3.84	0.00	0.34	4.19	200	0.40%	20.7	0.66	0.31	0.66	OK	0.20
	SA3	SA2	4		10	0.07	380.0	1.95	4.03		0.00	6.21	0.00	0.55	6.75	200	0.40%	20.7	0.66	0.39	0.66	OK	0.33
	SA2	SA1			0		532.5	2.75	3.96		1.29	8.54	0.75	0.77	10.06	200	0.40%	20.7	0.66	0.49	0.66	OK	0.48
	SA1	PS			0		532.5	2.75	3.96		1.29	8.54	0.75	0.77	10.06	200	0.40%	20.7	0.66	0.49	0.66	OK	0.48

Jewell Engineering Inc
1-71 Millennium Parkway
Belleville, ON, K8N 4Z5

Ph. 613-969-1111
Fx. 613-989-8988
www.jewelleng.ca

Designed:	Julie Humphries, C.E.T.
Checked:	Bryon Keene, P.Eng.
Date:	June 18, 2025

Project: Porta