Functional Servicing & Stormwater Management Report

Residential Development

2225 Erin Mills Parkway

City of Mississauga, Region of Peel

08 May 2023 *(Revision 0)*

fabian papa & partners

A Division of FP&P HydraTek Inc

3901 Highway 7, Suite 500 | Vaughan, Ontario | L4L 8LS
Tel: 905-264-2420 | www.fabianpapa.com

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APPENDICES

1.0 INTRODUCTION

fabian papa & partners have been retained by Sheridan Retail Inc. to prepare this Functional Servicing & Stormwater Management Report in support of an Official Plan Amendment and Re-Zoning Application for the property municipally known as 2225 Erin Mills Parkway. This report discusses the provision of municipal services required for the developments as well as the stormwater management strategy.

The subject site is located at the south-east corner of Erin Mills Parkway and Lincoln Green Way in the City of Mississauga, Region of Peel. The site currently contains a large shopping centre that is encompassed by Fowler Drive to the east and south, Lincoln Green Way to the North and Erin Mills Parkway to the west. The site also contains 4 small commercial buildings within the parking lot of the mall fronting Erin Mills Parkway. The balance of the site consists of surface parking lots, pedestrian walkways, and parking/landscaped islands. A key plan and aerial photograph of the subject site can be found in Appendix A.

The development proposal for 2225 Erin Mills Parkway contemplates two 15-storey residential condominiums. The first condominium is proposed to be built on the northwest corner of the site with a site are of approximately 5,351 m^2 (0.5351 ha), and is hereinafter referred to as “Building A”. Building A will have a total of 249 residential units and two underground parking levels which will be accessed from a proposed drive isle entrance on the northeast side of the building which connects to an existing drive isle running along the south side of the building with access from Erin Mills Parkway.

The second condominium is proposed to be built on east side of the shopping centre within the existing parking area of the mall with a site are of approximately 4,704 m^2 (0.4704 ha), and is herein referred to as “Building G”. Building G will have a total of 371 residential units and four underground parking levels, which will be accessed on the south side of the site from a proposed drive isle running along the west side of the building that will connect to Fowler Drive on the northeast corner of the site. Architectural schematic floor and elevation plans for the two developments can be found in Appendix A as reference.

2.0 WATER SUPPLY

The existing municipal water infrastructure surrounding the subject site consists of a 400 mm diameter watermain along the east side of Erin Mills Parkway, a 400 mm diameter watermain on the north side of Lincoln Green Way, and a 750 mm diameter watermain on the south side of Lincoln Green Way. There is also a 300 mm diameter watermain along Fowler Drive that is connected to the same water distribution system on Erin Mills Parkway as mentioned above. The plan and profile drawings for these municipal roads were obtained from the City, with pertinent information included on the Site Servicing Schematics provided in Appendix F; excerpt copies of the plan and profile drawings are provided in Appendix B for reference.

At the time of writing, a hydrant flow test could not be completed, however, one has been commissioned and is scheduled to be completed in the summer of 2023. According to Region of Peel water system mapping, the site is serviced by Pressure Zone 2 (PD2). Based on HGL observations, it is assumed that the static pressure in the vicinity of the site ranges from 70 to 80 psi. For the purposes of this report, the static is pressure is assumed to be 65 psi, however it is always recommended that a hydrant flow test be conducted prior to detailed design in order to better understand the local hydraulic response in the vicinity of the subject site.

2.1. Supply Demands

The domestic water demand for Building A and Building G have been calculated based on the Region of Peel design criteria. The demands are summarized as follows (refer to Appendix C for the detailed calculations):

Domestic Water Supply Demand

	Ave. Domestic Demand, ADD (L/s)	Peak Hour Demand, PHD (L/s)	Max Day Demand, MDD (L/s)
Building A	2.2	6.5	4.4
Building G	3.3	9.7	6.5

The recommended fire demand for Building A is calculated using the criteria outlined in the Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey. Appropriate reductions and increases have been applied and are shown below:

Reductions & Increases for Fire Flow

	Co. for Fire Resistive Construction	Adjustment for Building Occupancy	Decrease for Fire Suppression System	Increase due to Proximity Exposure
Building A	0.6	-15%	-40%	+15%
Building G	0.6	-15%	-40%	+40%

The detailed fire demand for Building A is calculated as follows (refer to Appendix C for more details):

- Area (A) = Area of the largest floor + 25% of two adjoining floors
- A = 1,783 m^2 + 0.25 x (1,783 m^2 + 1,783 m^2) = 2,675 m^2
- F = 220 x C x $A^{0.5}$ = 220 x 0.6 x (2675 m^2)$^{0.5}$ = 6,826 L/min
- F = 7,000 L/min (rounded to nearest 1,000)
- Fire Flow = 7,000 x (1 - 0.15) – 5,950 x 0.40 + 5,950 x 0.15 = 4,463 L/min
- Fire Flow = 4,000 L/min (rounded to nearest 1,000) = 66.7 L/s

The design flows applied in the design of the service connections to Building A are as follows:

- Domestic Supply Line (PHD): 6.5 L/s
- Total Fire Flow (MDD + Fire): 4.4 L/s + 66.7 L/s = **71.0 L/s** (*Building A*)

The detailed fire demand for Building G is calculated as follows (refer to Appendix C for more details):

- Area (A) = Area of the largest floor + 25% of two adjoining floors
- A = 2,164 m^2 + 0.25 x (2,164 m^2 + 2,164 m^2) = 3,246 m^2
- F = 220 x C x $A^{0.5}$ = 220 x 0.6 x (3,246 m^2)$^{0.5}$ = 7,521 L/min
- F = 8,000 L/min (rounded to nearest 1,000)
- Fire Flow = 8,000 x (1 - 0.15) – 6,800 x 0.40 + 6,800 x 0.40 = 6,800 L/min

- Fire Flow = 7,000 L/min (rounded to nearest 1,000) = 116.7 L/s

The design flows applied in the design of the service connections to Building G are as follows:

- Domestic Supply Line (PHD): 9.7 L/s
- Total Fire Flow (MDD + Fire): 6.5 L/s + 116.7 L/s = **123.2 L/s** (*Building G*)

2.2. Proposed Connections and Layout

Per Ontario Building Code (OBC) Clause 3.2.9.7.4, any building above 84 m in height must be protected by two independent fire service connections separated by an isolation valve. Furthermore, Clause 7.12.2 of Standard 14 from the National Fire Protection Association (NFPA), classifies buildings greater than 23 m as high-rise and they require a second siamese connection.

2.2.1. Building A

Based on the demands for Building A calculated in Section 2.1, we propose a 150 mm diameter fire line to be connected to the existing 400 mm diameter watermain on Erin Mills Parkway. Also, there will be a 100 mm diameter domestic supply line that branches off of the 150 mm fire service. The water meter and back-flow preventers required by the Region will be installed within the mechanical room located in the parking level.

Building A is proposed to have a height of 52.1 m± and it will be sprinklered. To satisfy OBC and NFPA requirements, two siamese connections are proposed, however a second fire service is not required. The location of the proposed water infrastructure is shown on the Building A Site Servicing Plan (refer to drawing SSS-A).

2.2.2. Building G

Based on the demands for Building G calculated in Section 2.1, we propose a 150 mm diameter fire line to be connected to the existing 300 mm diameter watermain on Fowler Drive, and there will be a 100 mm diameter domestic supply line that branches off of the 150 mm fire service. The water meter and back-flow preventers required by the Region will be installed within the mechanical room located in the parking level.

Building G is proposed to have a height of 52.1 m± and it will be sprinklered. To satisfy OBC and NFPA requirements, two siamese connections are proposed, however a second fire service is not required. The location of the proposed water infrastructure is shown on the Building G Site Servicing Plan (refer to drawing SSS-G).

2.3. Domestic and Fire Flow Analysis

2.3.1 Building A

The pressure at the building face of building A is calculated as the residual pressure at the main less the head loss in the supply line. Based on the estimated static pressure at the existing main and using the Hazen-Williams formula to determine the head losses in the lines, the resulting residual pressure at the building face for each connection is as follows (refer to Appendix C for the detailed calculations):

Hazen Williams Formula: $Q = 0.278 \times C \times D^{2.63} \times (H_f / L)^{0.54}$

Service Connections	Flow (L/s)	Head Loss (psi)	Head Loss (kPa)	Residual pressure at main (psi/kPa)	Residual pressure at Building (psi/kPa)
100 mm Domestic (PHD)	6.5	0.1	0.8	64.9/447	**64.8**/447
150 mm Fire (MDD + Fire)	71.0	2.2	14.9	56.4/389	**54.2**/374

The calculations above show that the residual pressures at the building face are above the Region's minimum acceptable pressures of 40 psi (275 kPa) for PHD and 20 psi (140 kPa) for MDD+Fire demand conditions, therefore, the existing municipal water infrastructure and the proposed internal water network can support the proposed Building A.

2.3.2 Building G

The pressure at the building face of building G is calculated as the residual pressure at the main less the head loss in the supply line. Based on the estimated static pressure at the existing main and using the Hazen-Williams formula to determine the head losses in the lines, the resulting residual pressure at the building face for each connection is as follows (refer to Appendix C for the detailed calculations):

Hazen Williams Formula: $Q = 0.278 \times C \times D^{2.63} \times (H_f / L)^{0.54}$

Service Connections	Flow (L/s)	Head Loss (psi)	Head Loss (kPa)	Residual pressure at main (psi/kPa)	Residual pressure at Building (psi/kPa)
100 mm Domestic (PHD)	9.7	0.3	1.8	64.8/447	**64.5**/445
150 mm Fire (MDD + Fire)	123.2	8.0	55.3	41.1/283	**33.1**/228

The calculations above show that the residual pressures at the building face are above the Region's minimum acceptable pressures of 40 psi (275 kPa) for PHD and 20 psi (140 kPa) for MDD+Fire demand conditions, therefore, the existing municipal water infrastructure and the proposed internal water network can support the proposed Building G.

3.0 SANITARY DRAINAGE

Local sanitary infrastructure consists of a 375 mm diameter sanitary sewer which travels south on Erin Mills Parkway, then turns east and enters an easement within the northwest limit of the property. This sewer flows in a northeasterly direction towards Fowler Drive and eventually drains through a residential development to the northeast of the site. Local sanitary infrastructure along Fowler Drive consists of a 250 mm diameter sanitary sewer draining north, which then connects to a 300 mm sanitary sewer which drains southwest on North Sheridan Way.

3.1. Sanitary Design Flow

The sanitary design flow for the both subject sites is calculated using the Region's design criteria for sanitary sewers. The relevant criteria are summarized below.

- Design Flow: 302.8 Lpcd used for pre- & post-development flows
- Peaking Factor: Calculated using the Harmon Formula
- Infiltration Flow: 0.2 L/s/ha
- Population Density: 2.7 people/unit for apartments
  70 people/ha for single family dwellings

3.1.1. Building A Design Flows

The pre-development flow for Building A is calculated as follows:

$$Q_{PRE-A} = \left(\frac{302.8\ \text{Lpcd} * 27\ \text{pers} * 4.36_{\text{Peaking}}}{86400\ \text{s / day}}\right) + 0.2\ \ \text{L/s/ha} * 0.5351\ \text{ha} = 0.52\ \text{L/s}$$

The post-development flow for Building A is calculated as follows:

$$Q_{POST\text{-}A} = \left(\frac{302.8\ \text{Lpcd} * 672\ \text{pers} * 3.90_{\text{Peaking}}}{86400\ \text{s / day}}\right) + 0.2\ \ \text{L/s/ha} * 0.5351\ \text{ha} = 9.3\ \text{L/s}$$

Based on the above, the increase in flow is calculated to be **8.8 L/s** (9.3 L/s – 0.5 L/s) for Building A. Please refer to Appendix D for the detailed sanitary design sheet.

3.1.2. Building G Design Flows

The pre-development flow for Building G is calculated as follows:

$$Q_{PRE} = \left(\frac{302.8\ \text{Lpcd} * 0\ \text{pers} * 4.50_{\text{Peaking}}}{86400\ \text{s / day}}\right) + 0.2\ \ \text{L/s/ha} * 0.4701\ \text{ha} = 0.1\ \text{L/s}$$

The post-development flow for Building G is calculated as follows:

$$Q_{POST} = \left(\frac{302.8\ \text{Lpcd} * 1002\ \text{pers} * 3.80_{\text{Peaking}}}{86400\ \text{s / day}}\right) + 0.2\ \ \text{L/s/ha} * 0.4701\ \text{ha} = 13.4\ \text{L/s}$$

Based on the above, the increase in flow is calculated to be **13.3 L/s** (13.4 L/s – 0.1 L/s) for Building G. Please refer to Appendix D for the detailed sanitary design sheet.

3.2. Municipal Service Connections

3.2.1. Building A

Building A will be serviced by a 150 mm diameter sanitary sewer which will be connected to the existing 375 mm sanitary sewer within the easement that drains east. As derived from the Region's plan and profile drawings, the invert of the existing sewer at the location of the proposed connection is approximately 126.44 m± (3.2 m± below the surface elevation). It is proposed that a manhole (MH.2A) be installed at the connection location outside the property limit. A 4.0 meter 150 mm diameter sewer will be installed from MH.2A to a precast sanitary control manhole (MH.1A) placed inside the property boundary just outside the parking level. This sewer will be connected at an invert elevation of 126.49 m, and placed at a gradient of 2.0% resulting in an invert of 126.57 m at MH.1A, and will operate at 41.4% of its flow capacity (22.5 L/s). The building connection will be a 2.3 m 150 mm diameter sewer installed from MH.1A to the building. The sewer will be connected at MH.1A at an invert of 126.67 m, and placed at a gradient of 2.0% resulting in an invert of 126.72 at the building, and will operate at 41.4% of its flow capacity (22.5 L/s).

It should be noted that the Owner wishes to relocate the existing municipal sanitary sewer along the existing easement north of Building A in order to support the development. As a result, the Owner wishes to dissolve the sewer easement (subject to further discussions with the Region of Peel to determine its possibility/feasibility).

3.2.2. Building G

Building G will be serviced by a 150 mm diameter sanitary sewer which will be connected to the existing 250 mm sanitary sewer on Fowler Drive. As derived from the Region's plan and profile drawings, the invert of the existing sewer at the location of the proposed connection is approximately 120.92 m± (3.0 m± below the surface elevation). The connection will require a manhole to be installed on Fowler Drive since the service connection is greater than 50% of the municipal sanitary sewer size. A cast-in-place control manhole will be located at the property line inside the P1 level of the building, outside of the municipal right-of-way. The connection is proposed to be placed at an invert of approximately 122.10 m± at the proposed manhole on the road, and constructed at a 2.2% gradient resulting in an invert of 122.50 m± at the building. The proposed service is expected to convey the sanitary flow from the site operating at 59.8% of full flow capacity (22.5 L/s).

4.0 STORM DRAINAGE

4.1. Overview

Local storm infrastructure consists of a 1200 mm diameter storm sewer which drains north along Erin Mills Parkway. The sewer then turns east continuing along an easement which runs along the south side of proposed Building A (within the existing Sheridan Centre parking lot). The sewer then turns north again passing along the east side of the site, then turns east once more before discharging into the Loyalist Creek stormwater conveyance system. In addition, there is a 1200 mm diameter storm sewer which drains north on Fowler Drive and passes by the Sheridan Centre near the proposed Building G. This sewer then continues to drain into a 1050 mm diameter storm sewer which continues north and eventually discharges into the Loyalist Creek. Refer to the Site Servicing Schematic for Building A (SSS-A), and Building G (SSS-G) in Appendix E for further infrastructure detail.

4.2. Design Criteria

The stormwater management servicing strategy for the subject development has been prepared in accordance with the City and CVC's design standards and criteria for the Loyalist Creek Sub-watershed. The relevant criteria are summarized below:

<u>Water Quantity Management</u>

- Per the Loyalist Creek quantity control criteria (East of Winston Churchill Boulevard), the 10-year post-development release rate must not exceed the 10-year pre-development, however, per the request from the City of Mississauga, the 10-year post-development release rate has been limited to the 2-year pre-development release rate.
- The pre-development runoff coefficient shall not exceed 0.50.
- An overland flow route shall be provided within the developed site to direct runoff in excess of the 10-year storm runoff to an approved overland flow outlet.

<u>Water Quality Management</u>

- All runoff from the site shall achieve a long-term average removal of 80% of Total Suspended Solids (TSS) on an annual loading basis.

<u>Water Balance Management</u>

- To achieve the water balance targets, a minimum of the first 5 mm from each rainfall event must be retained on-site for rainwater reuse or infiltration.

4.3. Pre-Development Conditions

Building A is currently serviced by parking lot catchbasins which collect stormwater flows from the site and convey these flows into the existing 1200 mm storm sewer located within the easement within the Sheridan Centre parking area. For Building G, the existing drainage is collected by parking lot catchbasins and directed to the existing 1050 mm diameter storm sewer located on Fowler Drive. Refer to Appendix E for Pre- & Post-Development Drainage Plans for both Building A and Building G (SWM-1A, SWM-2A, SWM-1G, SWM-2G).

The pre-development weighted runoff coefficient for Building A is estimated to be 0.88, and for Building G it is estimated to be 0.82, however, since the City of Mississauga's stormwater criteria limits the pre-development coefficient to a maximum of 0.50, then this value is what governs for estimating target post-development release rates to the storm sewer system.

The 2-year return period design rainfall intensity is calculated per City standards as follows:

$$I_2 = \frac{610}{(T+4.6)^{0.78}} = \frac{610}{(15+4.6)^{0.78}} = 59.9 \text{ mm / hr}$$

The corresponding 2-year pre-development flow for Building A is calculated as follows:

$$Q_{\text{2-Year Pre Building A}} = \frac{(A * R) * I_2}{360} = \frac{(0.5351 \text{ ha} * 0.50) * 59.9 \text{ mm / hr}}{360} = 0.0445 \text{ m}^3 \text{ / s} = 44.5 \text{ L / s}$$

The total allowable stormwater sewer discharge from the Building A shall be limited to **44.5 L/s**.

The corresponding 2-year pre-development flow for Building G is calculated as follows:

$$Q_{\text{2-Year Pre Building G}} = \frac{(A * R) * I_2}{360} = \frac{(0.4704 \text{ ha} * 0.50) * 59.9 \text{ mm / hr}}{360} = 0.0391 \text{ m}^3 \text{ / s} = 39.1 \text{ L / s}$$

The total allowable stormwater sewer discharge from the Building G shall be limited to **39.1 L/s**. Please refer to the detailed storm sewer design sheet provided in Appendix E for the calculations shown above.

4.4. Water Quantity Management

The post-development hydrologic conditions for the site have been established using the City's criteria which include the City's IDF data, a recommended time of concentration of 15 minutes, and the following run-off coefficient adjustments:

Runoff Coefficients

Bare Roof	0.90
Hardscape	0.90
Permeable Pavers	0.50
Green Roof	0.50
Landscaped Areas	0.25

These design parameters are used in the subsequent sections to determine the on-site storage requirements to meet the target release rates for both Building A and Building G.

The 10-year return period design rainfall intensity is calculated per City standards as follows:

$$I_{10} = \frac{1010}{(T+4.6)^{0.78}} = \frac{1010}{(15+4.6)^{0.78}} = 99.2 \text{ mm / hr}$$

To ensure the design criteria in Section 4.2 is met, it is proposed that drainage from proposed buildings, drive isles, surrounding landscaped and walking areas be collected and controlled to the pre-development release rate, and released via the proposed storm service connection to the existing municipal storm sewer.

To attenuate flows to the target rate, on-site storage will be required. Typically, a combination of roof top, surface and/or underground storage is used to achieve the required volumes. In both cases (for Building A and Building G), an underground storage tank is proposed to contain and control flows from each site for all storm events up to the 10-year level.

4.4.1. Building A Storage Tank

The 10-year post-development release rate is a combination of flow rates from various areas which include, rooftop areas, permeable paver areas, ground floor outdoor amenity space, and landscape areas. Adding these flow rates will yield the total 10-year flow rate from the site in which the underground storage structure must control before discharging to the storm sewer. Below are the necessary calculations:

$$Q_{\text{10-Yr, Unattenuated}} = \frac{(A \times R) \times I_{10}}{360} = \frac{(0.5351 \text{ ha} \times 0.58) \times 99.2 \text{ mm/hr}}{360} = 0.0851 \text{ m}^3\text{/s} = 85.1 \text{ L/s}$$

The total flow of 85.1 L/s is greater than the allowable release rate of 44.5 L/s. To attenuate flows from the site, an underground stormwater tank (2.55 m high above the orifice invert with a minimum footprint area of 43.3m²), complete with an orifice plate (upstream of the control manhole), is proposed along the south frontage of the property (within the P1 level of underground parking). Storm runoff generated on site will be collected and directed to this tank. It is important to stress that regular maintenance inspections of the storage tank, the orifice, and the control manhole should be conducted to ensure that there are no blockages or other conditions which would prevent the proper functioning of this design element. The recommended minimum frequency of such inspections is annually.

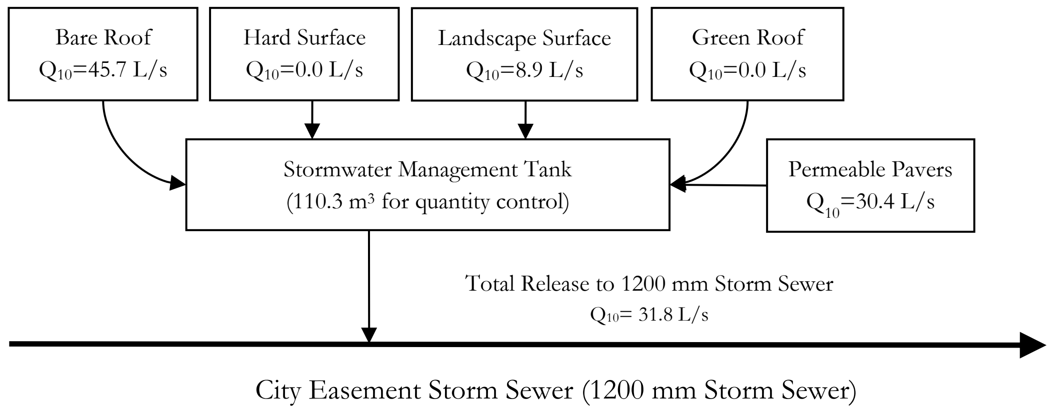
Utilizing a 120 mm diameter orifice plate and with the 10-Year storage depth in the system set at 1.11 m, the orifice discharge is calculated as follows:

$$Q_{\text{Orifice (SWM Tank)}} = (0.62) \times \frac{\pi \times (0.120)^2}{4} \times \sqrt{2 \times 9.81 \times (1.11\text{-}0.120/2)} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 31.8 \text{ L/s}$$

Using the City's IDF curve parameters for the 10-year storm event, and the orifice flow equation, the storage requirements for the tank are summarized as follows:

Total Allowable Site Release Rate (see above):	44.5 L/s
Proposed Release Rates:	
From SWM Tank:	31.8 L/s
Storage required (quantity control only):	48.0 m^3
Storage provided (to top of Tank):	110.3 m^3

A schematic representation of the proposed stormwater management system is provided as follows:



$$Q_{\text{SITE RELEASE}} \leq Q_{\text{ALLOWABLE DISCHARGE}}$$

$$31.8 \text{ L/s} \leq 44.5 \text{ L/s}$$

Based on the above, the total release rate from Building A is less than the target post-development 2-year release rate and thus is deemed acceptable.

4.4.1. Building G Storage Tank

The 10-year post-development release rate is a combination of flow rates from various areas which include, rooftop areas, permeable paver areas, ground floor outdoor amenity space, and landscape areas. Adding these flow rates will yield the total 10-year flow rate from the site in which the underground storage structure must control before discharging to the storm sewer. Below are the necessary calculations:

$$Q_{\text{10-Yr, Unattenuated}} = \frac{(A \times R) \times I_{10}}{360} = \frac{(0.4704 \text{ ha} \times 0.66) \times 99.2 \text{ mm/hr}}{360} = 0.0852 \text{ m}^3/\text{s} = 85.2 \text{ L/s}$$

The total flow of 85.2 L/s is greater than the allowable release rate of 39.1 L/s. To attenuate flows from the site, an underground stormwater tank (1.70 m high above the orifice invert with a minimum footprint area of 54.6m²), complete with an orifice plate (upstream of the control manhole), is proposed along the east frontage of the property (within the P1 level of underground parking). Storm runoff generated on site will be collected and directed to this tank. It is important to stress that regular maintenance inspections of the storage tank, the orifice, and the control manhole should be conducted to ensure that there are no blockages or other conditions which would prevent the proper functioning of this design element. The recommended minimum frequency of such inspections is annually.

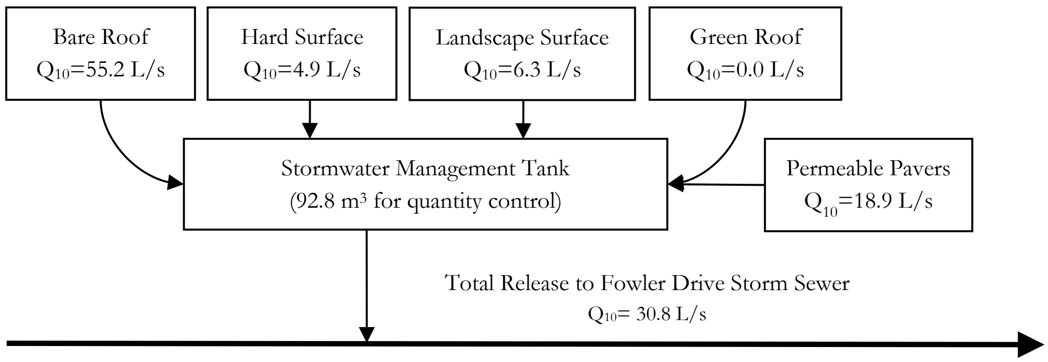
Utilizing a 125 mm diameter orifice plate and with the 10-Year storage depth in the system set at 0.90 m, the orifice discharge is calculated as follows:

$$Q_{\text{Orifice (SWM Tank)}} = (0.62) \times \frac{\pi \times (0.125)^2}{4} \times \sqrt{2 \times 9.81 \times (0.90\text{-}0.125/2)} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 30.8 \text{ L/s}$$

Using the City's IDF curve parameters for the 10-year storm event, and the orifice flow equation, the storage requirements for the tank are summarized as follows:

Total Allowable Site Release Rate (see above):	39.1 L/s
Proposed Release Rates:	
From SWM Tank:	30.8 L/s
Storage required (quantity control only):	49.0 m^3
Storage provided (to top of Tank):	92.8 m^3

A schematic representation of the proposed stormwater management system is provided as follows:



Fowler Drive (1200 mm Storm Sewer)

$$Q_{\text{SITE RELEASE}} \leq Q_{\text{ALLOWABLE DISCHARGE}}$$

$$30.8 \text{ L/s} \leq 39.1 \text{ L/s}$$

Based on the above, the total release rate from Building G is less than the target post-development 2-year release rate and thus is deemed acceptable.

4.5. Water Quality Management

Pursuant to the City's Design Criteria, stormwater quality controls are required to be implemented on-site to achieve a minimum of 80% long-term total suspended solid (TSS) removal. With respect to both sites, Building A and Building G, the drive isle and walkways are represented as permeable pavers in order to improve TSS removal rates. Refer to Appendix E for pre- and post-development storm drainage plans. For the purposes of determining the quality control achieved on the site, the following TSS removal rates will be applied for the various site areas (Building A and Building G):

TSS Removal Rates

Site Area	TSS Removal Rate
Roof	95%
Green Roof	100%
Landscaped Areas	100%
Permeable Pavers	80%
Hard Surface	0%

4.5.1. Water Quality - Building A

Based on the above, the following chart summarizes the inferred TSS removal rates for Building A:

TSS Removal Rates (Building A)

Site Area	Area (m^2)	(% of Total)	TSS Removal Rate	Overall
Bare Roof	1,844	34.5%	95%	32.7%
Green Roof	0	0.0%	100%	0.0%
Landscape	1,296	24.2%	100%	24.2%
Permeable Pavers	2,211	41.3%	80%	33.1%
Hard Surface	0	0.0%	0%	0.0%
Total	5,351	100 %		**90.0%**

As can be seen above, Building A meets the City's 80% TSS removal target. Therefore, Building A will not require additional treatment, and no further remediation will need to be done for stormwater quality.

4.5.2. Water Quality - Building G

Based on the above, the following chart summarizes the inferred TSS removal rates for Building G:

TSS Removal Rates (Building G)

Site Area	Area (m^2)	(% of Total)	TSS Removal Rate	Overall
Bare Roof	2,228	47.4%	95%	45.0%
Green Roof	0	0.0%	100%	0.0%
Landscape	911	19.4%	100%	19.4%
Permeable Pavers	1,369	29.1%	80%	23.3%
Hard Surface	196	4.2%	0%	0.0%
Total	4,704	100 %		**87.6%**

As can be seen above, Building G meets the City's 80% TSS removal target. Therefore, Building G will not require additional treatment, and no further remediation will need to be done for stormwater quality.

4.6. Water Balance Management

In order to promote preservation of the site's natural hydrological water balance, the City recommends that a minimum volume of 5 mm over the total site area must be retained, and re-used on-site.

4.6.1. Water Balance - Building A

Based on the inferred initial abstraction rates for the various site surfaces for Building A, the total abstraction is calculated as follows:

Initial Abstraction Table (Building A)

Site Area	Area (m²)	% of Total	Initial Abstraction for Site Area	Total Initial Abstraction for Site Area
Bare Roof	1,844	34.5%	1 mm	0.3 mm
Green Roof	0	0.0%	5 mm	0.0 mm
Landscape	1,296	24.2%	5 mm	1.2 mm
Permeable Pavers	2,211	41.3%	5 mm	2.1 mm
Hard Surfaces	0	0.0%	1 mm	0.0 mm
Total	5,351	100 %		**3.6 mm**

The total water balance required to be retained on-site is 5.0 mm, thus an additional 1.4 mm of rainfall (i.e., 5.0 mm – 3.6 mm) needs to be collected and retained on site. The total volume to be retained is calculated below:

$$V_{\text{required Building A}} = \text{Depth} \times A_{\text{Building A}} = 1.4 \text{ mm} \times 5{,}351 \text{ m}^2 / 1000 \text{ mm} = 7.4 \text{ m}^3$$

Per standard current industry practices, acceptable methods for water balance reuse include:

- Irrigation of landscaped areas (including evapo-transpiration), and/or
- Groundwater infiltration, and/or
- Greywater Toilet systems.

Given that the underground portion of the building extends to the property limits, infiltration is not possible. As such, we recommend that rain harvesting storage be provided within the sump of the Building A's stormwater tank (0.3 m depth) to retain the required volume for the irrigation system. The rainwater harvesting chamber can retain the following volume of water:

$$V_{\text{Sump-Building A}} = \text{Depth} \times \text{Area} = 0.3 \text{ m} \times (43.3 \text{ m}^2) = 13.0 \text{ m}^3$$

The specific details relating to the irrigation system will be provided by the Mechanical Consultant and Landscape Architect during the detailed design for building permits.

4.6.2. Water Balance - Building G

Based on the inferred initial abstraction rates for the various site surfaces for Building G, the total abstraction is calculated as follows:

Initial Abstraction Table (Building G)

Site Area	Area (m²)	% of Total	Initial Abstraction for Site Area	Total Initial Abstraction for Site Area
Bare Roof	2,228	47.4%	1 mm	0.5 mm
Green Roof	0	0.0%	5 mm	0.0 mm
Landscape	911	19.4%	5 mm	1.0 mm
Permeable Pavers	1,369	29.1%	5 mm	1.5 mm
Hard Surfaces	196	4.2%	1 mm	0.0 mm
Total	4,704	100 %		**2.9 mm**

The total water balance required to be retained on-site is 5.0 mm, thus an additional 2.1 mm of rainfall (i.e., 5.0 mm – 2.9 mm) needs to be collected and retained on site. The total volume to be retained is calculated below:

$$V_{\text{required Building G}} = \text{Depth} \times A_{\text{Building G}} = 2.1\text{ mm} \times 4{,}704\text{ m}^2 / 1000\text{ mm} = 9.7\text{ m}^3$$

Per standard current industry practices, acceptable methods for water balance reuse include:

- Irrigation of landscaped areas (including evapo-transpiration), and/or
- Groundwater infiltration, and/or
- Greywater Toilet systems.

Given that the underground portion of Building G extends close to the property limits, infiltration is not possible. As such, we recommend that rain harvesting storage be provided within the sump of Building G's stormwater tank (0.3 m depth) to retain the required volume for the irrigation system. The rainwater harvesting chamber can retain the following volume of water:

$$V_{\text{Sump}} = \text{Depth} \times \text{Area} = 0.3\text{ m} \times (54.6\text{ m}^2) = 16.4\text{ m}^3$$

The specific details relating to the irrigation system will be provided by the Mechanical Consultant and Landscape Architect during the detailed design for building permits.

Based on the above calculations, the total volume of storm water retained and available for re-use on Building A is 13.0 m^3 which is greater than the required volume of 7.4 m^3. The total volume of storm water retained and available for re-use on Building G is 16.4 m^3 which is greater than the required volume of 9.7 m^3. Therefore the water balance objectives for the City of Mississauga stormwater criteria have been met.

4.7. Municipal Service Connections

4.7.1. Building A

The storm runoff from the site will be captured and directed to a new 200 mm diameter storm service. Based on information gathered from the City's plan and profile drawings, the invert of the existing sewer at the proposed storm service connection is approximately 126.02 m±. The proposed storm service will be connected to the new manhole with an invert of 127.55 m and be constructed at a 2.0% slope. The resultant invert of the storm service at the control manhole will be 127.80 m. A cast-in-place (1.2 m × 1.2 m) control manhole will be located adjacent to the property line on the south side of the site and outside of the municipal right-of-way. The service has adequate capacity to convey the post-development storm flow from the site and will operate at 65.7% of full flow capacity (48.4 L/s) under 10-year controlled flow conditions.

4.7.2. Building G

The storm runoff from Building G will be captured and directed to a new 200 mm diameter storm service. Based on information gathered from the City's plan and profile drawings, the invert of the existing sewer at the proposed storm service connection is approximately 118.80 m±. The proposed storm service will be connected to the new manhole with an invert of 120.55 m and be constructed at a 2.0% slope. The resultant invert of the storm service at the control manhole will be 120.80 m. A cast-in-place (1.2 m × 1.2 m) control manhole will be located adjacent to the property line on the south side of the site and outside of the municipal right-of-way. The service has adequate capacity to convey the post-development storm flow from the site and will operate at 63.6% of full flow capacity (48.4 L/s) under 10-year controlled flow conditions.

4.8. Emergency Overflow

4.8.1. Building A

In the event that a storm greater than the 10-year level occurs, or if there is a blockage in the drainage system, the underground stormwater storage tank will be provided with an access frame and cover with an 'open grate' type (OPSD 400.100) which will act as an emergency overflow, located at the northwest corner of the property. The lid will be at an elevation of approximately 131.98 m±, and if an emergency overflow is experienced, the storage manhole will surcharge and spill stormwater toward Erin Mills Parkway in a southwest direction. We recommend backwater valves be installed between the stormwater storage structure and the incoming connections to prevent water from entering the individual leads should a blockage occur within the stormwater storage structure. We also recommend this facility to be inspected and maintained on an annual basis to ensure it is operating as designed.

4.8.2. Building G

In the event that a storm greater than the 10-year level occurs, or if there is a blockage in the drainage system, the underground stormwater storage tank will be provided with an access frame and cover with an 'open grate' type (OPSD 400.100) which will act as an emergency overflow, located at the northwest corner of the property. The lid will be at an elevation of approximately 123.30 m±, and if an emergency overflow is experienced, the storage manhole will surcharge and spill stormwater toward Fowler Drive in an easterly direction. We recommend backwater valves be installed between the stormwater storage structure and the incoming connections to prevent water from entering the individual leads should a blockage occur within the stormwater storage structure. We also recommend this facility to be inspected and maintained on an annual basis to ensure it is operating as designed.

4.9. Sediment and Erosion Control

In accordance with the Erosion and Sediment Control Guidelines for Urban Construction, temporary erosion and sediment control measures are required for any development application. It is proposed that a sediment control fence be installed along the entire perimeter of both sites per the City of Mississauga standard drawing 2940.010. Any existing / adjacent catch basins shall be protected with a Terrafix 360R geotextile fabric (or approved equal). In addition, a mud mat shall be installed along the frontage of the site to prevent any mud tracking onto the municipal roads.

5.0 CONCLUSIONS

This report illustrates that the proposed development is feasible from municipal servicing and stormwater management perspectives.

Proposed domestic water and fire demands can be accommodated by the existing municipal water supply infrastructure within the Region of Peel, although it is to be confirmed with hydrant flow test data.

It is assumed the receiving municipal sanitary drainage network can accommodate the increase in sanitary wastewater flow as calculated herein, subject to the Region of Peel's confirmation through their typical internal modelling analysis.

The receiving storm drainage network can accommodate the proposed development without improvements and the proposed internal storm sewer network, on-site underground storage tank, and the controlled discharge release rate to the receiving sewer satisfy the City's stormwater management objectives.

We trust that this satisfies your current needs. Should you have any questions, or require additional information, please do not hesitate to contact the undersigned.

Respectfully Submitted,

fabian papa & partners
A Division of FP&P HydraTek Inc.

LICENSED PROFESSIONAL ENGINEER
P. F. ALBANESE
2023 05 08
PROVINCE OF ONTARIO

Alessandro Stefenatti, B.Eng
Engineering Assistant

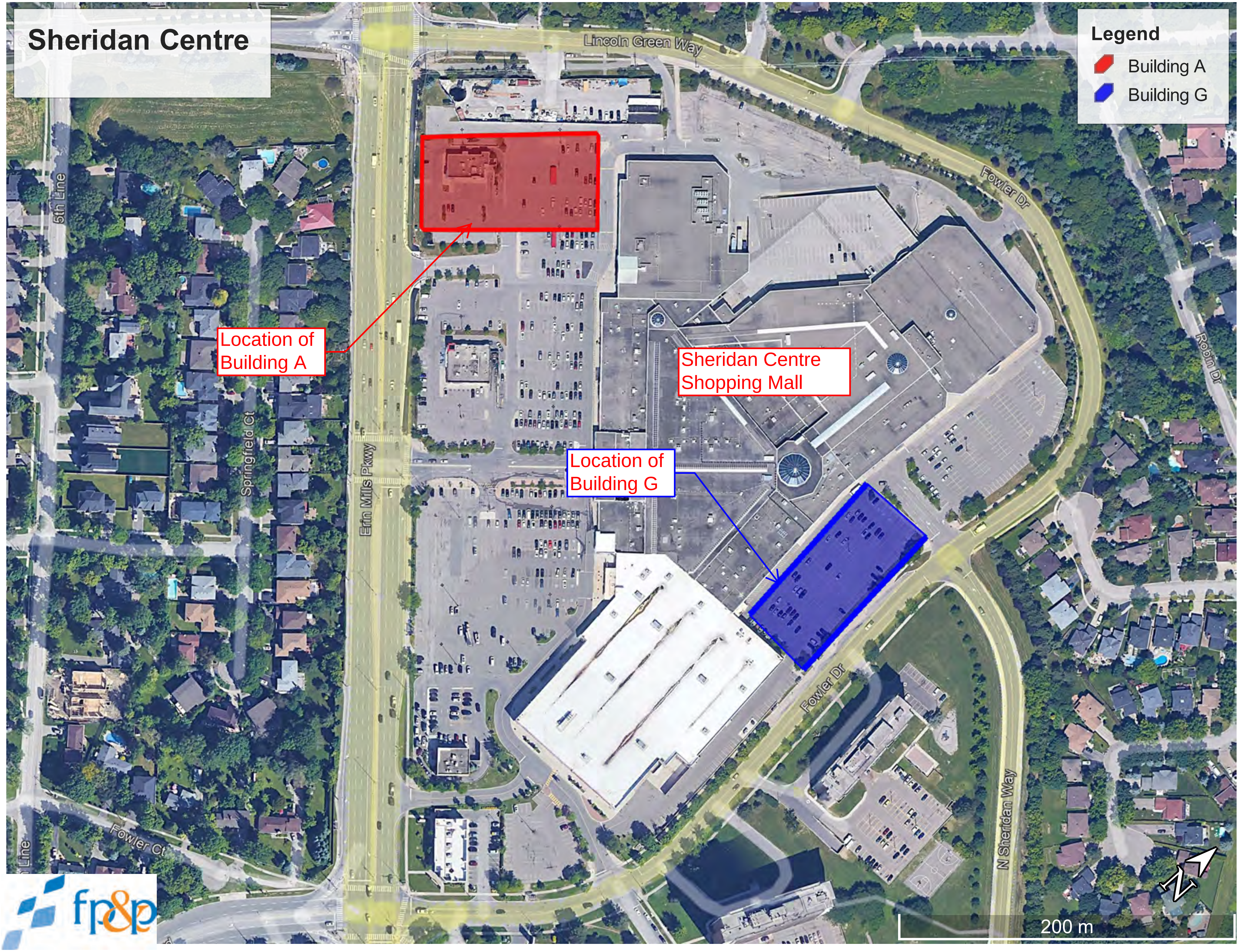
Tel: +1.905.264.2420 x360
E-Mail: astefenatti@fabianpapa.com

Paolo Albanese, P.Eng.
PEO Designated Consulting Engineer
Partner

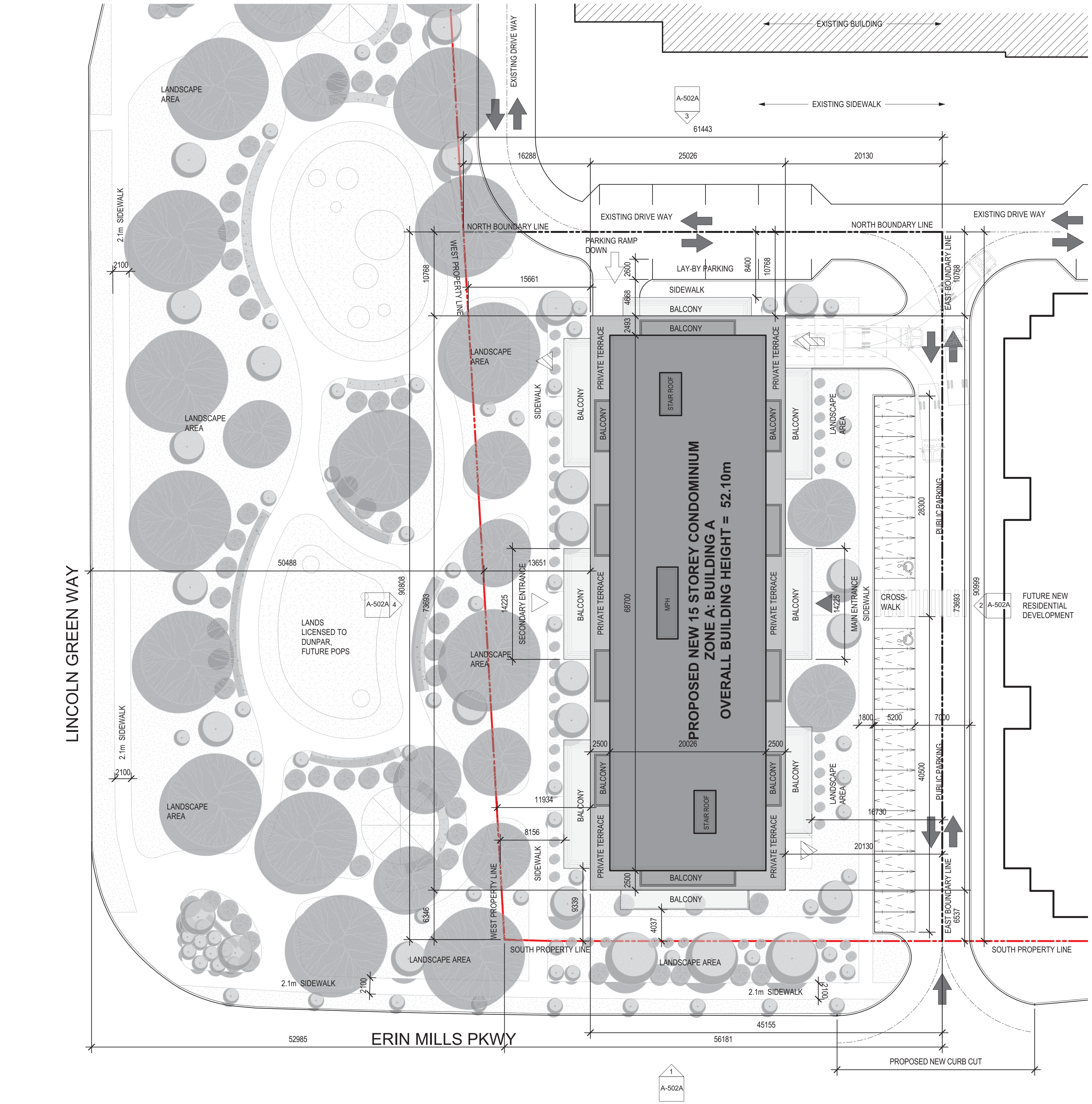
Tel: +1.905.264.2420 x420
E-Mail: palbanese@fabianpapa.com

APPENDIX A

Key Map with Architectural Drawings -



Sheridan Centre
Legend
Building A
Building G
Lincoln Green Way
Fowler Dr
5th Line
Robin Dr
Location of Building A
Sheridan Centre Shopping Mall
Location of Building G
Springfield Ct
Erin Mills Pkwy
Fowler Dr
N Sheridan Way
Fowler Ct
Line
N
200 m
fp&p



CONTEXT PLAN

PROJECT NORTH

LEGEND

- BUILDING SECONDARY ENTRANCE
- BUILDING EXIT
- BUILDING ENTRANCE
- BLOCK BOUNDARY LINE
- PROPERTY LINE
- RAMP TO UNDERGROUND PARKING LEVELS
- OVERHEAD DOOR
- PRIVATE DRIVEWAY

PROJECT STATISTICS - 2225 ERIN MILLS PARKWAY - ZONE A CONDOMINIUM TOWER

Proposed Residential Development
Etobicoke, Ontario

New 15 Storey Mid Rise
2022-12-14

1.0 Site Area (ZONE A)

	acres	sq.m.	sq.ft.
Total	1.32	5,351.00	57,597.68

2.0 GFA

2.1 Proposed Residential Above Grade GFA

	Levels	Deductions per floor sq.m	Total GFA sq.m.	Total GFA sq.ft.
Level 1	1	TBD	1,672	17,997
Level 2 to 6	5	TBD	8,915	95,960
Level 7 to 15	9	TBD	11,835	127,391
Total	15		22,422.00	241,348.40

2.2 Proposed Residential GFA Below Grade

	Levels	(deductions per floor) sq.m.	Total GFA sq.m.	Total GFA sq.ft.
Level P2	1 x	TBD	TBD	#VALUE!
Level P1	1 x	TBD	TBD	#VALUE!
Total	2		0.00	0.00

2.3 Proposed Total GFA

	sq.m.	sq.ft.
Total Residential GFA (ZONE A)	22,422.00	241,348.40

2.4 Propsed Floor Space Index (FSI) ZONE A

Total
4.19

5.0 Proposed Unit Count (Required min 10% of 3BR suite)

	Levels	Studio	1BR	1BR+D	2BR.	2BR+D	3B	Units
Level 1	1	0	0	0	0	0	0	0
Level 2 to Level 6	5	0	1	2	12	6	0	105
Level 7 to Level 12	6	0	8	2	0	4	2	96
Level 13	1	0	8	2	0	4	2	16
Level 14	1	0	8	2	0	4	2	16
Level 15	1	0	8	2	0	4	2	16
Total	15	0	77	28	60	66	18	249
		0%	31%	11%	24%	27%	7%	100%

6.0 Parking

6.1 Required Vehicular Parking Calculation

	Parking Ratio	Parking Spaces
Residents (0.8 spaces per unit for 1BR+D)	105 x 1	105
Residents (0.9 spaces per unit for 2BR)	126 x 1	126
Residents (1.1 spaces per unit for 3BR)	18 x 1	18
Visitors (0.15 spaces per unit)	249 x 0.15	37
Total		286

6.2 Provided Vehicular Parking Calculation

	Parking Spaces
Residents	249
Visitors	37
Total	286

6.3 Provided Vehicular Parking

	Levels	Residents	Visitors	Parking Spaces
Level P1	1 x	106	37	143
Level P2	1 x	143	0	143
Total				286

PLAN OF SURVEY AND TOPOGRAPHY

PART OF LOT 14, RANGE 3 CREDIT INDIAN RESERVE

CITY OF MISSISSAUGA

REGIONAL MUNICIPALITY OF PEEL

D
DUNPAR
BETTER BY DESIGN

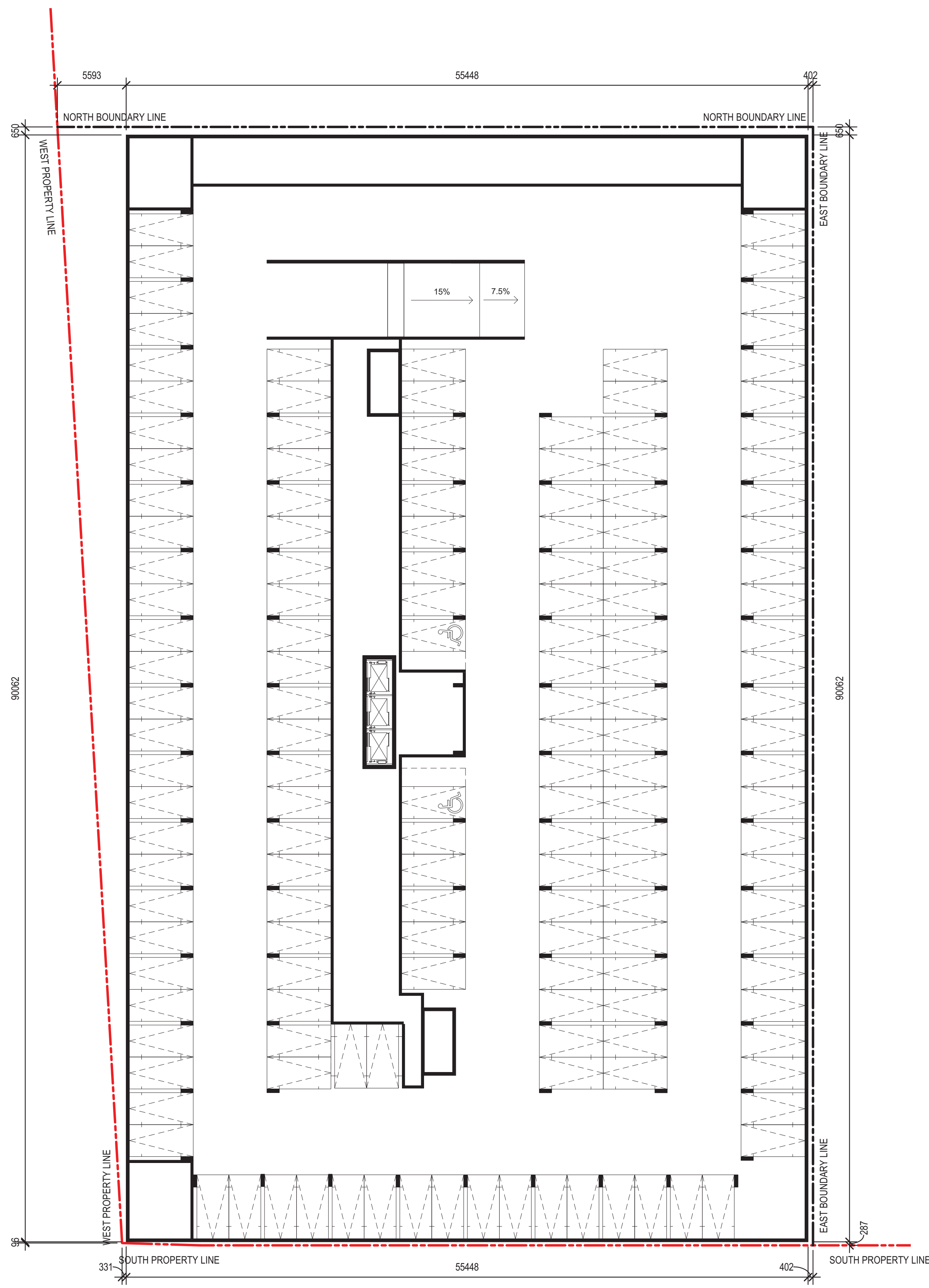
2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
ZONE A

Scale | North

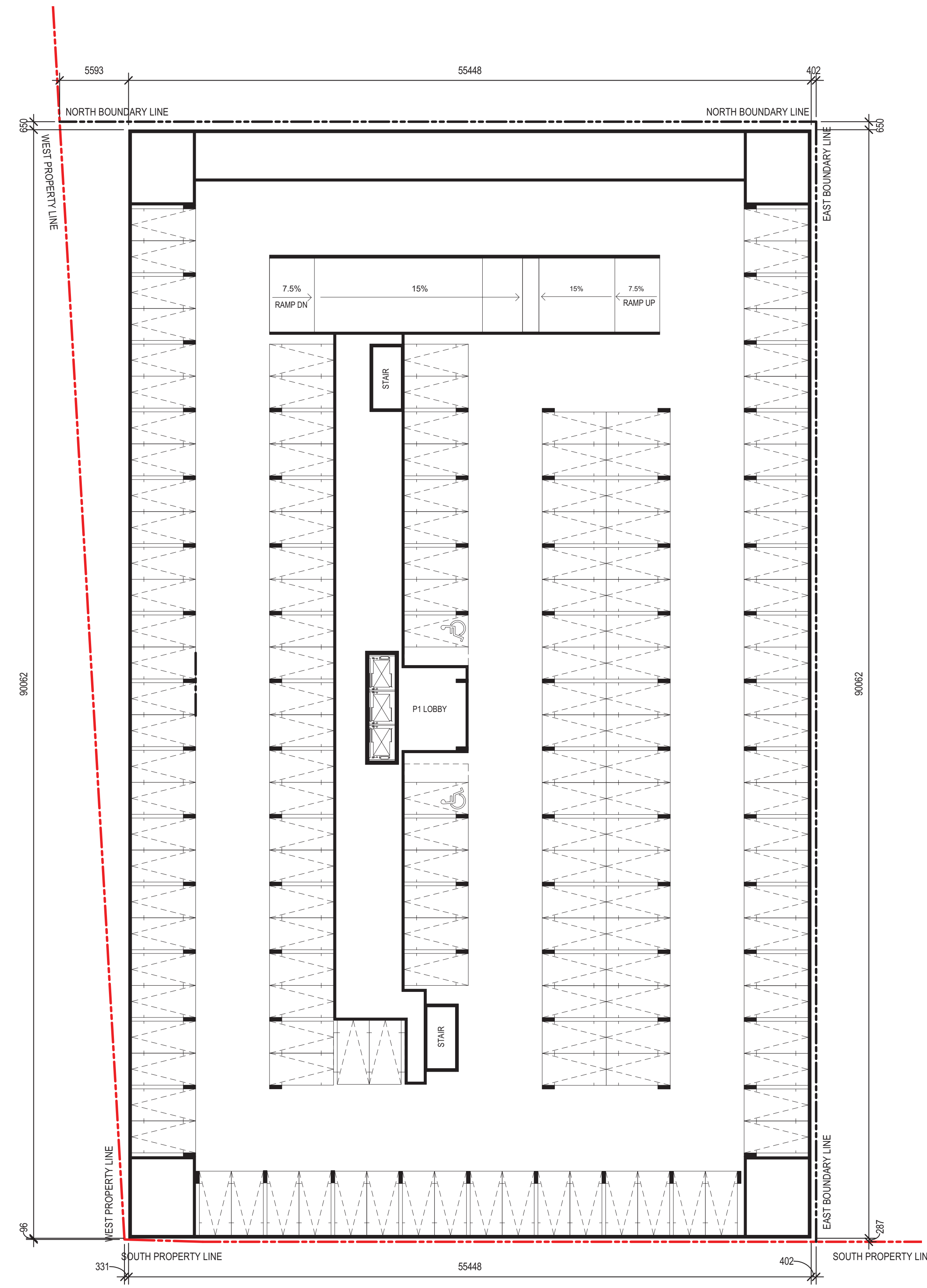
Drawing Title
SITE PLAN

Client

Drawn by	Author	Page No.
Checked by	Checker	
Date	11/06/18	A-100A



2 A-501A P2 Level
1 : 200



1 A-501A P1 Level
1 : 200

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DUNPAR
BETTER BY DESIGN

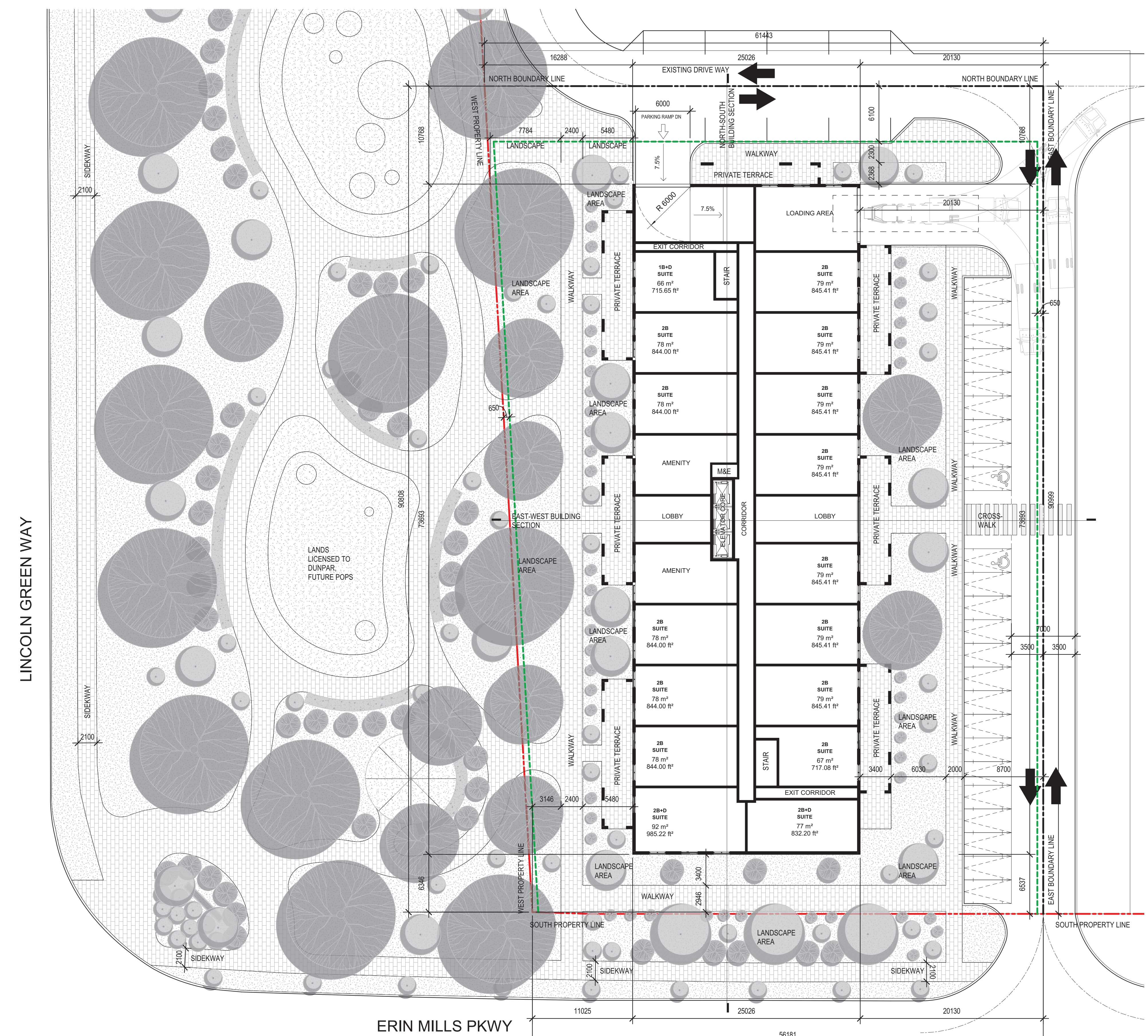
2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
ZONE A

Scale	North

Drawing Title
UNDERGROUND PARKING
LEVEL P1 AND P2

Client

Drawn by	Author	Page No.
Checked by	Checker	A-101A
Date	11/29/22	



D
DUNPAR
BETTER BY DESIGN

2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
ZONE A

Scale	North
Drawing Title	GROUND FLOOR PLAN
Client	----------
Drawn by: Author	Page No.
Checked by: Checker	A-201A
Date: 11/29/22	

CONTEXT PLAN

PROJECT NORTH

PROJECT STATISTICS

New 15 Storey Mid Rise

2022-12-14

1.0 Site Area (ZONE G)

	acres	sq.m.	sq.ft.
Total	1.16	4,705.00	50,644.20

2.0 GFA

2.1 Proposed Residential Above Grade GFA (BUILDING G1)

	Levels	Deductions per floor sq.m	Total GFA sq.m.	Total GFA sq.ft.
Level 1	1	TBD	1,834	19,741
Level 2 to 7	6	TBD	12,984	139,759
Level 8 to 12	5	TBD	10,760	115,820
Level 13 to 15	3	TBD	4,962	53,411
Roof	1	TBD	-	-
Total	15		30,540.00	328,729.82

2.2 Proposed Residential GFA Below Grade

	Levels	(deductions per floor) sq.m.	Total GFA sq.m.	Total GFA sq.ft.
Level P1	1	TBD	TBD	#VALUE!
Level P2	1	TBD	TBD	#VALUE!
Level P3	1	TBD	TBD	#VALUE!
Level P4	1	TBD	TBD	#VALUE!
Total	4		0.00	0.00

2.3 Proposed Total GFA

	sq.m.	sq.ft.
Total Residential GFA (ZONE G)	30,540.00	328,729.82

2.4 Propsed Floor Space Index (FSI) ZONE G

Total
6.49

5.0 Proposed Unit Count (Required min 10% of 3BR suite)

	Levels	Studio	1BR	1BR+D	2BR.	2BR+D	3B	Units
Level 1	1	0	0	2	15	2	0	19
Level 2 to Level 7	6	0	3	2	17	4	0	156
Level 8 to Level 12	5	0	1	5	16	4	0	130
Level 13	1	0	3	15	0	0	4	22
Level 14	1	0	3	15	0	0	4	22
Level 15	1	0	3	15	0	0	4	22
Total	15	0	32	84	197	46	12	371
		0%	9%	23%	53%	12%	3%	100%

6.0 Parking

6.1 Required Vehicular Parking Calculation

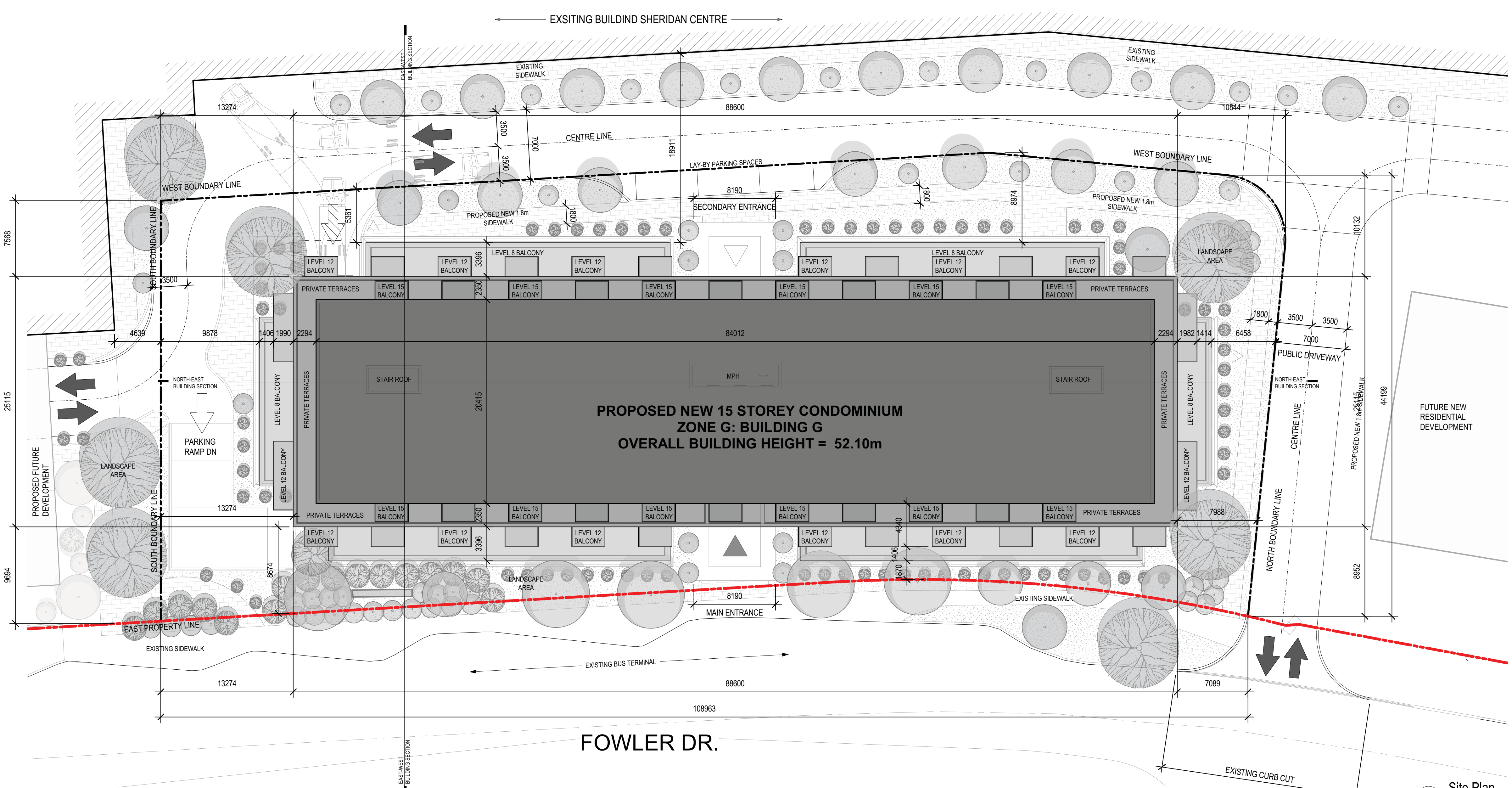
	Parking Ratio	Parking Spaces
Residents (0.8 spaces per unit for 1BR+D)	116 x 1	116
Residents (0.9 spaces per unit for 2BR)	243 x 1	243
Residents (1.1 spaces per unit for 3BR)	12 x 1	12
Visitors (0.15 spaces per unit)	371 x 0.15	56
Total		427

6.2 Provided Vehicular Parking Calculation

	Parking Spaces
Residents	371
Visitors	56
Total	427

6.3 Provided Vehicular Parking

	Levels	Residents	Visitors	Parking Spaces
Level P1	1 x	44	56	100
Level P2	1 x	119	0	119
Level P3	1x	119	0	119
Level P4	1x	89	0	89
Total				427



1 A-501G Site Plan 1 : 200

LEGEND

- BUILDING SECONDARY ENTRANCE
- BUILDING EXIT
- BUILDING ENTRANCE
- BLOCK BOUNDARY LINE
- PROPERTY LINE
- RAMP TO UNDERGROUND PARKING LEVELS
- OVERHEAD DOOR
- PRIVATE DRIVEWAY

PLAN OF SURVEY AND TOPOGRAPHY

PART OF LOT 14,
RANGE 3 CREDIT
INDIAN RESERVE

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY
OF PEEL

DUNPAR
BETTER BY DESIGN

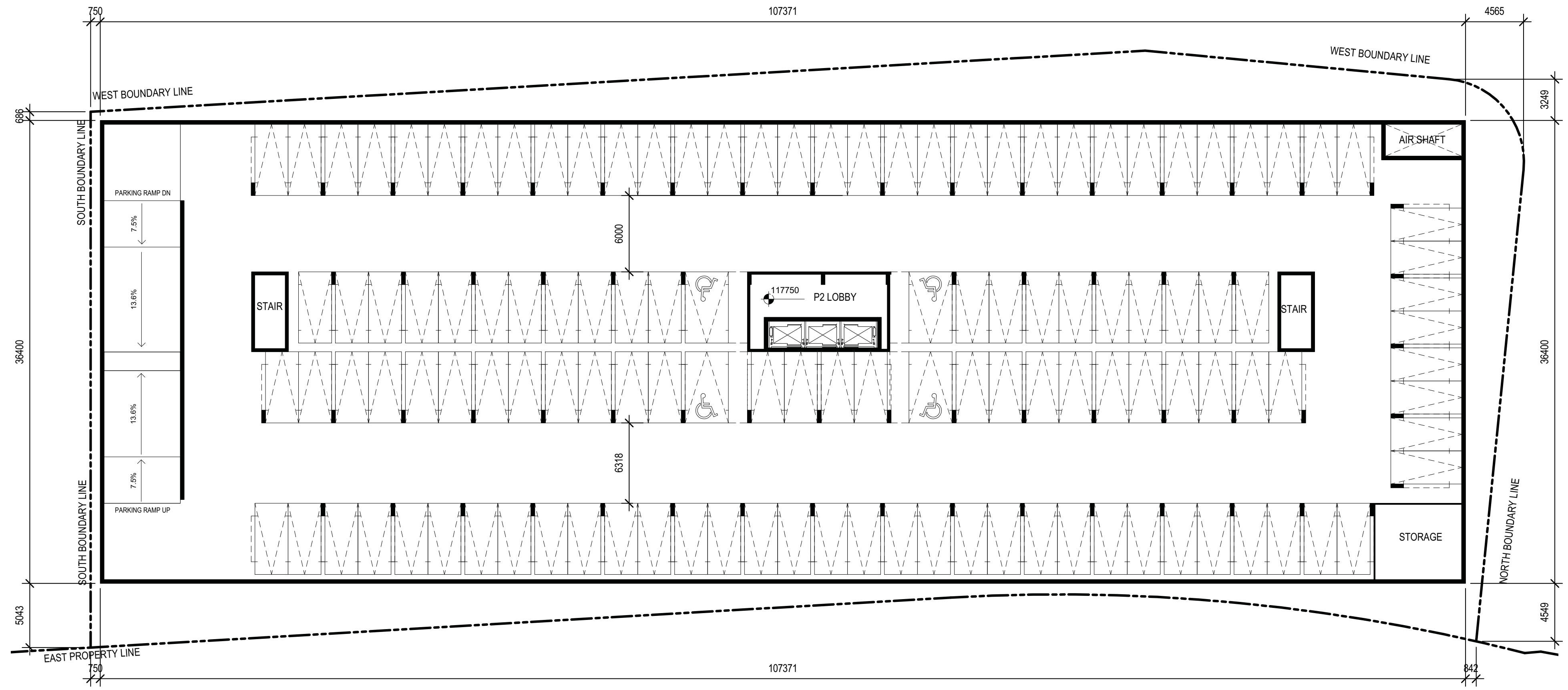
2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
TOWER ZONE G

Scale	North

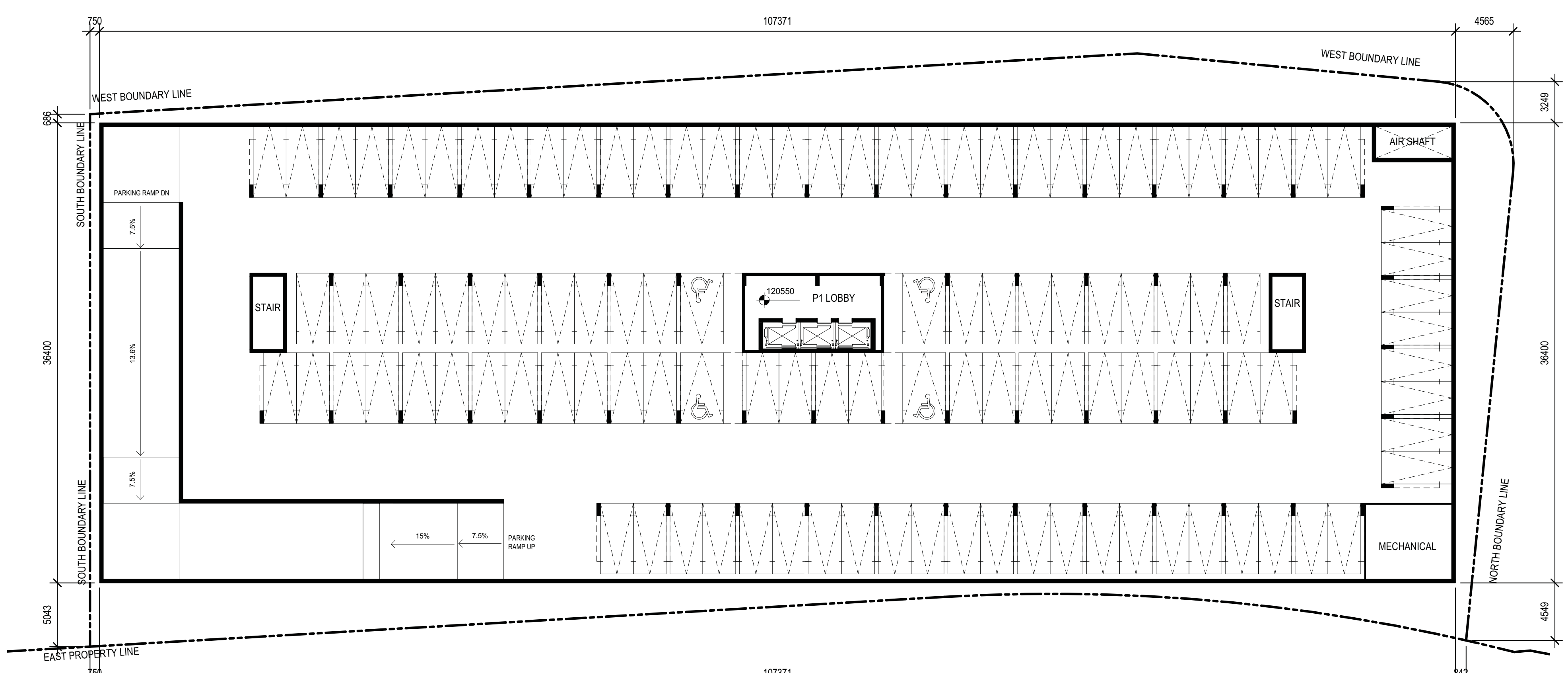
Drawing Title
SITE PLAN & PROJECT STATISTICS

Client

Drawn by	Author	Page No.
Checked by	Checker	
Date	09/15/21	A-001G



2 P2 LEVEL PLAN
1 : 200



1 P1 LEVEL PLAN
1 : 200

D
DUNPAR
BETTER BY DESIGN

2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
TOWER ZONE G

Scale	North
Drawing Title UNDERGROUND PARKING P1 TO P2 PLANS	
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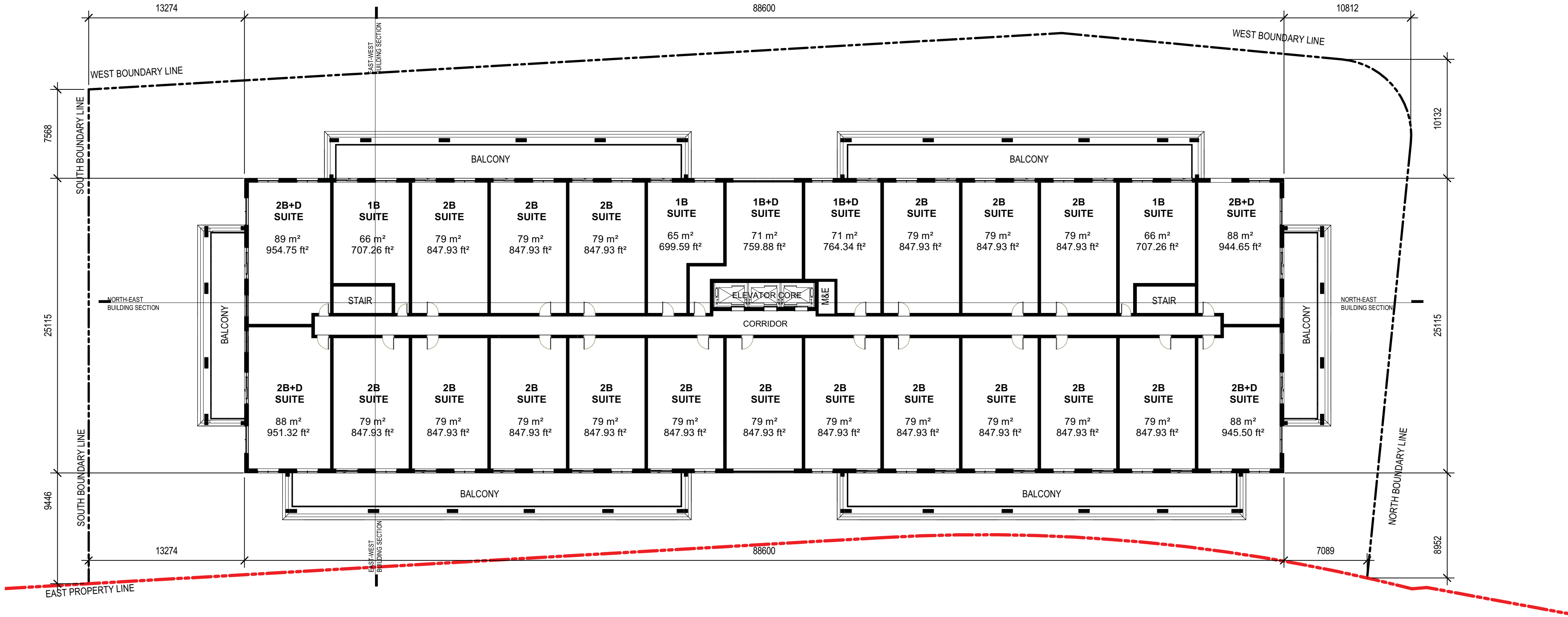
Checked by Checker	
Date 09/30/22	A-100G



3
A-501G
TYP. LEVEL 2 TO LEVE 7 FLOOR PLAN
1 : 200

2
A-501G
GROUND FLOOR PLAN
1 : 200

D
DUNPAR
BETTER BY DESIGN

2225 ERIN MILLS PARKWAY
PROPOSED NEW CONDOMINIUM
TOWER ZONE G

Scale	North

Drawing Title
GROUND TO LEVEL 7 FLOOR PLANS

Client

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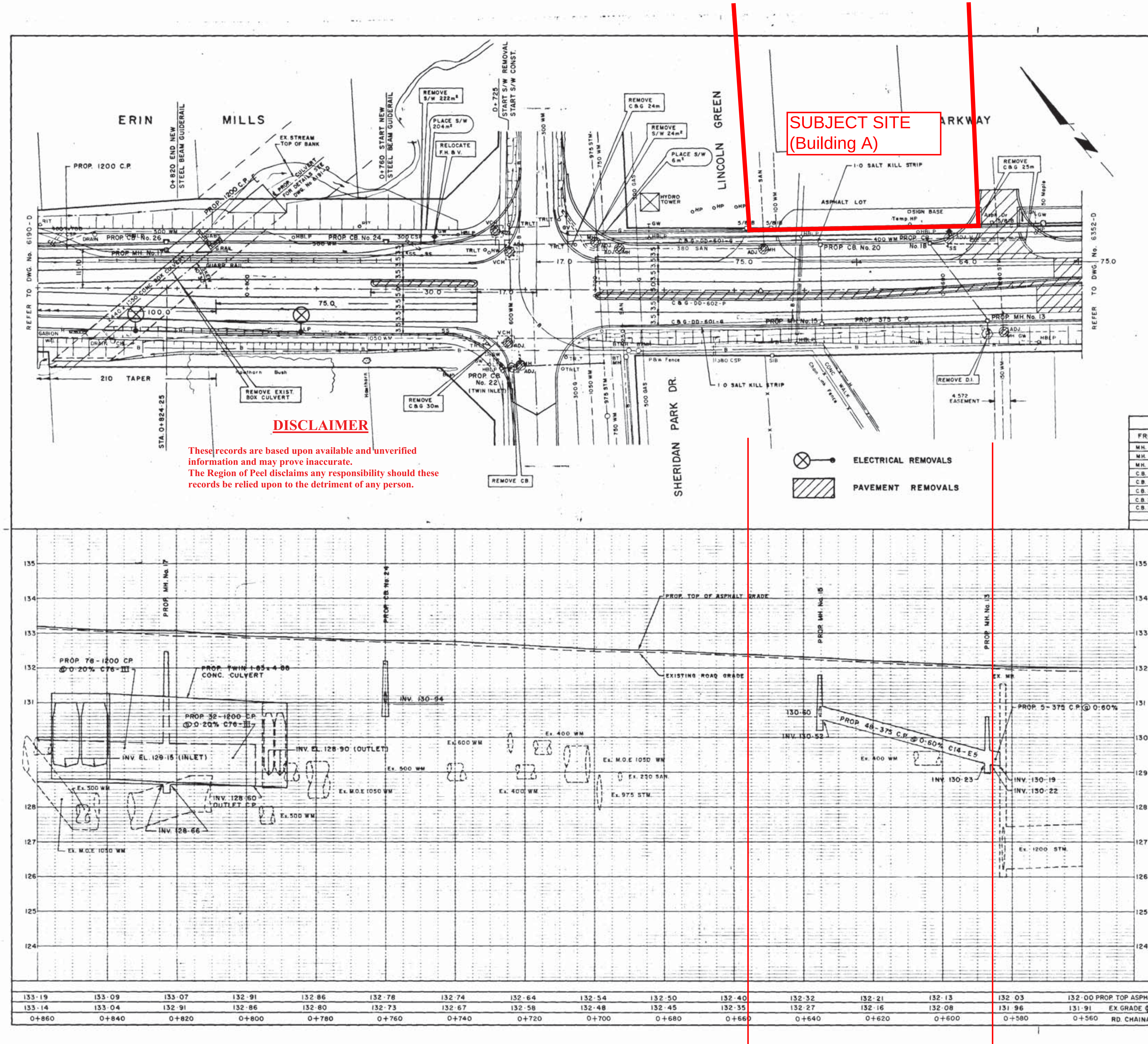
Date	09/15/21	A-200G

APPENDIX B

Plan & Profile Drawings - (Erin Mills Parkway, Region of Peel Easement & Fowler Drive)



DISCLAIMER

These records are based upon available and unverified information and may prove inaccurate.
The Region of Peel disclaims any responsibility should these records be relied upon to the detriment of any person.

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN. SEWERS			GAS MAINS		
STORM SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		

REVISIONS

DATE	DETAILS	INIT.
MAR. 1986	AS CONSTRUCTED	A.S.

MANHOLE AND CATCH BASIN DATA

No.	STA.	OFFSET	TYPE	GRATE	GRATE EL.
MH. 13	0+587	9.50L	DD-720	DD-713-B	131.88
MH. 15	0+635	9.50L	DD-720	DD-713-B	132.10
MH. 17	0+820	11.25R	DD-723	DD-704-B	132.74
C.B. 18	0+604	13.00R	DD-729A	DD-713-B	131.89
C.B. 20	0+636	13.00R	DD-729A	DD-713-B	132.03
C.B. 22	0+727	22.50L	DD-728	DD-713-B	132.38
C.B. 24	0+760	13.00R	DD-729A	DD-713-B	132.52
C.B. 26	0+822	13.00R	DD-729A	DD-713-B	132.75

SEWER DATA

FROM	TO	SIZE	CLASS	LGTH.	INLET	OUTLET
MH. 13	EX. MH.	375	C14-E.S.	5	130.22	130.19
MH. 15	MH. 13	375	C14-E.S.	48	130.52	130.23
MH. 17	OUTLET	1200	C76-III	32	128.66	128.60
C.B. 18	MH. 13	300	C14-E.S.	28	130.51	130.23
C.B. 20	MH. 15	300	C14-E.S.	23	130.83	130.60
C.B. 22	EX. PIPE	375	C14-E.S.	4	130.88	130.84
C.B. 24	OUTLET	375	C.S.P.-1.321	9	130.94	130.85
C.B. 26	MH. 17	300	C14-E.S.	1	129.27	129.26

General Notes

- -- All Driveways Gravel Unless Otherwise Noted.
- -- All Service Locations Are Approximate And Must Be Located Accurately In Field.
- ● Denotes Building – Not Located
- ⊏ Denotes Building Located
- Type 'B' Bedding Unless Otherwise Noted (SAN)

B.M. Nº Elev.

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by ____ Chkd. ____

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL TELEPHONE COMPANY
CONSUMERS GAS COMPANY
MINISTRY OF TRANSPORTATION
MINISTRY OF ENVIRONMENT
HYDRO ELECTRIC POWER COMM. OF ONTARIO
HYDRO ELECTRIC COMM. CITY OF MISSISSAUGA
HYDRO ELECTRIC COMM. CITY OF BRAMPTON
HYDRO ELECTRIC COMM. PORT CREDIT
HYDRO ELECTRIC COMM. STREETSVILLE
CABLE TELEVISION

Region of Peel

Department of Public Works

ERIN MILLS PARKWAY

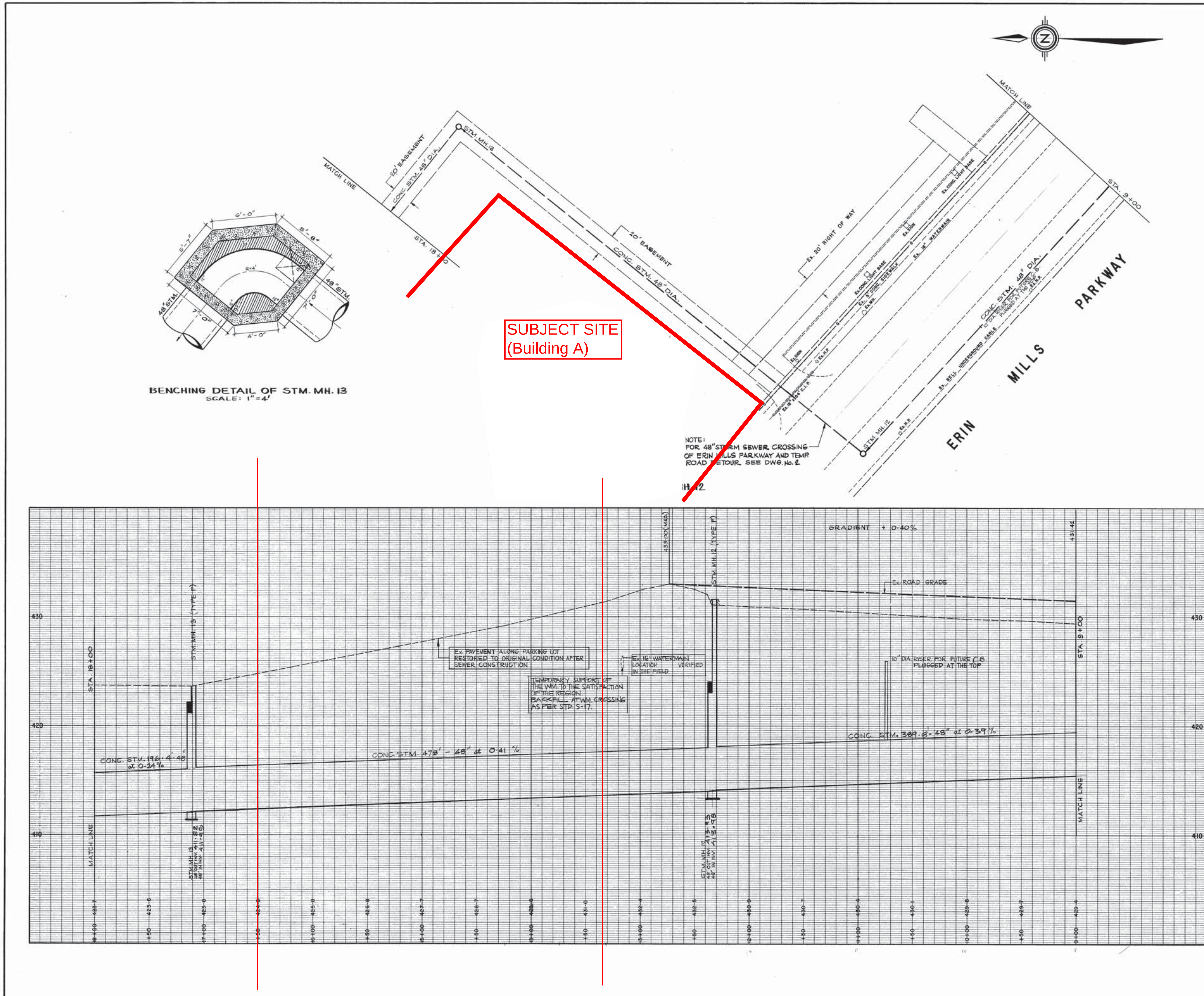
(REGION ROAD No. 1)

PAVEMENT & ELECTRICAL REMOVALS

Sta. 0+560 **To Sta.** 0+860

Lots 31 Con. 1, S.D.S.	Area Z-18	Project No. 83-4010
Scale Hor. 1:500 Ver. 1:50	Drawn by A.S.	Checked by
Date FEB. 1984	Sheet 3 of 4	Plan No. 6353-D

6353-D



SERVICE	DATA				
SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN. SEWERS			GAS MAINS		
STORM SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		

REVISIONS

DATE	DETAILS	INIT.

REQUIRED ROAD BASE THICKNESS

STORM SEWER DATA

1. ALL CONCRETE PIPES UP TO AND INCLUDING 18" SHALL BE A.S.T.M. C-14-65 S.S. UNLESS OTHERWISE NOTED.
2. ALL CONCRETE PIPES 21" AND OVER SHALL BE A.S.T.M. C-76-65 T CLASS III.
3. ALL PIPES WITH STANDARD RUBBER GASKETS WITHOUT MORTAR.
4. BEDDING AS PER TOWN STD. DWG. A-5508 TYPE B UNLESS OTHERWISE NOTED.
5. MANHOLES AS SHOWN ON PROFILES.

SANITARY SEWER DATA

1. ALL CONCRETE PIPES SHALL BE A.S.T.M. C-14-65 S.S. UNLESS OTHERWISE NOTED.
2. ALL PIPES WITH PRIME RUBBER GASKETS WITHOUT MORTAR.
3. BEDDING AS PER TOWN STD. DWG. A-5508 TYPE B UNLESS OTHERWISE NOTED.
4. MANHOLES AS SHOWN ON PROFILES.
5. ALL ASBESTOS CEMENT PIPES SHALL BE EQUAL TO A.S.T.M. SPECIFICATIONS C-428 CLASS 2400 OR LATEST AMENDMENT.
6. THE MAXIMUM ALLOWABLE TRENCH WIDTH AT THE TOP OF THE PIPE SHALL NOT EXCEED THE FOLLOWING: 10"-12" PIPE: 3'-0", 15"-30" PIPE: O.D.+2'-0", 33"-72" PIPE: O.D.+2'-6"
7. THE CONTRACTOR IS RESPONSIBLE FOR SUPPLYING EXTRA BEDDING AND/OR STRONGER PIPE IF ACTUAL TRENCH WIDTHS EXCEED THE DESIGN WIDTHS.

WATERMAIN

D.I. CEMENT LINED PIPE WITH TYTON JOINT A.N.S.I. CLASS II, SIZE 4" TO 12" DIA.
EXISTING WATERMAIN AS SHOWN ON PLAN.
3/4" WATER CONNECTIONS SHOWN THUS W (SINGLE).
1" WATER CONNECTIONS SHOWN THUS W (DOUBLE).
WATER SERVICES ARE TO HAVE 4' MIN. HORIZONTAL CLEARANCE FROM ALL OTHER UTILITIES.
WATER SERVICES TO SINGLE AND SEMI-DETACHED LOTS TO BE LOCATED AS SHOWN ON PLAN.
WATERMAIN AND WATER SERVICES ARE TO HAVE 5'-6" MINIMUM COVER.

BENCH MARK

IN THE CENTRE OF RED BRICK CHIMNEY ON THE NORTH FACE OF A STUCCO HOUSE Nº 2239 ON THE EAST SIDE OF FIFTH LINE WEST 800 FEET MORE OR LESS NORTH OF FOWLER DR.
ELEVATION 447.79

NOTE

WHERE BELL AND HYDRO CABLES ARE TO BE INSTALLED ALONG SIDE LOT LINES, SEWER CONNECTION AT RESPECTIVE LOT LINES IN THIS SET OF DRAWINGS MUST BE REPLACED BY SINGLES LOCATED AT LEAST FIVE FEET FROM THE LOT LINES
THE SAME REVISION WILL APPLY TO EXISTING TREES DEDICATED TO BE PRESERVED AND CONNECTIONS TO BE BORED UNDER TREES.

AS CONSTRUCTED, MARCH 1980

REGISTERED PROFESSIONAL ENGINEER
B. THOMAS
PROVINCE OF ONTARIO

DESIGNED BY ______ CHK'D. ______ | APPROVED BY ______

FRED SCHAEFFER & ASSOCIATES INC
Urban Planners and Consulting Engineers
465 Wilson Avenue
Downsview, Ontario, M3H 1T9
Telephone (416) 635-5424

J. D. S. MONRO FARM

CITY OF MISSISSAUGA
COUNTY OF PEEL
ENGINEERING DEPARTMENT

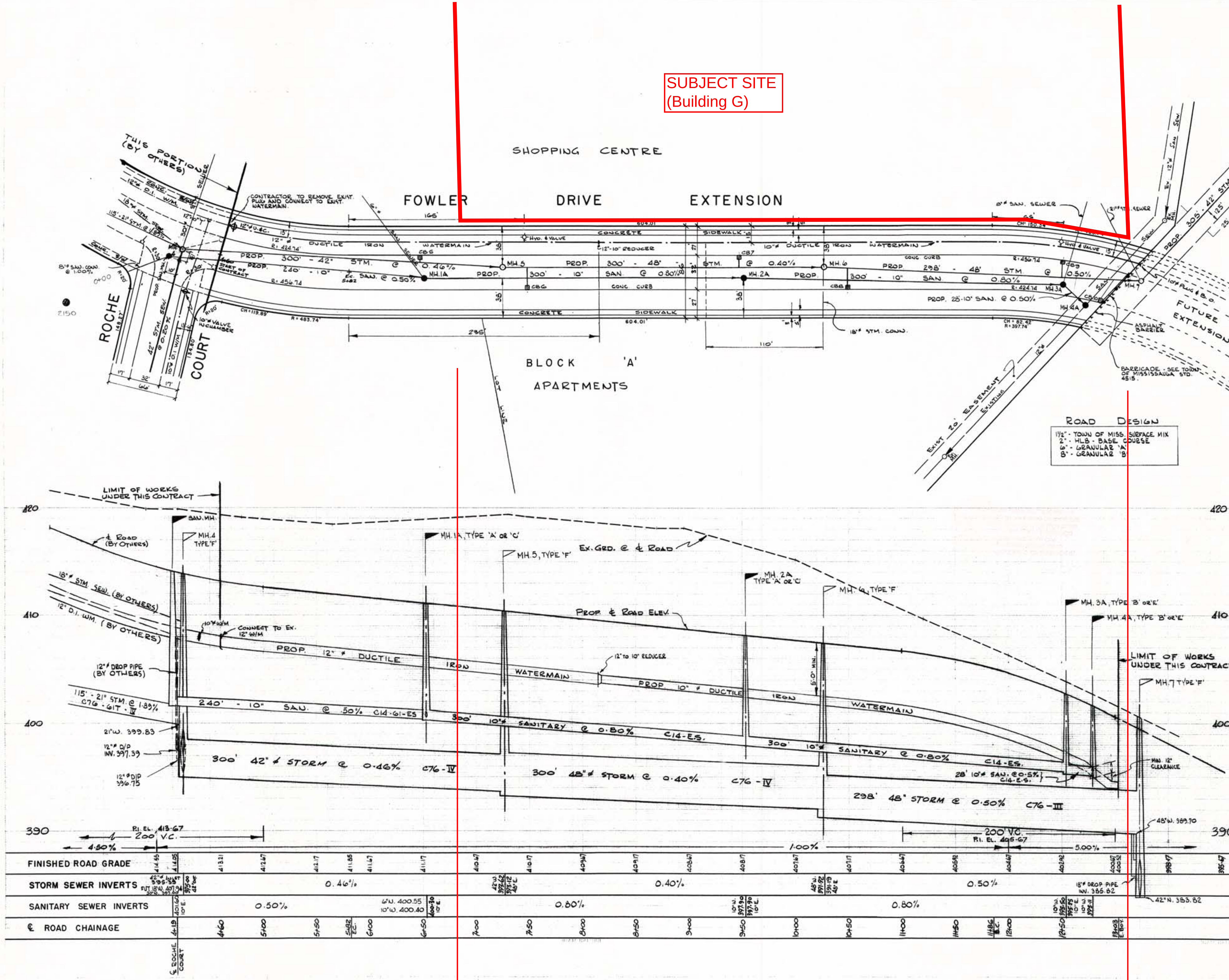
PLAN AND PROFILE
OF
STORM SEWER OUTFALL ON ERIN MILLS PARKWAY
ALONG 20' EASEMENT
FROM STA. 9+00 TO STA. 18+00

PART OF LOT	CON.	PROJECT No. 74-ES-287
SCALE: HOR. 1" = 40' VER. 1" = 4'	DRAWN BY A.M. Roberto	CHECKED BY B.THOMAS P.Eng.
DATE: NOV. 1976	SHEET 16 OF 19	PLAN No. C-17174

C.17174

P.N. 73-113

13



NOTES

1. STANDARD RUBBER GASKET JOINTS TO BE USED THROUGHOUT STORM SEWER SYSTEM AND PREMIUM RUBBER GASKETS THROUGHOUT THE SANITARY SEWER SYSTEM.
2. STANDARD TOWN OF MISSISSAUGA DRAWINGS TO BE CONSTITUTED AS PART OF THIS CONTRACT.
3. BENCH MARK: No. 477 ON THE EAST FACE AT THE NORTH CORNER OF GARAGE OF A RED BRICK HOUSE No. 2094 AT THE NORTH WEST CORNER OF SOUTHDOWN RD. AND NORTH SHERIDAN WAY. ELEV. 425.72.
4. ALL CURB RADII TO BE 25'-0" UNLESS OTHERWISE INDICATED.
5. ALL CATCH-BASIN LEADS TO BE 10" CONC. C 14-61-ES UNLESS OTHERWISE NOTED.
6. RISERS TO BE INSTALLED FOR SANITARY SEWER CONNECTIONS TO CROSS OVER STORM SEWER WHERE NECESSARY.
7. ALL DIMENSIONS TO BE CHECKED AND VERIFIED ON THE SITE. ANY DISCREPANCIES SHALL BE REPORTED TO THE ENGINEER.
8. PROP. 3/4" DIA. WATER SERVICE SHOWN THUS ----- TO BE INSTALLED AT ℄ OF EACH LOT UNLESS OTHERWISE NOTED.
9. CLASS "B" BEDDING REQUIRED THROUGHOUT UNLESS OTHERWISE NOTED.
10. FOWLER DR. EXT. TO BE CONSTRUCTED ACCORDING TO TOWN OF MISSISSAUGA STD. A-5491 WITH A TOTAL PAVEMENT THICKNESS OF 17 ½".

DATE	REVISION	APPRVD.

REGISTERED PROFESSIONAL ENGINEER
J. D. HANSON
PROVINCE OF ONTARIO

TOWN OF MISSISSAUGA

FOWLER DRIVE EXTENSION

ROCHE APARTMENT SITE P.N. 59-68

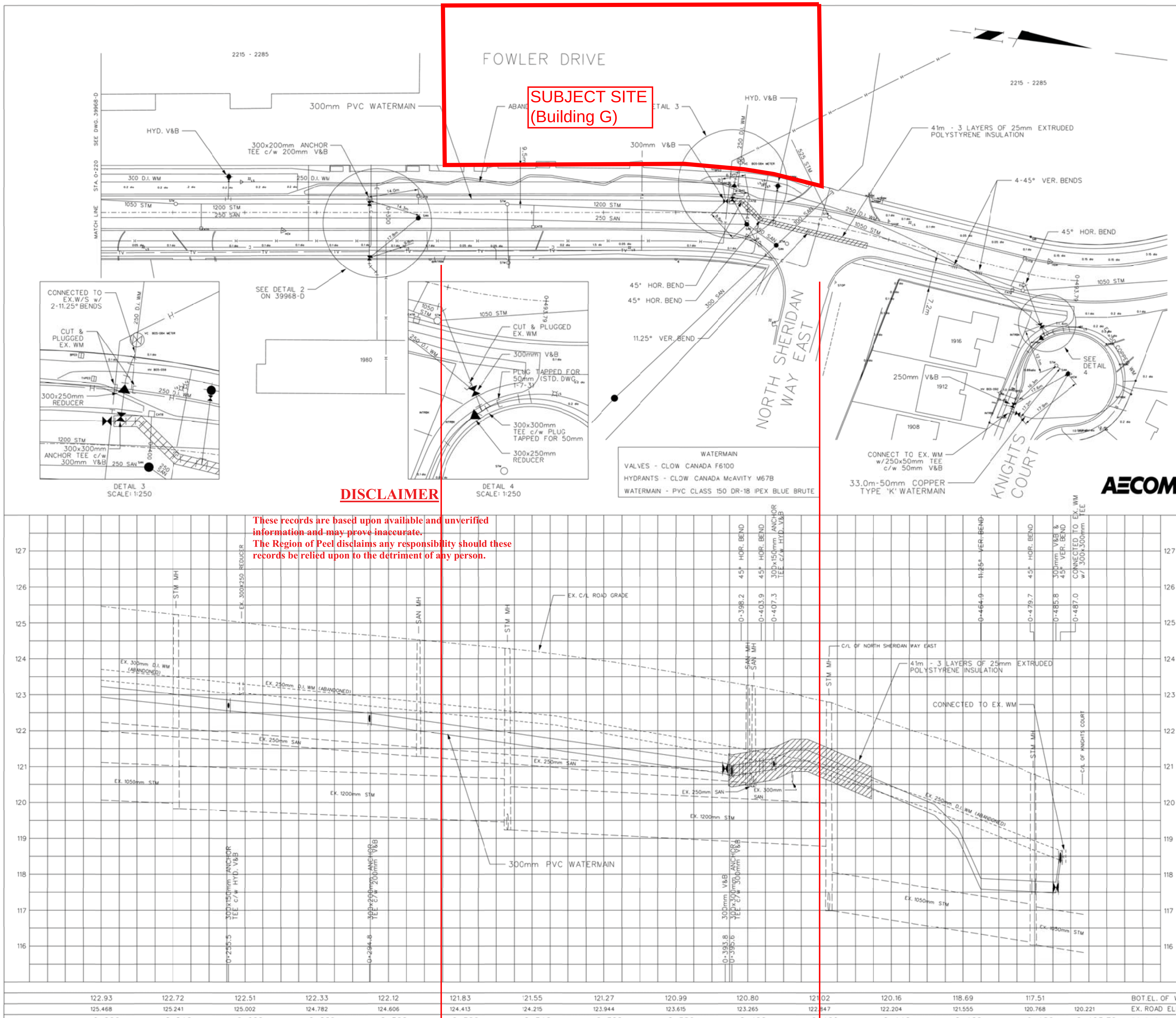
SCALE	HOR. 1" = 40'-0" VERT. 1" = 4'-0"
DATE	APRIL 1968
DRAWN BY	A. J.
CHECKED BY	J. D. H.
DRAWING NO.	67 - 48 - 7
SHEET	1 OF 1

G. ERIC HANSON ASSOCIATES LIMITED
PROFESSIONAL ENGINEERS & TOWN PLANNERS TORONTO

C-9620

C-09620

59-68



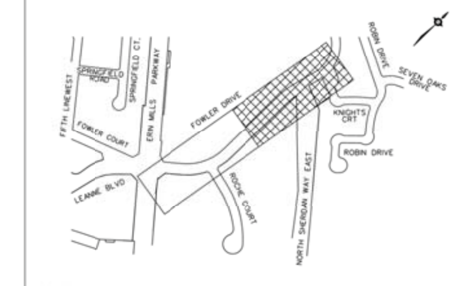
DISCLAIMER

These records are based upon available and unverified information and may prove inaccurate.
The Region of Peel disclaims any responsibility should these records be relied upon to the detriment of any person.

SERVICE DATA					
SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS			GAS MAINS		
STORM SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		
TRANSIT			HYDRO ONE		
PARKS & REC.			CTV		
ONT. CLEAN WATER			COMMUNIC. CABLES		

REVISIONS

DATE	DETAILS	INIT.
19/01/2010	90% SUBMISSION	SJ
17/03/2010	ISSUED FOR TENDER	C.E.A.
07/04/2010	ISSUED FOR CONSTRUCTION	SJ
13/12/2012	AS-BUILT SUBMISSION	PK



KEY PLAN (N.T.S.)

FOR GENERAL NOTES AND DETAILS SEE DRAWING No. 39965-D (KENNA COURT)

LEGEND

W/S 20mm DIA. or 25mm DIA. COPPER AS STATED ON THE DRAWING.
35 — HOUSE NUMBER
25mmC — PROPOSED SERVICE SIZE AND TYPE

Disclaimer

As-constructed dimensions, measurements and other details contained in this record drawing have been obtained by AECOM CANADA LTD. ("AECOM") from various sources. Such information represents the best information available to AECOM at the time of preparation of this record drawing. AECOM does not in any way represent or warrant that such information is accurate and assumes no responsibility for any errors or omissions contained therein.

General Notes

All Driveways ASPHALT Unless Otherwise Noted.
All Service Locations Are Approximate And Must Be Located Accurately In The Field
Denotes Building - Not Located
Denotes Building Located
Type 'B' Bedding Unless Otherwise Noted (SAN)

B.M. No. Elev.

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction Location of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by ____ Chkd. ____ Approved by ____

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL	CABLE TELEVISION/FIBREOPTIC PROVIDERS:
CITY OF MISSISSAUGA WORKS DEPT.	BELL CANADA
CITY OF BRAMPTON WORKS DEPT.	ENERSOURCE TELECOM
TOWN OF CALEDON WORKS DEPT.	HYDRO ONE TELECOM
BELL CANADA	ROGERS CABLE
ENBRIDGE INCORPORATED-GAS DISTRIBUTION	ALLSTREAM
ONTARIO MINISTRY OF TRANSPORTATION	PSN (PUBLIC SECTOR NETWORK)
ONTARIO CLEAN WATER AGENCY	FUTUREWAY (FCI BROADBAND)
HYDRO ONE NETWORKS	
ENERSOURCE, HYDRO MISSISSAUGA	
HYDRO ONE BRAMPTON	

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

FOWLER DRIVE

FROM ERIN MILLS PARKWAY TO KNIGHTS COURT
300mm PVC WATERMAIN
50mm COPPER WATERMAIN
STA. 0+220 TO STA. 0+493.79

CAD Area	Area Z-17	Project No. 09-1346S
Checked by	Drawn by SJ	
Date AUGUST 2011	Sheet 2 of 2	Plan No. 39969-D

APPENDIX C

Water Demand & Capacity Analysis Calculations -

2225 - Erin Mills Parkway, Mississauga - Residential Development

Building A - Water Demand Calculations

Designed By: **Benjamin Schrempf**
Checked By: **Paolo Albanese**, P.Eng
File No.: 23007
Date: 08 May 2023

Domestic Water Supply Demands:
Per Region of Peel Watermain Design Criteria for Water Distribution Systems
- assume Average Day demand is 280 L/capita/day for residential uses
- assume Average Day demand is 300 L/capita/day for ICI uses
- assume Population Density (see chart)

Unit Type	Population Density
Residential Apartments	2.7 Pers / unit
Commercial	50 Pers / hectare

Building	Building Data		Population[3]	Ave. Day Flow	Peak Hour, ADxPH [1]	Max. Day, ADxMD [2]
	Units	(sq.m)	pers	(L/s)	(L/s)	(L/s)
Apartments	249	n/a	673	2.18	6.54	4.36
Retail	n/a	n/a	0	0.00	0.00	0.00
Office	n/a	n/a	0	0.00	0.00	0.00
Total	249		673	2.2	6.5	4.4

[1] Peak Hour Demand, PHD, is 3.0 for residential and 3.0 for ICI
[2] Max Day Demand, MDD, is 2.0 for residential and 1.4 for ICI
[3] Population based on 2.7 people/unit

Fire Protection Supply Demands:
Per Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey

STEP 1: Calculate Fire Flow

$$F = 220 \cdot C \cdot \sqrt{A} \cdot (\text{various adjustments}) \ \text{L/min}$$

C = Coefficient related to type of construction:
= 1.5 for wood frame construction (Structure essentially all combustible)
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non combustible construction (unprotected metal structure components, masonry or metal walls)
= 0.6 for fire resistive construction (fully protected frame, floors, roof)

C =	0.6		
Largest Floor Area =	1783	m^2	
Floor Area Above =	1783	m^2	
Floor Area Below =	1783	m^2	
A =	2,675	m^2	Largest Floor + 25% x (Floor Above + Floor Below)
F =	6,826	L/min	
F =	7,000	L/min	Round to the nearest 1000

STEP 2: Adjust for building occupancy (Note: Number shall not be less then 2000 L/min)

= - 25% (Non-Combustible)
= - 15% (Limited Combustible)
= 0 (Combustible)
= + 15% (Free Burning)
= + 25% (Rapid Burning)

Factor =	-15%	
F1 = F x Factor =	5,950	L/min

fp&p

2225 - Erin Mills Parkway, Mississauga - Residential Development

Building A - Water Demand Calculations

STEP 3: Decrease F1 if building contains fire suppression system

= - 50% (Automatic Sprinklers)
= - 30% (Adequately Designed System)
= Additional -10% if the water supply is standard for the system and the fire department hose lines required
= Additional -10% if the system is fully supervised

Factor =	40%	
F2 = F1 x Factor =	2,380	L/min

STEP 4: Increase F1 due to exposure / close pr Distances = NE 31.5m / SW > 45m / NW > 45m / SE 24.7m

= 25% (0m to 3m) Factors = 5% + 0% + 0% + 10%
= 20% (3.1m to 10m)
= 15% (10.1m to 20m) Factor = 15% (max 75%)
= 10% (20.1m to 30.1m) F3 = F1 x Factor = 893 L/min
= 5% (30.1m to 45m)
= 0% (Greater then 45m)

STEP 5: Calculate Fire Flow (Note: Fire flow shall not be less then 2000 L/min or greater then 45,000 L/min)

Fire Flow = F1 - F2 + F3

F1 =	5,950	L/min	
- F2 =	2,380	L/min	
+ F3 =	893	L/min	
Fire Flow =	4,463	L/min	
Fire Flow =	4,000	L/min	Round to the nearest 1000
Fire Flow =	66.7	L/s	

STEP 6: Calculate Total Water Demand (Max Day Demand + Fire Flow)

Recall Max Day Demand (from chart above) =	4.4	L/s
TOTAL Fire Demand =	71.0	L/s

2225 - Erin Mills Parkway, Mississauga - Residential Development

Building A - Supply Line Head Loss Calculations

Designed By: **Alessandro Stefenatti**, B.Eng
Checked By: **Paolo Albanese**, P.Eng
File No.: 23007
Date: 08 May 2023

Total Fire Demand = 71.0 L/s (Taken From Fire Flow Spreadsheet)

Hydrant Flow Test Results

Flow (gpm)	Flow (L/s)	Pressure (psi)	Pressure (kPa)
0	0.0	65	450
1,126	71	56	389
2,000	126	40	276

Assumed Data Values

Estimated available Fire Flow calculated using:

$$Q_R = Q_F \times \frac{h_r^{0.54}}{h_f^{0.54}}$$

Hazen-Williams formula for watermain head loss:

$h_L = (10.675 * L * Q^{1.85}) / (C^{1.85} * D^{4.8655})$

where
h_L = pressure drop (m)
L = length of pipe (m)
Q = flow rate (m^3/s)
C = roughness coefficient
D = inside hydraulic diameter (m)

New 100 mm Domestic Watermain

L= 4.8 m
D= 100 mm
C= 100

L= 5.7 m
D= 150 mm
C= 100

Peak Hour Flow		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
6.5	0.01	0.1	3.1	0.11	0.78	64.9	447	**64.8**	**447**

[1] Residual pressure taken from above

New 150 mm Fire Line

L= 9.3 m
D= 150 mm
C= 100

Total Fire Flow (Max Day + Fire Flow)		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
71.0	0.07	1.52	59.7	2.16	14.86	56.4	389	**54.2**	**374**

[1] Residual pressure taken from above

2225 - Erin Mills Parkway, Mississauga - Residential Development

Building G - Water Demand Calculations

Designed By: **Benjamin Schrempf**
Checked By: **Paolo Albanese**, P.Eng
File No.: 23007
Date: 08 May 2023

Domestic Water Supply Demands:
Per Region of Peel Watermain Design Criteria for Water Distribution Systems
- assume Average Day demand is 280 L/capita/day for residential uses
- assume Average Day demand is 300 L/capita/day for ICI uses
- assume Population Density (see chart)

Unit Type	Population Density
Residential Apartments	2.7 Pers / unit
Commercial	50 Pers / hectare

Building	Building Data		Population[3]	Ave. Day Flow	Peak Hour, ADxPH [1]	Max. Day, ADxMD [2]
	Units	(sq.m)	pers	(L/s)	(L/s)	(L/s)
Apartments	371	n/a	1002	3.25	9.74	6.49
Retail	n/a	n/a	0	0.00	0.00	0.00
Office	n/a	n/a	0	0.00	0.00	0.00
Total	371		1002	3.2	9.7	6.5

[1] Peak Hour Demand, PHD, is 3.0 for residential and 3.0 for ICI
[2] Max Day Demand, MDD, is 2.0 for residential and 1.4 for ICI
[3] Population based on 2.7 people/unit

Fire Protection Supply Demands:
Per Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey

STEP 1: Calculate Fire Flow

$$F = 220 \cdot C \cdot \sqrt{A} \cdot (\text{various adjustments})\ \text{L/min}$$

C = Coefficient related to type of construction:
= 1.5 for wood frame construction (Structure essentially all combustible)
= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior)
= 0.8 for non combustible construction (unprotected metal structure components, masonry or metal walls)
= 0.6 for fire resistive construction (fully protected frame, floors, roof)

C =	0.6		
Largest Floor Area =	2164	m^2	
Floor Area Above =	2164	m^2	
Floor Area Below =	2164	m^2	
A =	3,246	m^2	Largest Floor + 25% x (Floor Above + Floor Below)
F =	7,521	L/min	
F =	8,000	L/min	Round to the nearest 1000

STEP 2: Adjust for building occupancy (Note: Number shall not be less then 2000 L/min)

= - 25% (Non-Combustible) — Factor = -15%
= - 15% (Limited Combustible) — F1 = F x Factor = 6,800 L/min
= 0 (Combustible)
= + 15% (Free Burning)
= + 25% (Rapid Burning)

2225 - Erin Mills Parkway, Mississauga - Residential Development
Building G - Water Demand Calculations

STEP 3: Decrease F1 if building contains fire suppression system
= - 50% (Automatic Sprinklers)
= - 30% (Adequately Designed System)
= Additional -10% if the water supply is standard for the system and the fire department hose lines required
= Additional -10% if the system is fully supervised

Factor =	40%	
F2 = F1 x Factor =	2,720	L/min

STEP 4: Increase F1 due to exposure / close pı Distances = N >16.1m / E > 45m / S 13.7m / W 20.3m

= 25% (0m to 3m) Factors = 15% + 0% + 15% + 10%
= 20% (3.1m to 10m)
= 15% (10.1m to 20m) Factor = 40% (max 75%)
= 10% (20.1m to 30.1m) F3 = F1 x Factor = 2,720 L/min
= 5% (30.1m to 45m)
= 0% (Greater then 45m)

STEP 5: Calculate Fire Flow (Note: Fire flow shall not be less then 2000 L/min or greater then 45,000 L/min)

Fire Flow = F1 - F2 + F3

F1 =	6,800	L/min	
- F2 =	2,720	L/min	
+ F3 =	2,720	L/min	
Fire Flow =	6,800	L/min	
Fire Flow =	7,000	L/min	Round to the nearest 1000
Fire Flow =	116.7	L/s	

STEP 6: Calculate Total Water Demand (Max Day Demand + Fire Flow)

Recall Max Day Demand (from chart above) =	6.5	L/s
TOTAL Fire Demand =	123.2	L/s

2225 - Erin Mills Parkway, Mississauga - Residential Development

Building G - Supply Line Head Loss Calculations

Designed By: **Alessandro Stefenatti**, B.Eng
Checked By: **Paolo Albanese**, P.Eng
File No.: 23007
Date: 08 May 2023

Total Fire Demand = 123.2 L/s (Taken From Fire Flow Spreadsheet)

Hydrant Flow Test Results

Flow (gpm)	Flow (L/s)	Pressure (psi)	Pressure (kPa)
0	0.0	65	450
1,952	123	41	283
2,000	126	40	276

Assumed Data Values

Estimated available Fire Flow calculated using:

$$Q_R = Q_F \times \frac{h_r^{0.54}}{h_f^{0.54}}$$

Hazen-Williams formula for watermain head loss:

$h_L = (10.675 * L * Q^{1.85}) / (C^{1.85} * D^{4.8655})$

where

- h_L = pressure drop (m)
- L = length of pipe (m)
- Q = flow rate (m^3/s)
- C = roughness coefficient
- D = inside hydraulic diameter (m)

New 100 mm Domestic Watermain

L= 4.9 m
D= 100 mm
C= 100

L= 9.0 m
D= 150 mm
C= 100

Peak Hour Flow		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
9.7	0.01	0.2	7.2	0.26	1.79	64.8	447	**64.5**	**445**

[1] Residual pressure taken from above

New 150 mm Fire Line

L= 12.5 m
D= 150 mm
C= 100

Total Fire Flow (Max Day + Fire Flow)		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
123.2	0.12	5.64	222.1	8.02	55.31	41.1	283	**33.1**	**228**

[1] Residual pressure taken from above

APPENDIX D

Sanitary Design Calculations -

2225 - Erin Mills Parkway, City of Mississauga, Region of Peel

Region of Peel - Engineering & Construction Services

SANITARY SEWER DESIGN SHEET

fp&p

NOTES

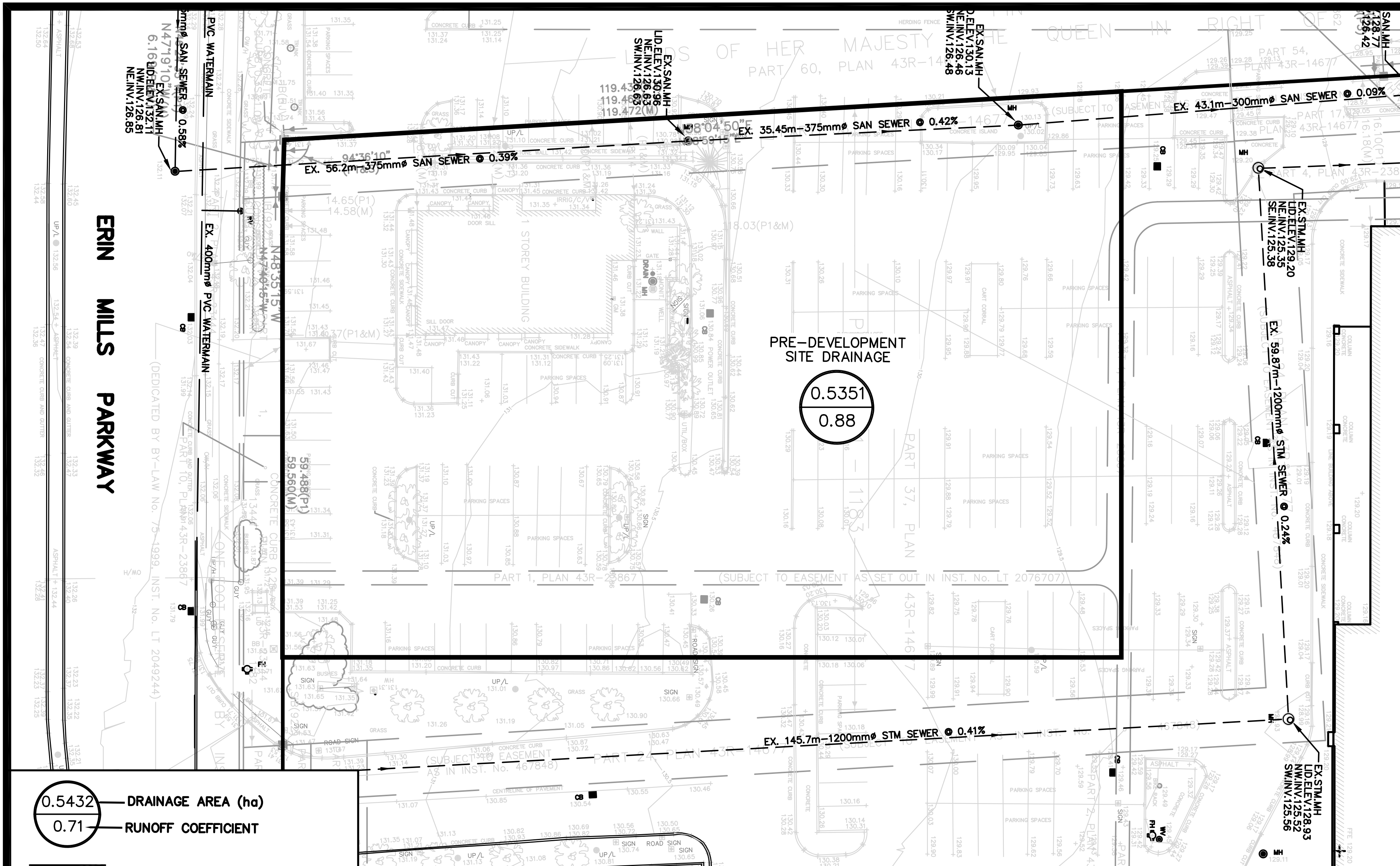
Pre-development domestic sewage flow based upon 302.8 Lpcd.
Post-development domestic sewage flow based upon a unit flow of 302.8 Lpcd.
Infiltration flow based upon a unit flow of 0.20 L/s/ha.
Maximum flow velocity for pipe flowing full = 3.5 m/s.
Minimum flow velocity for pipe flowing partially full (actual flow) = 0.75 m/s.

Designed By: **Alessandro Stefenatti**, B.Eng.
Checked By: **Paolo Albanese**, P.Eng.
File No.: 23007
Date: 08 May 2023

			DESIGN FLOW CALCULATIONS										SEWER DESIGN & ANALYSIS							
	from M.H.	to M.H.	Area (ha) or No. Units	Density (p/ha) OR (p/unit)	Population	Cumulative Area (ha)	Cumulative Population	Peaking Factor M	Sewage Flow (1) (L/s)	Infiltration Flow (2) (L/s)	Foundation Drain (3) (L/s)	Total Flow, Qd (1)+(2)+(3) (L/s)	Nominal Diameter (mm)	Pipe Slope (%)	Pipe Length (m)	Nominal Full Flow Capacity, Qf (L/s)	Nominal Full Flow Velocity (m/s)	Percent of Full Flow (Qd/Qf)	Actual Flow Velocity V (m/s)	Remarks
PRE-DEVELOPMENT - Population Scenario																				
				Floor Area represented in m^2																
2225 Erin Mills Parkway - Zone A	Existing	Population	0.5351	50	27															
	Existing	Total Site	0.5351			0.5351	27	4.36	0.41	0.11		0.5								
2225 Erin Mills Parkway - Zone G	Existing	Population	0		0															
	Existing	Total Site	0.4701			0.4701	0	4.50	0.00	0.09		0.1								
POST-DEVELOPMENT - Population Scenario																				
	Proposed Building "A"																			
2225 Erin Mills Parkway - Zone A	Building A	Population	249	2.7	672															
	Building A	Total Site	0.5351			0.5351	672	3.90	9.2	0.11		9.3								
								Total Increase in Flow from Building "A" -				8.8								
	Building A	MH.1A				0.5351	672	3.90	9.2	0.11		9.3	150	2.0%	2.3	22.5	1.23	41.4%	1.2	Self Cleansing Ok
	MH.1A	MH.2A				0.535	672	3.90	9.2	0.11		9.3	150	2.0%	4.0	22.5	1.23	41.4%	1.2	Self Cleansing Ok
	Proposed Building "G"																			
2225 Erin Mills Parkway - Zone G	Building G	Population	371	2.7	1002															
	Building G	Total Site	0.4701			0.4701	1002	3.80	13.3	0.09		13.4	150	2.0%	17.8	22.5	1.23	59.8%	1.3	Self Cleansing Ok
								Total Increase in Flow from Building "G" -				13.3								

APPENDIX E

Pre- & Post-Development Drainage Plans -
Storm Sewer Design Calculations -
Stormwater Storage Calculations -
Stormwater Quality Calculations -
Water Balance Calculations -



0.5432 — DRAINAGE AREA (ha)
0.71 — RUNOFF COEFFICIENT

STORM DRAINAGE AREA

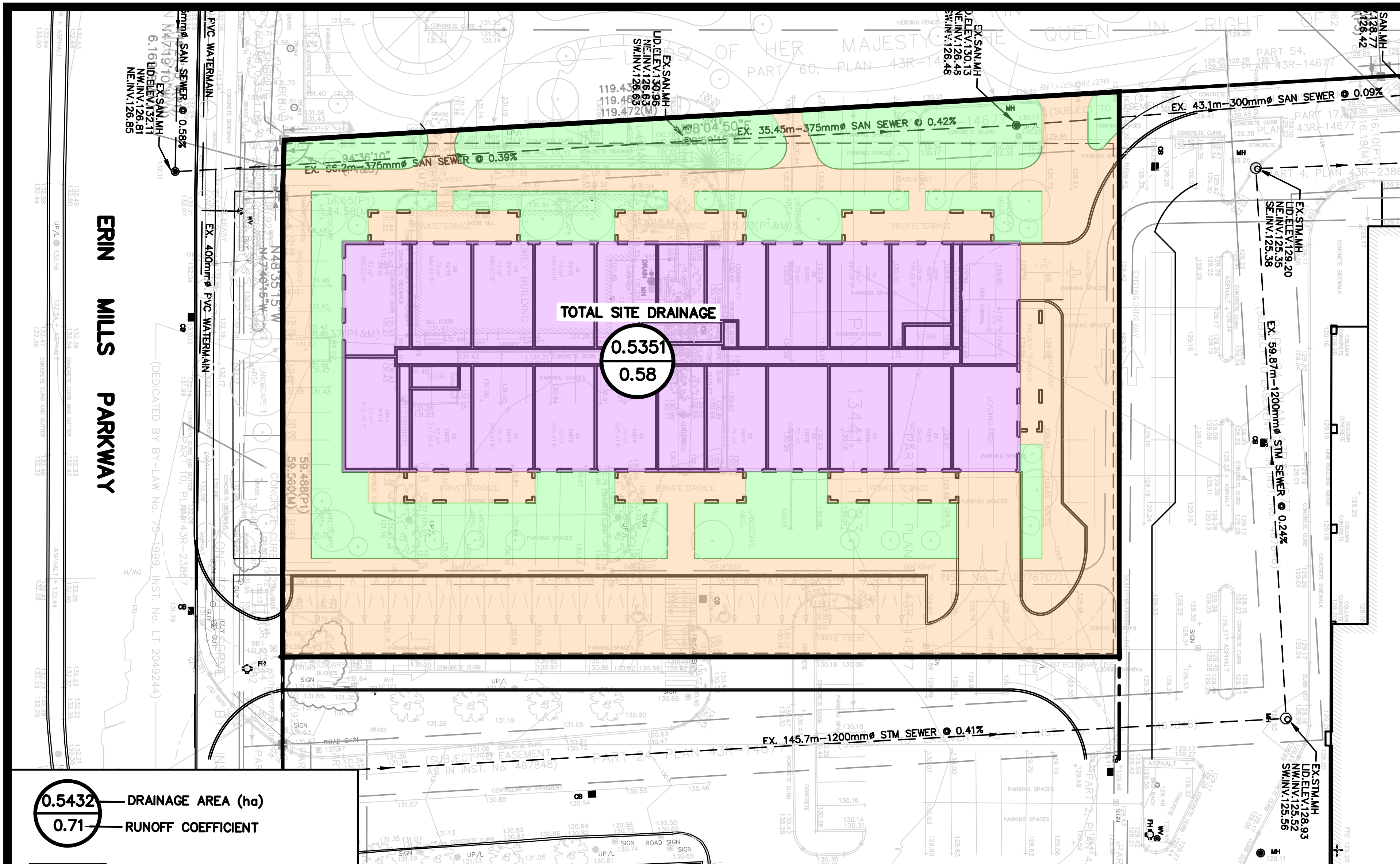
DWN. BY:	A.S.
DESIGNED BY:	A.S.
CHECKED BY:	P.F.A.
SCALE:	1:600
DATE:	MAY 2023
SHEET NO:	1 OF 4

fabian papa & partners
A Division of FP&P HydraTek Inc.
3901 Highway 7, Suite 500
Vaughan, Ontario, L4L 8L5
t: 905-264-2420
www.fabianpapa.com

PROJECT NO:	DWG NO:	REV NO:
23007	SWM–1A	0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
ZONE 'A' PRE–DEVELOPMENT STORMWATER DRAINAGE PLAN



0.5432 — DRAINAGE AREA (ha)
0.71 — RUNOFF COEFFICIENT

STORM DRAINAGE AREA
UNCONTROLLED DRAINAGE AREA
ROOFTOP DRAINAGE AREA
PERMEABLE PAVER DRAINAGE AREA
LANDSCAPED DRAINAGE AREA

DWN. BY:	A.S.
DESIGNED BY:	A.S.
CHECKED BY:	P.F.A.
SCALE:	1:600
DATE:	MAY 2023
SHEET NO:	2 OF 4

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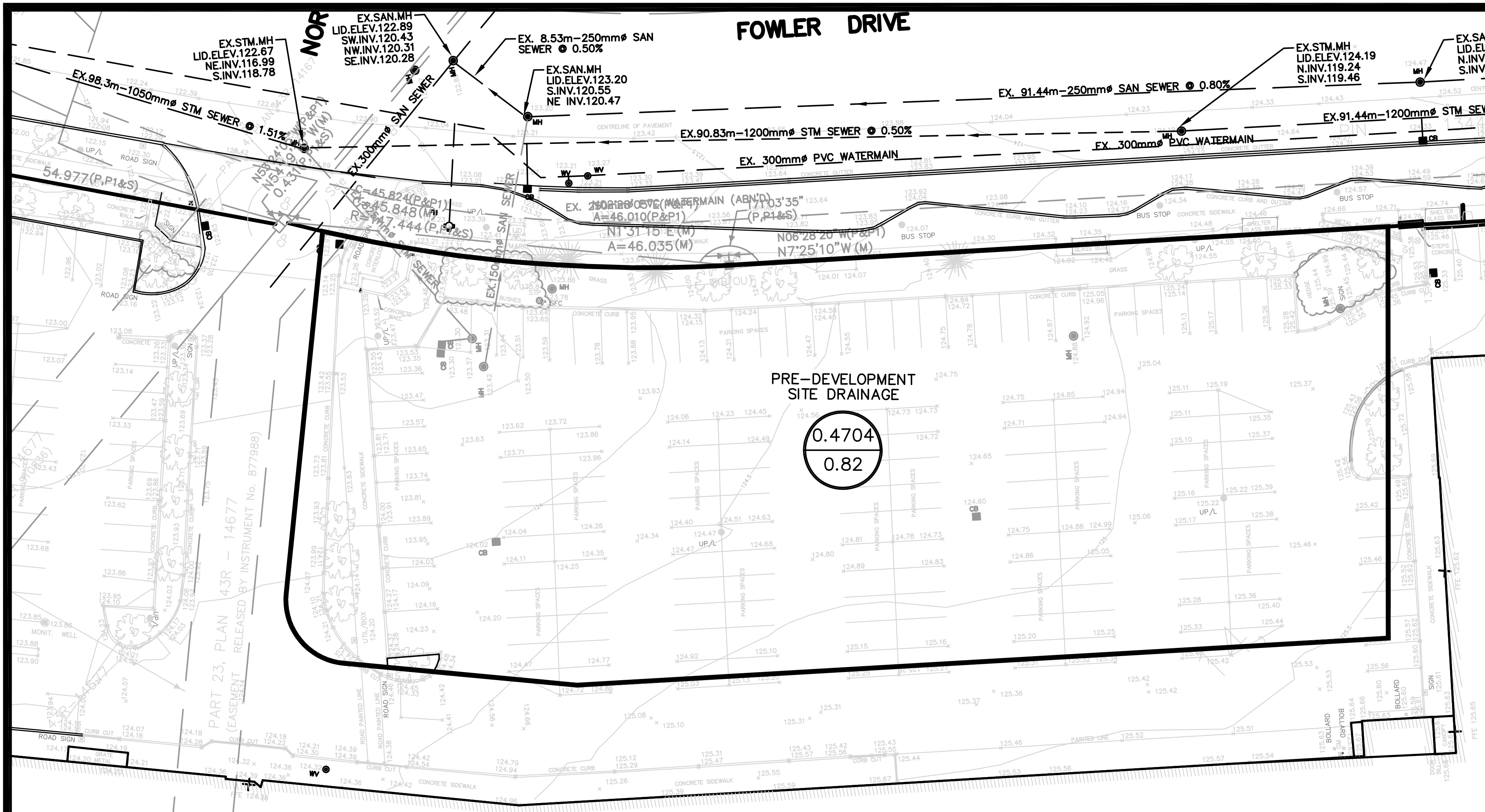
PROJECT NO: 23007
DWG NO: SWM-2A
REV NO: 0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
ZONE 'A' POST-DEVELOPMENT STORMWATER DRAINAGE PLAN

H:\FP&P HydraTek\Projects\2023\23007 - 2225 Erin Mills Parkway, Mississauga\Drawings\ZBA - Revision 0\23007 - Civil Engineering Plans (Revision 0) - GRADING.dwg (May 02, 2023 - 1:26pm)



0.5432	DRAINAGE AREA (ha)
0.71	RUNOFF COEFFICIENT
(box)	STORM DRAINAGE AREA

DWN. BY:	A.S.
DESIGNED BY:	A.S.
CHECKED BY:	P.F.A.
SCALE:	1:600
DATE:	MAY 2023
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PROJECT NO:	DWG NO:	REV NO:
23007	SWM–1G	0

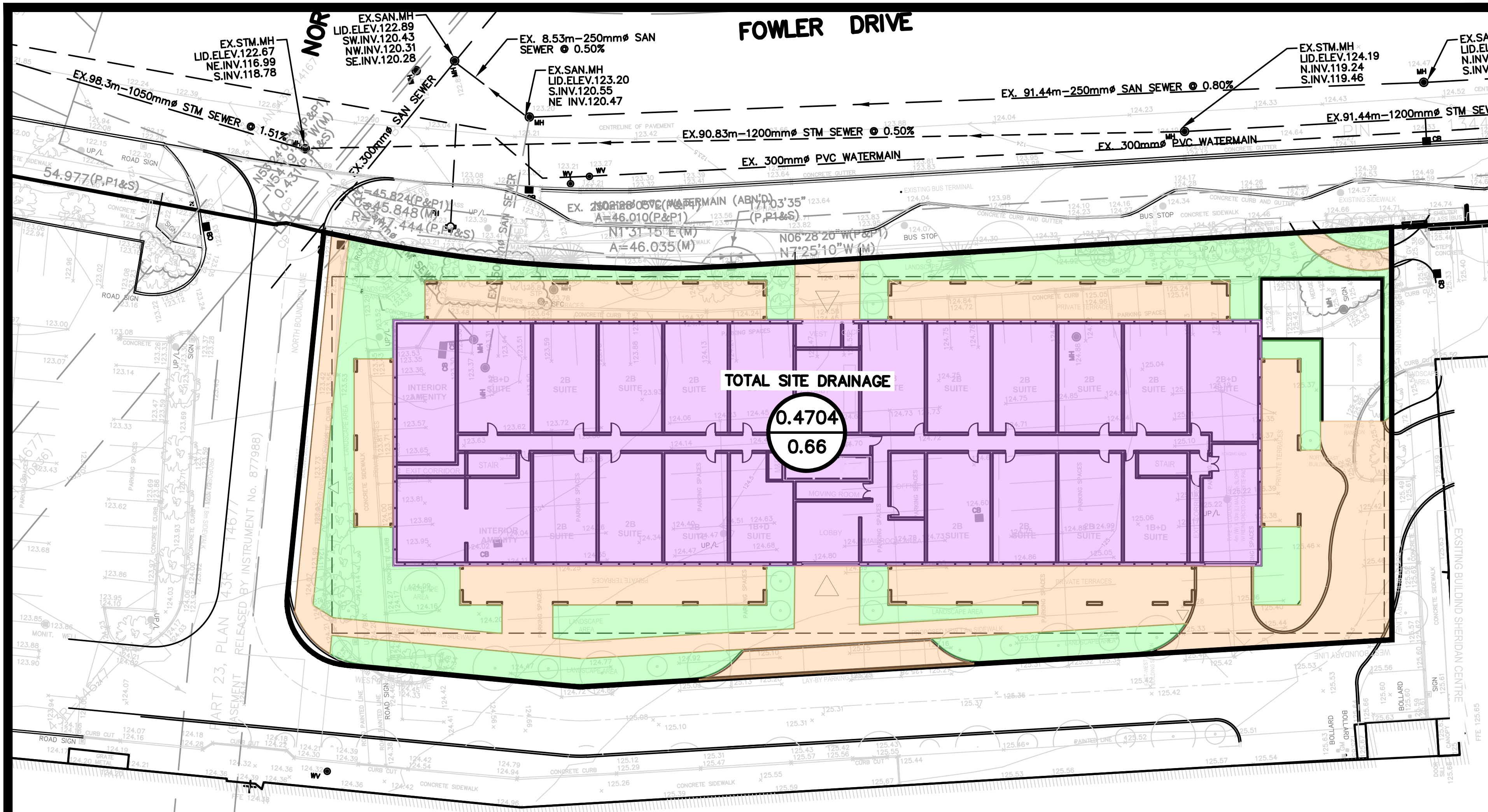
PROJECT NAME:

2225 ERIN MILLS PARKWAY

RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

ZONE 'G' PRE–DEVELOPMENT STORMWATER DRAINAGE PLAN



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0.71	RUNOFF COEFFICIENT
	STORM DRAINAGE AREA
	UNCONTROLLED DRAINAGE AREA
	ROOFTOP DRAINAGE AREA
	PERMEABLE PAVER DRAINAGE AREA
	LANDSCAPED DRAINAGE AREA

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PROJECT NO:	DWG NO:	REV NO:
23007	SWM-2G	0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
ZONE 'G' POST-DEVELOPMENT STORMWATER DRAINAGE PLAN

2225 Erin Mills Parkway - Residential Development

Weighted Run-Off Coefficient Calculations

Based on City of Mississauga Storm Design Criteria

Designed By: **Alessandro Stefenatti**, B.Eng.
Checked By: **Paolo Albanese**, P.Eng.
File No. 23007
Date: 08 May 2023

Existing Site Conditions - Building "A"				
Surface	Area (m^2)			
Roof Bare	309.0	5.8%	0.90	0.05
Green Roof	0.0	0.0%	0.50	0.00
Landscape	162.0	3.0%	0.25	0.01
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	4880.0	91.2%	0.90	0.82
	5351.0	100%		0.88

Existing Site Conditions - Building "G"				
Surface	Area (m^2)			
Roof Bare	0.0	0.0%	0.90	0.00
Green Roof	0.0	0.0%	0.50	0.00
Landscape	569.9	12.1%	0.25	0.03
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	4134.1	87.9%	0.90	0.79
	4704.0	100%		0.82

Proposed Development Conditions - Building "A"				
Surface	Area (m^2)			
Roof Bare	1844.2	34.5%	0.90	0.31
Green Roof	0.0	0.0%	0.50	0.00
Landscape	1296.0	24.2%	0.25	0.06
Permeable	2210.8	41.3%	0.50	0.21
Hard Surface	0.0	0.0%	0.90	0.00
	5351.0	100%		0.58

% IMP 50.4

Proposed Development Conditions - Building "G"				
Surface	Area (m^2)			
Roof Bare	2227.7	47.4%	0.90	0.43
Green Roof	0.0	0.0%	0.50	0.00
Landscape	911.2	19.4%	0.25	0.05
Permeable	1368.8	29.1%	0.50	0.15
Hard Surface	196.4	4.2%	0.90	0.04
	4704.0	100%		0.66

% IMP 62.7

2225 Erin Mills Parkway - Residential Development

City of Mississauga - Engineering & Construction Services

2 / 10 YEAR STORM SEWER DESIGN SHEET

2-Year IDF Curve - $I_{2-yr} = \dfrac{610}{(T_c + 4.6)^{0.78}}$ 10-Year IDF Curve - $I_{10-yr} = \dfrac{1010}{(T_c + 4.6)^{0.78}}$

Designed By: **Alessandro Stefenatti**, B.Eng.
Checked By: **Paolo Albanese**, P.Eng.
File No.: 23007
Date: 08 May 2023

Street	From MH	To MH	A (ha)	R	A x R	Accum. A x R	T_c (min)	I (mm/hr)	Q_{act} (L/s)	Size of Pipe (mm)	Slope (%)	Nominal Capacity Q_{cap} (L/s)	Full Flow Velocity (m/s)	Actual Flow Velocity (m/s)	Length (m)	Time in Sect. (min)	Total Time (min)	Q_{act}/Q_{cap}	Remarks
PRE-DEVELOPMENT CONDITIONS																			
2-YEAR PRE-DEVELOPMENT - Site Conditions																			
2225 Erin Mills Parkway	Site "A"	Street	0.5351	0.88	0.471	0.471	15.0	59.9	78.4	- Actual Flow									
2225 Erin Mills Parkway	Site "A"	Street	0.5351	0.50	0.268	0.268	15.0	59.9	**44.5**	- Site "A" Post-Development Storm Discharge Limited Flow									
2225 Erin Mills Parkway	Site "G"	Street	0.4704	0.82	0.386	0.386	15.0	59.9	64.3	- Actual Flow									
2225 Erin Mills Parkway	Site "G"	Street	0.4704	0.50	0.235	0.235	15.0	59.9	**39.1**	- Site "G" Post-Development Storm Discharge Limited Flow									
POST-DEVELOPMENT CONDITIONS																			
10-YEAR POST-DEVELOPMENT - Site Conditions																			
2225 Erin Mills Parkway	Site "A"		0.5351	0.58	0.309	0.309	15.0	99.2	85.1	Unattenuated Site Discharge									
Site "A" - Controlled Storm Discharge	SWM Tank	CNTRL.MH.1	0.5351	0.58					31.8	- 71.5% of Allowable Site Discharge									
	CNTRL.MH.1	STM Sewer	0.5351	0.58					**31.8**	200	2.0	48.4	1.48	1.57	13.5	0.2	0.2	65.7%	
2225 Erin Mills Parkway	Site "G"		0.4704	0.66	0.309	0.309	15.0	99.2	85.2	Unattenuated Site Discharge									
Site "G" - Controlled Storm Discharge	SWM Tank	CNTRL.MH.1	0.4704	0.66					30.8	- 78.7% of Allowable Site Discharge									
	CNTRL.MH.1	STM Sewer	0.4704	0.66					**30.8**	200	2.0	48.4	1.48	1.56	13.5	0.2	0.2	63.6%	

2225 Erin Mills Parkway - Residential Development

Stormwater Storage Calculations using Rational Method

10-year Storm - City of Mississauga IDF Data

Building "A" - SWM Tank Design $I_{10-yr} = \frac{1010}{(T_c + 4.6)^{0.78}} = 99.2\ mm/hr$

Project No.	**23007**		Area (ha)	0.5351
Analysis By:	**Alessandro Stefenatti**		Total Runoff Coefficient	0.58
Last Revised:	**08 May 2023**		Maximum Site Discharge (L/s)	31.8

Time (min)	Intensity (mm/hr)	Q-10 (L/s)	Q-stored (L/s)	Storage Volume (cu.m)
0	0.0	0.0	0.0	0.0
15	99.2	85.1	53.3	48.0
25	71.9	61.7	29.9	44.8
35	57.3	49.2	17.4	36.5
45	48.1	41.2	9.4	25.5
55	41.7	35.7	3.9	13.0
65	36.9	31.7	0.0	0.0
75	33.2	28.5	0.0	0.0
85	30.3	26.0	0.0	0.0
95	27.9	23.9	0.0	0.0
105	25.9	22.2	0.0	0.0
115	24.2	20.8	0.0	0.0
125	22.7	19.5	0.0	0.0
135	21.4	18.4	0.0	0.0
145	20.3	17.4	0.0	0.0
155	19.3	16.6	0.0	0.0
165	18.4	15.8	0.0	0.0
175	17.6	15.1	0.0	0.0
185	16.9	14.5	0.0	0.0
195	16.2	13.9	0.0	0.0
205	15.6	13.4	0.0	0.0
215	15.1	12.9	0.0	0.0
225	14.5	12.5	0.0	0.0
235	14.1	12.1	0.0	0.0
245	13.6	11.7	0.0	0.0
255	13.2	11.3	0.0	0.0
265	12.8	11.0	0.0	0.0
275	12.5	10.7	0.0	0.0
285	12.1	10.4	0.0	0.0
295	11.8	10.1	0.0	0.0
305	11.5	9.9	0.0	0.0
315	11.2	9.6	0.0	0.0
325	11.0	9.4	0.0	0.0
335	10.7	9.2	0.0	0.0
345	10.5	9.0	0.0	0.0
355	10.3	8.8	0.0	0.0
365	10.0	8.6	0.0	0.0

Storage Volume Required (cu.m)	**48.0**
Storage Volume Provided (cu.m)	**110.3**
Depth at Outlet (m)	**1.11**
Maximum Discharge Flow (L/s)	**31.8**
Outlet Type	**120 mm Orifice**

2225 Erin Mills Parkway - Residential Development

Stormwater Storage Calculations using Rational Method

10-year Storm - City of Mississauga IDF Data

Building "G" - SWM Tank Design $I_{10-yr} = \frac{1010}{(T_c + 4.6)^{0.78}} = 99.2\ mm/hr$

Project No.	**23007**		Area (ha)	0.4704
Analysis By:	**Alessandro Stefenatti**		Total Runoff Coefficient	0.66
Last Revised:	**08 May 2023**		Maximum Site Discharge (L/s)	30.8

Time (min)	Intensity (mm/hr)	Q-10 (L/s)	Q-stored (L/s)	Storage Volume (cu.m)
0	0.0	0.0	0.0	0.0
15	99.2	85.2	54.4	49.0
25	71.9	61.8	31.0	46.5
35	57.3	49.2	18.5	38.7
45	48.1	41.3	10.5	28.4
55	41.7	35.8	5.0	16.5
65	36.9	31.7	0.9	3.6
75	33.2	28.6	0.0	0.0
85	30.3	26.0	0.0	0.0
95	27.9	24.0	0.0	0.0
105	25.9	22.3	0.0	0.0
115	24.2	20.8	0.0	0.0
125	22.7	19.5	0.0	0.0
135	21.4	18.4	0.0	0.0
145	20.3	17.5	0.0	0.0
155	19.3	16.6	0.0	0.0
165	18.4	15.8	0.0	0.0
175	17.6	15.1	0.0	0.0
185	16.9	14.5	0.0	0.0
195	16.2	13.9	0.0	0.0
205	15.6	13.4	0.0	0.0
215	15.1	12.9	0.0	0.0
225	14.5	12.5	0.0	0.0
235	14.1	12.1	0.0	0.0
245	13.6	11.7	0.0	0.0
255	13.2	11.4	0.0	0.0
265	12.8	11.0	0.0	0.0
275	12.5	10.7	0.0	0.0
285	12.1	10.4	0.0	0.0
295	11.8	10.2	0.0	0.0
305	11.5	9.9	0.0	0.0
315	11.2	9.7	0.0	0.0
325	11.0	9.4	0.0	0.0
335	10.7	9.2	0.0	0.0
345	10.5	9.0	0.0	0.0
355	10.3	8.8	0.0	0.0
365	10.0	8.6	0.0	0.0

Storage Volume Required (cu.m)	**49.0**
Storage Volume Provided (cu.m)	**92.8**
Depth at Outlet (m)	**0.90**
Maximum Discharge Flow (L/s)	**30.8**
Outlet Type	**125 mm Orifice**

2225 Erin Mills Parkway - Residential Development

Water Quality, Initial Abstraction and Water Balance Calculations
Based on Stomwater Criteria - City of Mississauga

Designed By: **Alessandro Stefenatti**, B.Eng.
Checked By: **Paolo Albanese**, P.Eng.
File No. 23007
Date: 08 May 2023

Water Quality Management

Building "A"

Inferred Water Quality			% TSS Removal	Overall
Roof Bare	1844.2	34.5%	95	32.7
Green Roof	0.0	0.0%	100	0.0
Landscape	1296.0	24.2%	100	24.2
Permeable	2210.8	41.3%	80	33.1
Hard Surface	0.0	0.0%	0	0.0
	5351.0	100%		90.0

Overall TSS Removal Achieved at 80%

Building "G"

Inferred Water Quality			% TSS Removal	Overall
Roof Bare	2228	47.4%	95	45.0
Green Roof	0	0.0%	100	0.0
Landscape	911	19.4%	100	19.4
Permeable	1369	29.1%	80	23.3
Hard Surface	196	4.2%	0	0.0
	4704	100%		87.6

Overall TSS Removal Achieved at 80%

Water Balance Management

Initial Abstraction (post Building A)

Roof Bare	1844	34.5%	1	0.34
Green Roof	0	0.0%	5	0.00
Landscape	1296	24.2%	5	1.21
Permeable	2211	41.3%	5	2.07
Hard Surface	0	0.0%	1	0.00
	5351	100.0%		3.6

Required Initial abstraction (mm)	1.4
Required infiltration volume (m^3):	7.4

Total Volume to be used for Irrigation &/or Infiltration = 7.4 cu.m

Proposed Sump in Building 'A' SWM Tank

Footprint Area	m^2		43.3
Depth	m		0.30
Provided Volume	m^3		13.0

Initial Abstraction (post Building G)

Roof Bare	2228	47.4%	1	0.47
Green Roof	0	0.0%	5	0.00
Landscape	911	19.4%	5	0.97
Permeable	1369	29.1%	5	1.45
Hard Surface	196	4.2%	1	0.04
	4704	100.0%		2.9

Required Initial abstraction (mm)	2.1
Required infiltration volume (m^3):	9.7

Total Volume to be used for Irrigation &/or Infiltration = 9.7 cu.m

Proposed Sump in Building 'G' SWM Tank

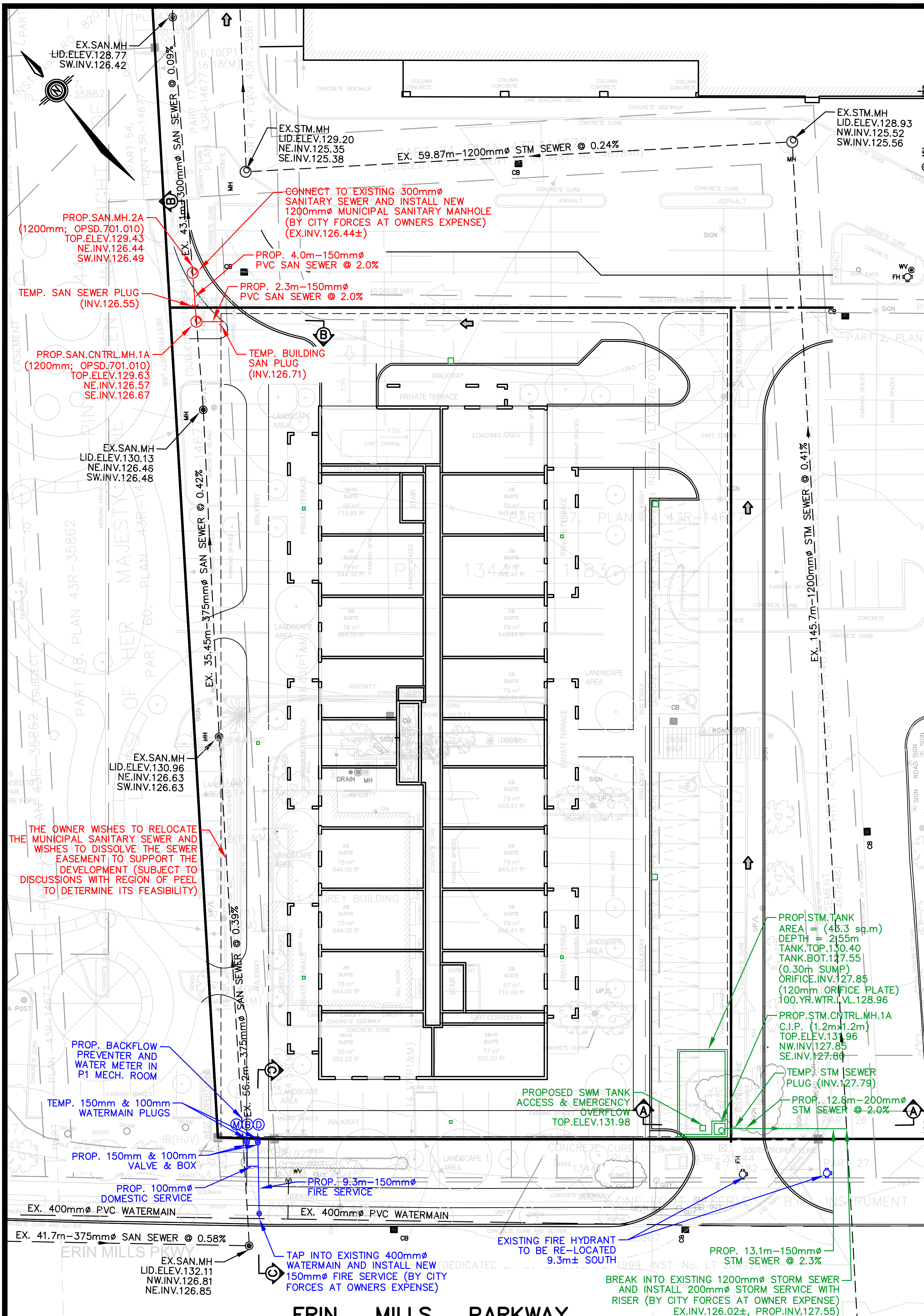
Footprint Area	m^2		54.6
Depth	m		0.30
Provided Volume	m^3		16.4

APPENDIX F

Civil Engineering Sketches

Site A (SSS-A, SCS-A, SGS-A) -

Site G (SSS-G, SCS-G, SGS-G) -



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23007	SSS-A	0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
BUILDING "A" SITE SERVICING SCHEMATIC

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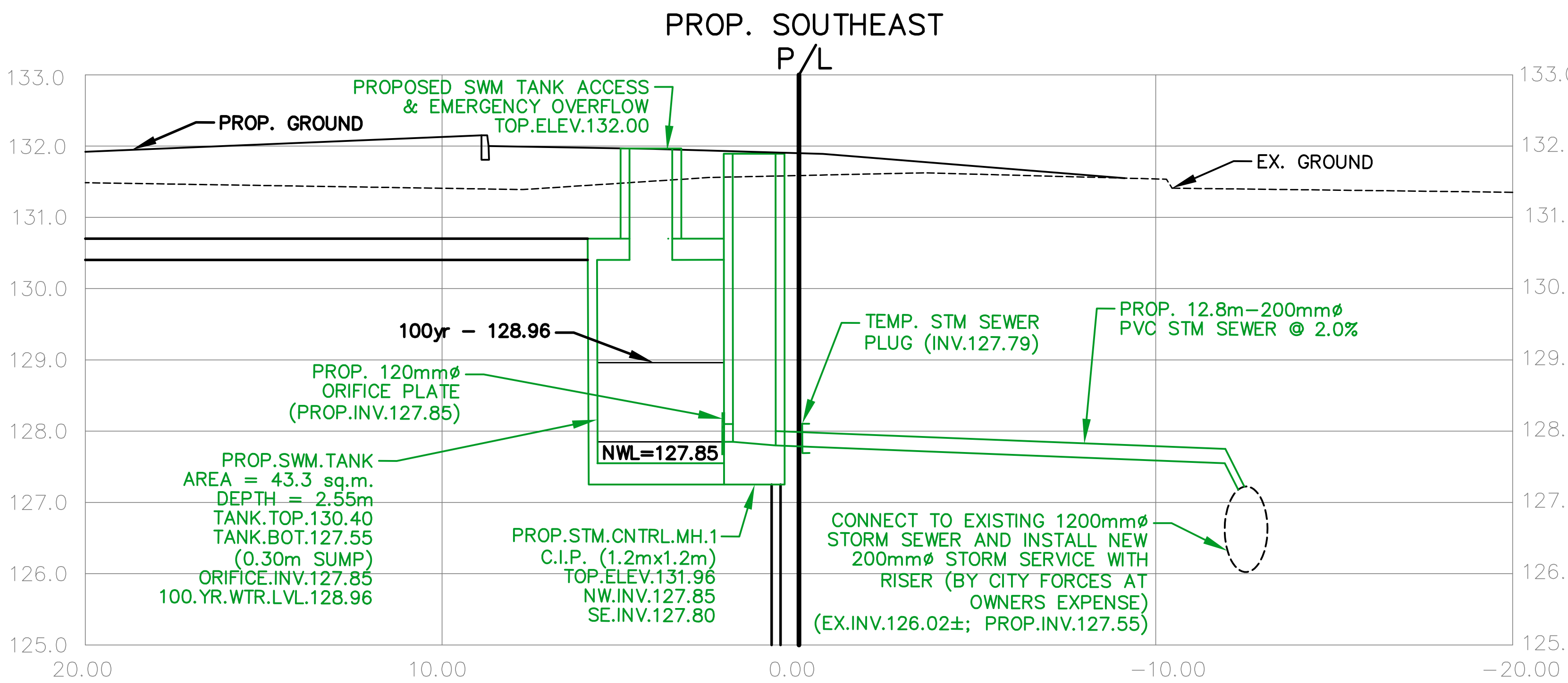

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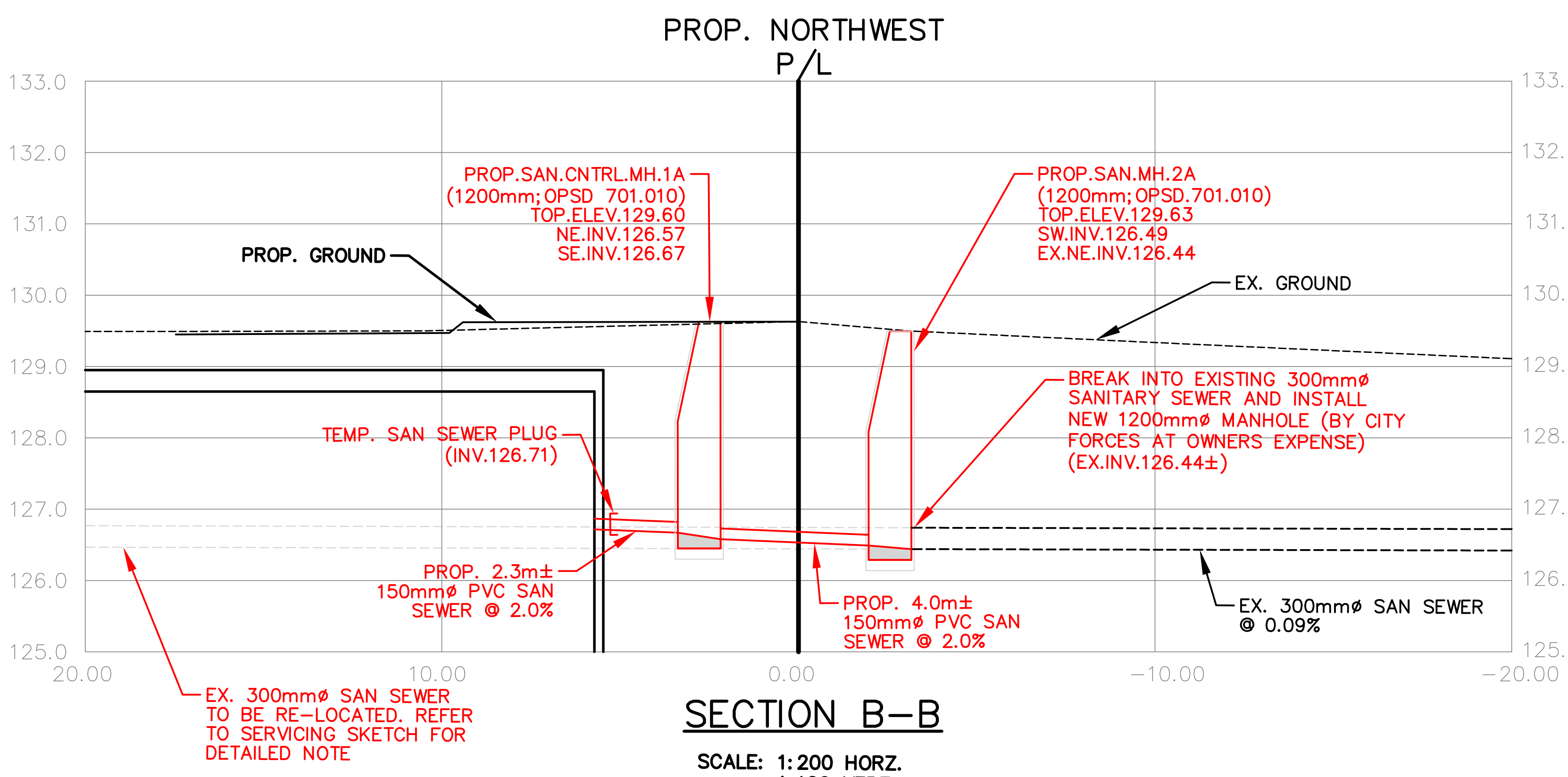

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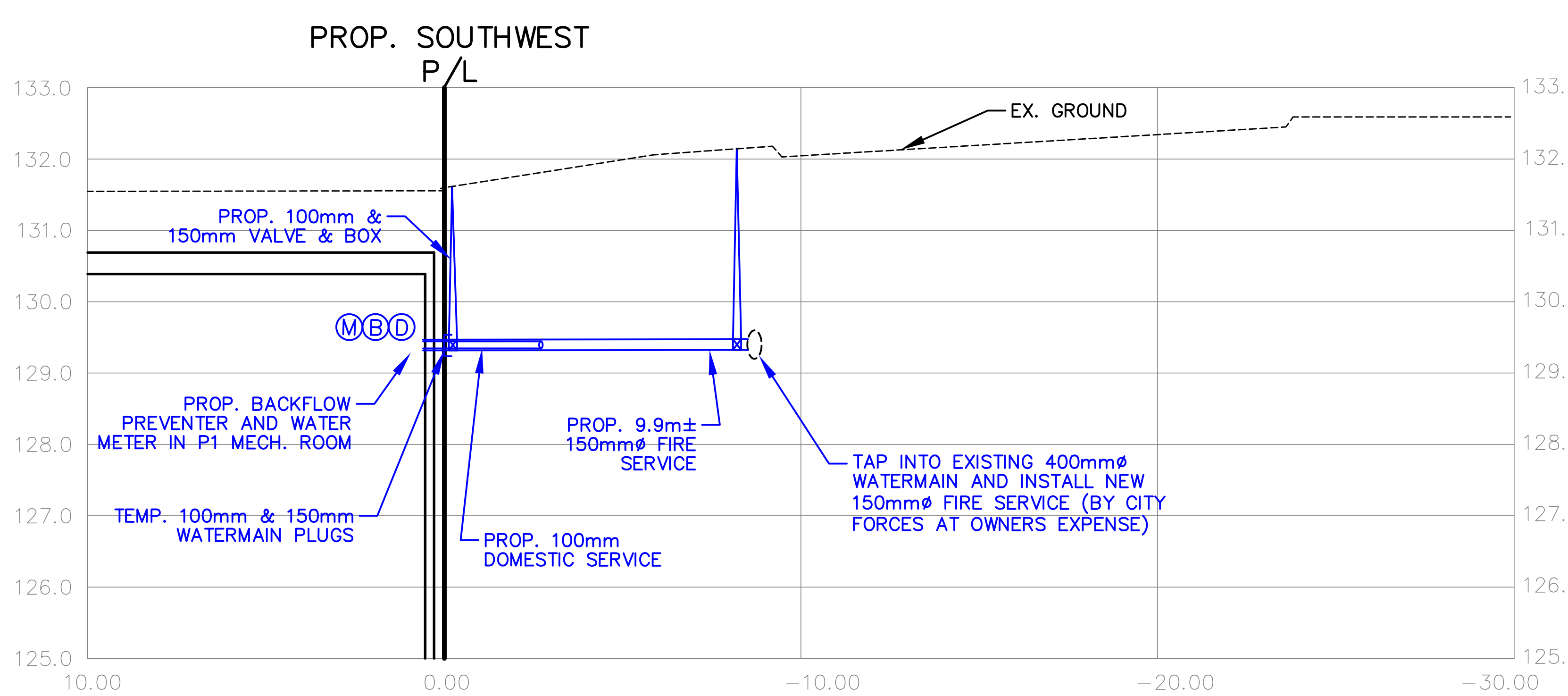

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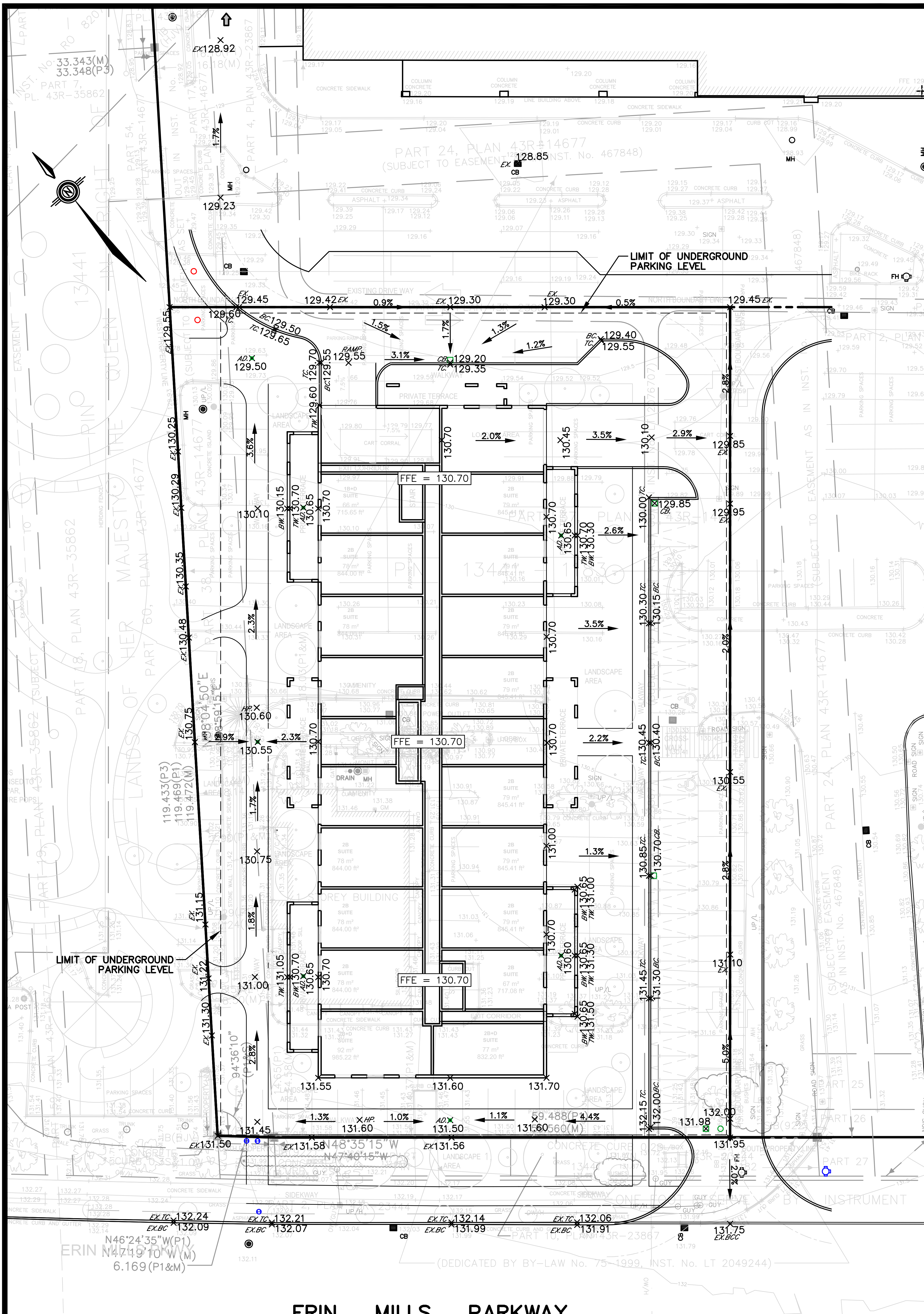
PROJECT NO:	DWG NO:	REV NO:
23007	SCS–A	0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
BUILDING "A" SERVICING CROSS–SECTIONS



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Vaughan, Ontario, L4L 8L5
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PROJECT NO:	DWG NO:	REV NO:
23007	SGS-A	0

PROJECT NAME:

2225 ERIN MILLS PARKWAY

RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

BUILDING "A" SITE GRADING SCHEMATIC

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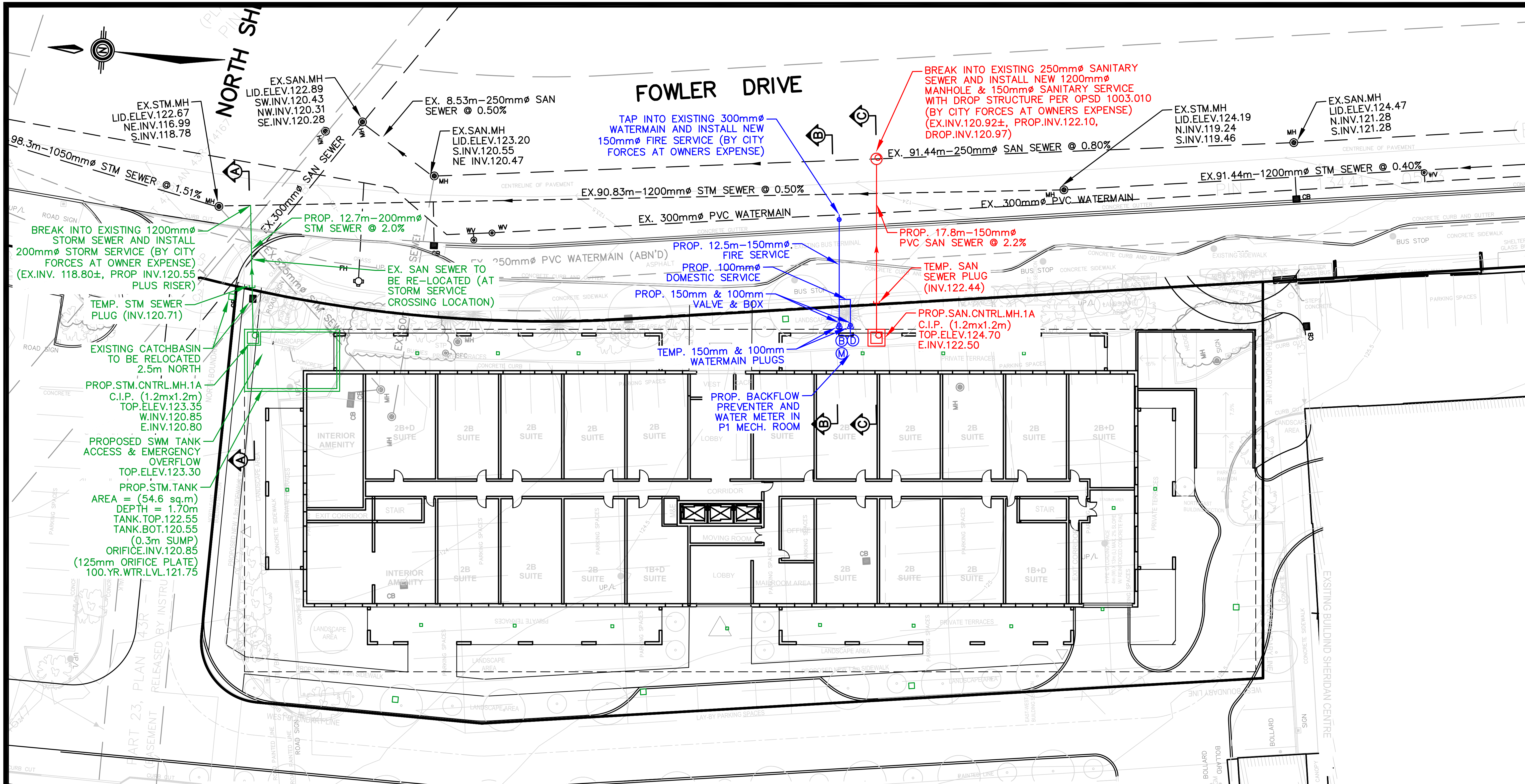

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PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
BUILDING "G" SITE SERVICING SCHEMATIC

PROJECT NO:	DWG NO:	REV NO:
23007	SSS-G	0

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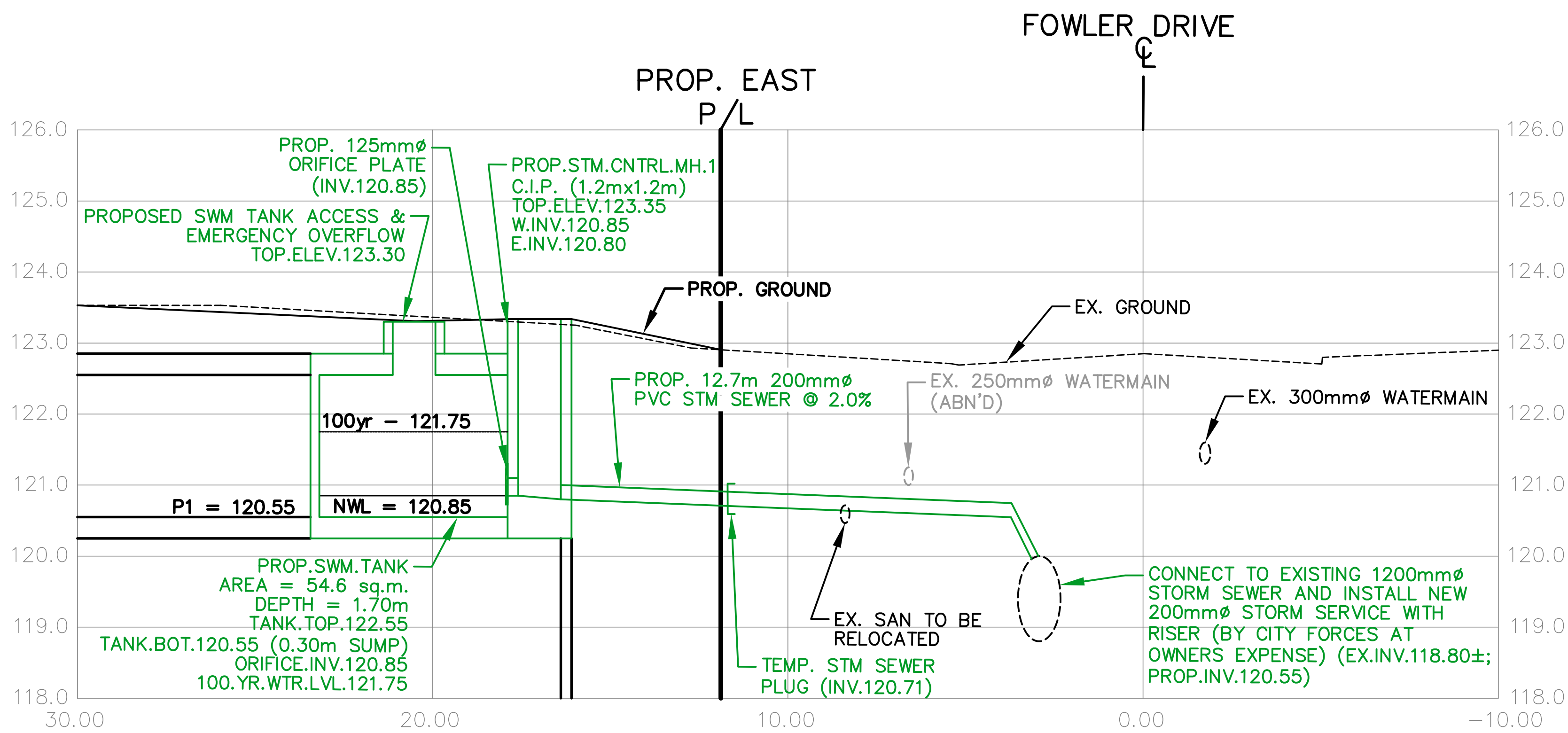

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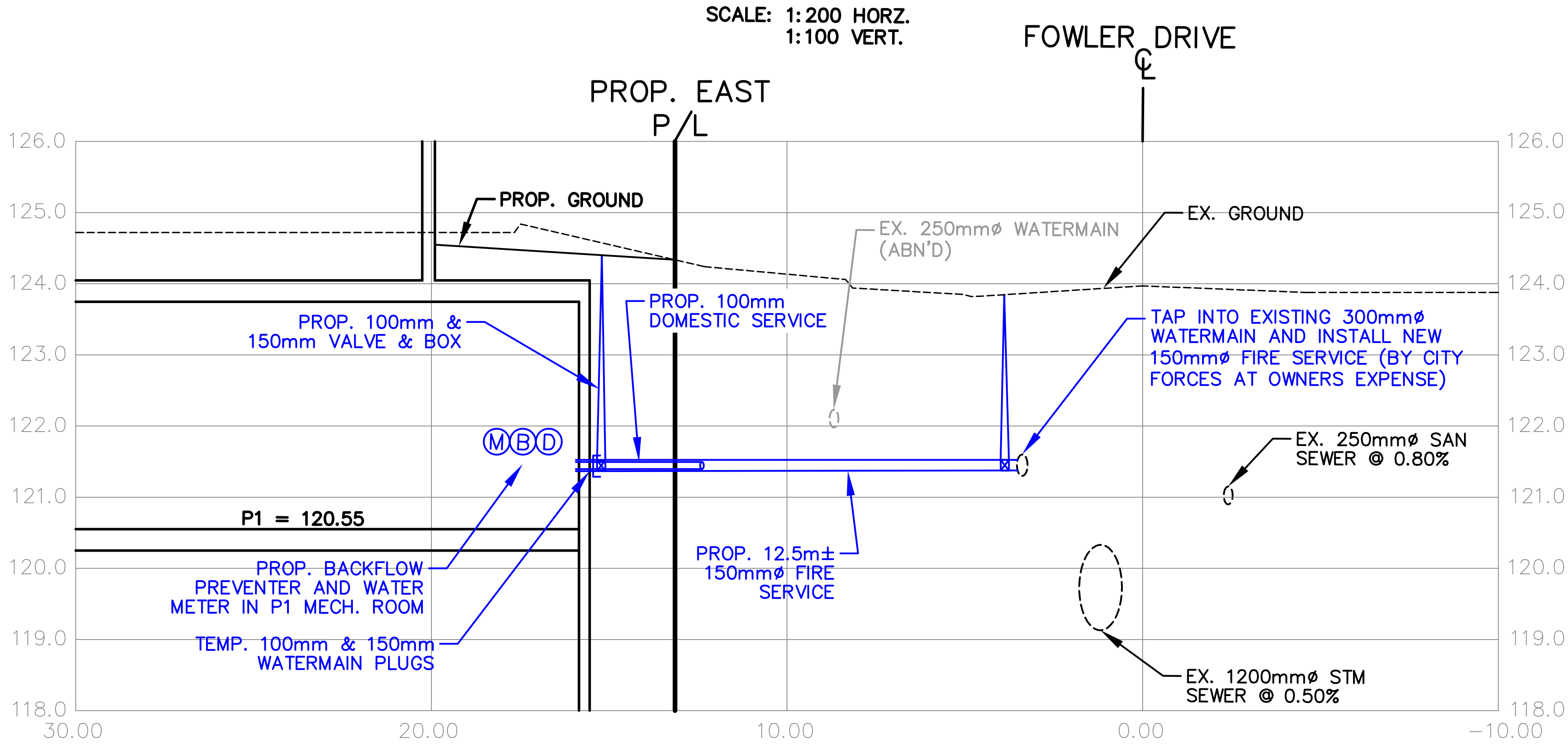

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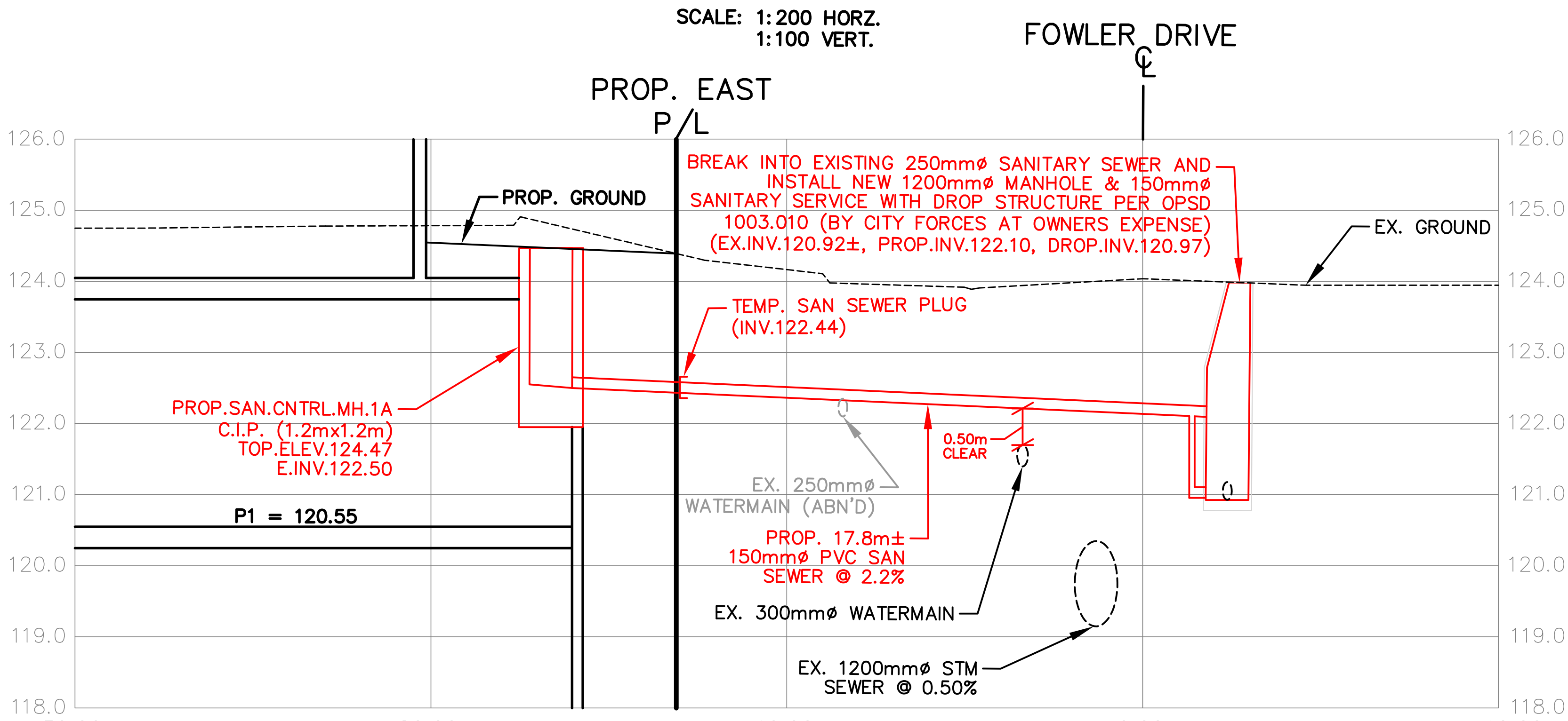

SECTION C–C

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CHECKED BY:	P.A.
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t: 905-264-2420
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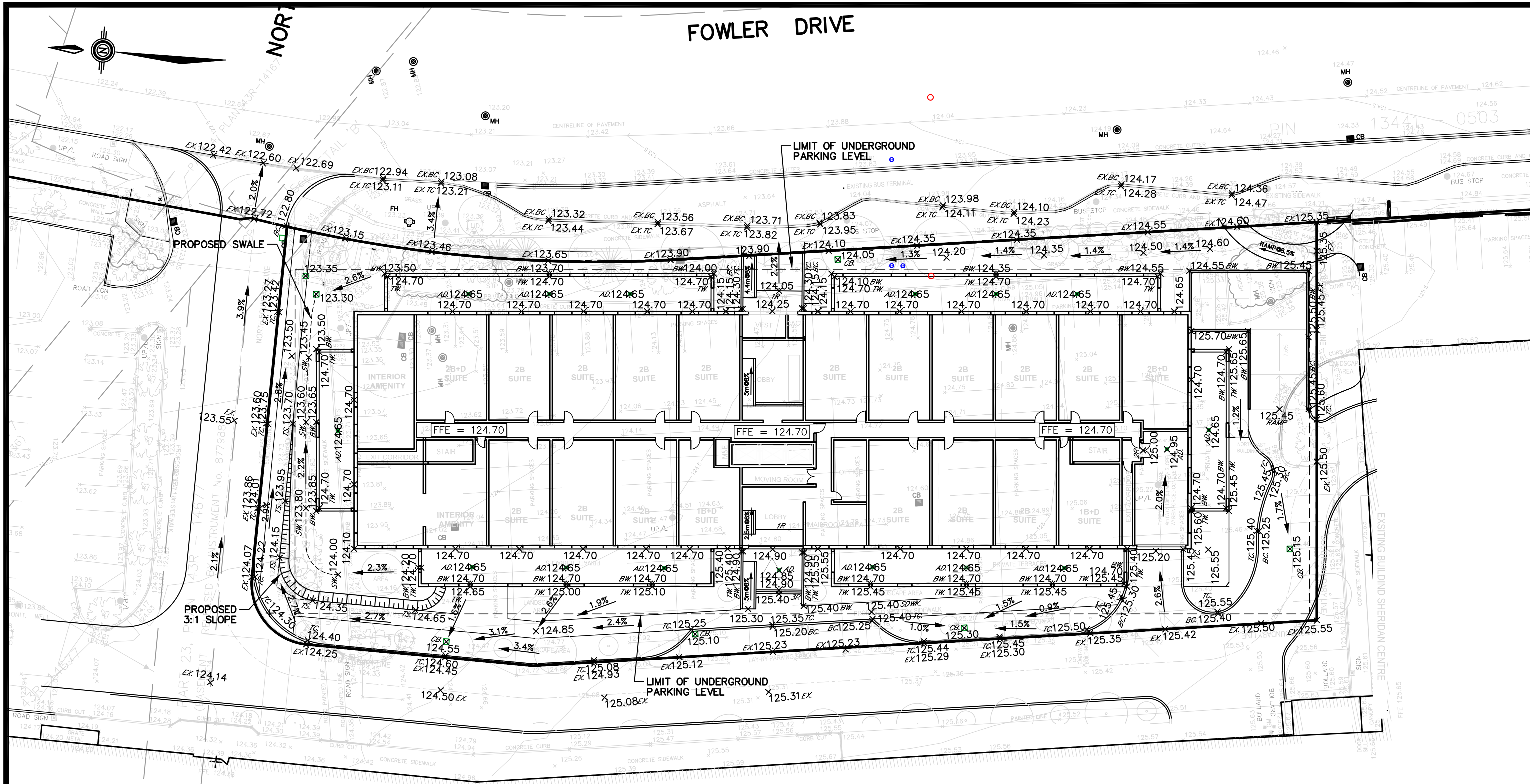
PROJECT NO:	DWG NO:	REV NO:
23007	SCS-G	0

PROJECT NAME:
2225 ERIN MILLS PARKWAY
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
BUILDING "G" SERVICING CROSS-SECTIONS



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PROJECT NAME:
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RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
BUILDING "G" SITE GRADING SCHEMATIC

PROJECT NO:	DWG NO:	REV NO:
23007	SGS-G	0

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