

Riverside South Community Centre and Library Phase 1

Site Servicing and Stormwater Management Report



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1 Introduction

Diamond and Schmitt Architects have commissioned Stantec Consulting Ltd. to prepare the following Site Servicing and Stormwater Management Report for development of the Riverside South Community Centre and Library. The subject property is located at the southwest parcel of phase 7A of the Riverside South Town Center community, south of Earl Armstrong Road within the City of Ottawa.

The block is currently zoned as Recreation Subzone 2 (REC2) and measures approximately 10.88 ha in area. The site itself is intended to be divided into two construction phases, with Phase 1 encompassing the eastern boundary of the site including the proposed library and community centre buildings and associated parking, whereas Phase 2 is to contain the remaining park programming, play fields, and a secondary parking area. The site is bordered by a future BRT corridor to the south, Jessie Chenevert Walk to the south, Portico Way to the west, and Town Square Street to the east.

The proposed Phase 1 development is comprised of a singular roughly 0.39ha building containing both the proposed community centre and library, including associated parking, fitness and play areas, a proposed splash pad and shade structure building. The objective of this report is to provide a servicing scenario for the site that is free of conflicts, provides on-site servicing in accordance with City of Ottawa design guidelines, and utilizes the existing and future local infrastructure in accordance with the various background studies including the *Design Brief Phase 7A* report (Arcadis, 2025), as outlined in Section 2.

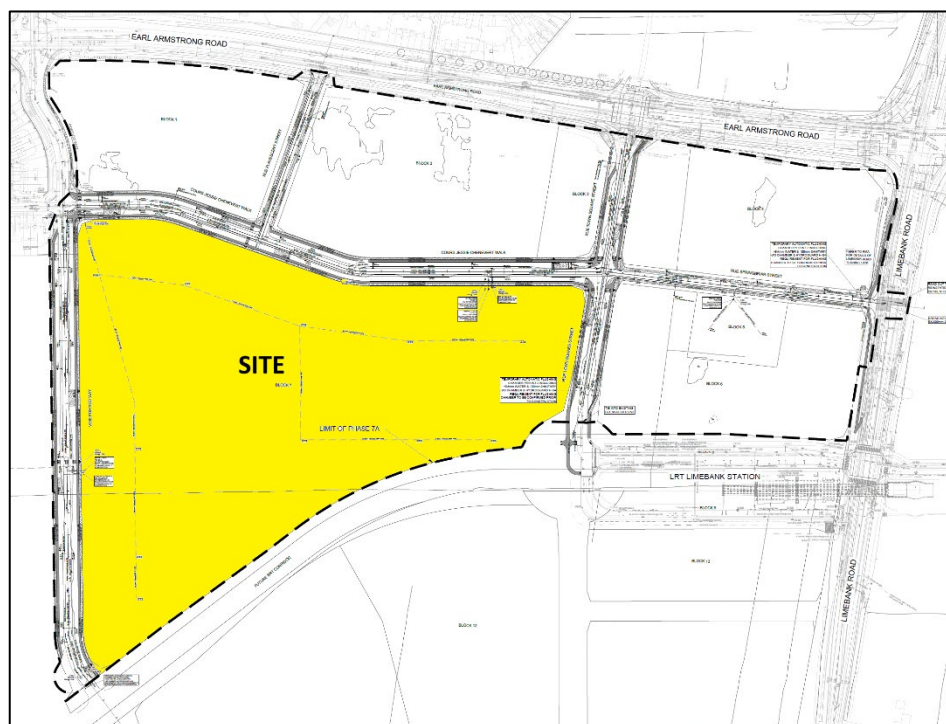


Figure 1.1: Location of Riverside South Community Centre and Library

2 Background

The following documents were referenced in the preparation of this report:

- *Design Brief Phase 7A – 980 Earl Armstrong Road and 4700 Limebank Road, Riverside South*, Arcadis, Revised March, 2025.
- *Geotechnical Investigation, Proposed Mixed-Use Development Town Center Phase 7A – Riverside South, Ottawa*, Paterson Group Inc., June 20, 2024.
- *Phase 1 Environmental Site Assessment Part of 980 Earl Armstrong Road and 4700 Limebank Road*, Paterson Group Inc., April 26, 2024.
- *Hydrogeological Review and Groundwater Impact Assessment Proposed Mixed-Use Development Town Center Phase 7A – Riverside South 980 Earl Armstrong Road*, Paterson Group Inc., October 3, 2024.
- *Stormwater Management Planning and Design Manual*, Ministry of the Environment (Ontario), March 2003
- *City of Ottawa Water Distribution Guidelines, 2nd Ed.*, City of Ottawa, December 2025
- *City of Ottawa Sewer Design Guidelines, 4th Ed.*, City of Ottawa, December 2025



3 Water Servicing

3.1 Background

The proposed development is currently within Pressure Zone 2W2C of the City of Ottawa's water distribution system, and forms part of the future SUC Zone reconstruction. As part of the detailed design of Phase 7A of the Riverside South subdivision, a water hydraulic analysis was completed to demonstrate that the water distribution network for the subdivision would adequately meet the domestic and fire supply requirements for development blocks. Results are documented in the Design Brief Phase 7A report (Arcadis, 2025). The proposed site (identified as Block 7 in background studies) will be serviced by the 200 mm diameter municipal watermain in Jessie Chenevert Walk that was constructed as part of the Phase 7A subdivision development.

3.2 Design Criteria

3.2.1 Water Demand and Allowable Pressure

The domestic water demand and allowable water pressure are assessed using the City of Ottawa Water Distribution Guidelines (2025) as amended.

Institutional Demand

Average Daily (AVDY)	28,000 L/ha/day
Maximum Daily (MXDY)	$1.5 \times \text{AVDY}$
Peak Hour (PKHR)	$1.8 \times \text{MXDY}$

Allowable Water Pressure

MXDY Flow	345 kPa (50 psi) to 552 kPa (80 psi)
PKHR Flow Minimum	276 kPa (40 psi)
MXDY + Fire Flow	140 kPa (20 psi)
Maximum Allowable for Occupied Area	552 kPa (80 psi)

3.2.2 Fire Flow and Hydrant Capacity

Detailed fire flow requirements are assessed using the Fire Underwriters Survey (FUS) methodology (2020). Site specific criteria considered are noted in **Section 3.3.2**.

The minimum number of fire hydrants required to meet the fire flow is assessed using Section 18.5 of the National Fire Protection Agency (NFPA) Fire Code document.

- The aggregate fire flow capacity of all fire hydrants within 305 m of the building cannot be less than the required fire flow (RFF)



- Table 18.5.4.3 provides the maximum fire flow capacity for which an individual fire hydrant can be credited based on the unobstructed distance from the hydrant to the building.

Table 3.1: Table 18.5.4.3 of the NFPA - Maximum Fire Hydrant Fire Flow Capacity

Distance to Building (m)	Maximum Capacity (L/min)
≤76	5,678
>76 and ≤152	3,785
>152 and ≤305	2,839

Furthermore, per Section 3.2.5.15 of the Ontario Building Code, the unobstructed distance between the nearest hydrant and the building's fire department connection (FDC) cannot exceed 45 m.

3.3 Water Demands

3.3.1 Domestic Water Demands

Based on the assumptions that the Community Centre and Library fall within the "Other" Commercial and Institutional Demand Rates from Table 4.2 of the WDG (2025) as amended and the splash pad water play features are timed to demand a maximum flow rate of US 50GPM (189.3 L/min). The estimated demands for the site are summarized in **Table 3.2** and detailed in **Appendix A.1**.

Table 3.2: Estimated Water Demands

	Floor Area (ha)	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Institutional	0.039	0.1	0.2	0.3
Splash Pad	-	3.2	3.2	3.2
Total	0.039	3.3	3.7	3.8

3.3.2 Fire Flow Demands

Noncombustible construction was considered in the assessment for fire flow requirements according to the Fire Underwriter's Survey (FUS) Guidelines. The FUS Guidelines indicate that low hazard occupancies include dwellings, apartments, dormitories, hotels, and schools. As such, a limited combustible building contents credit was applied for the building. Based on the FUS 2020 methodology and assuming the community centre and library will be fully sprinklered in conformance with NFPA 13, the worst-case required fire flow for the site is 6,000 L/min (100 L/s). As per the Arcadis Design Brief Phase 7A (2025) the Block was simulated with a fire flow of 13,000 L/min (216 L/s)

On-site fire protection will be provided by one new private hydrant and several existing public hydrants located near the building. The FUS calculation sheets and correspondence are attached in **Appendix A.2**.



3.3.3 Boundary Conditions

The Arcadis Design Brief Phase 7A (2025) models the worst-case fire flow of 217 L/s (13,000 L/min) and an assumed second fire flow of 167 L/s (10,000 L/min), using the boundary conditions provided by the City of Ottawa to establish the pressure and flow in the main on Jessie Chenevert Walk in the ultimate condition. The boundary conditions used for the Phase 7A analysis were identified as Connections 1-5, including Portico Way South, Portico Way North, Earl Armstrong Road East, and Limebank Road. Excerpts from the Design Brief are included in **Appendix A.3**.

The Arcadis Design Brief Phase 7A (2025) allows for the watermain demands at Node C22 (Town Square Place) 10.60 ha Gross Institutional Area, as summarized in **Table 3.3**, which allows for the estimated water demands identified in **Table 3.2**.

Table 3.3: Institutional Water Demands – Design Brief (2025)

	Gross Area (ha)	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Institutional	10.60	3.4	5.2	9.3

3.3.4 Hydraulic Analysis

The approved hydraulic analysis provided in the Arcadis Design Brief Phase 7A (2025) includes the results at Node 22 as summarized in **Table 3.4**. The results shown are for the SUC Zone Reconstruction. The elevation at ground level is modelled to be 92.50m.

Table 3.4: Summary of Hydraulic Analysis Node 22 – Design Brief (2025)

Condition	Design Flow (L/s)	HGL (m)	Residual Pressure (kPa)	Residual Pressure (psi)
PKHR Min. HGL	3.44	143.37	498.49	72.3
AVDY Max. HGL	5.15	146.79	531.96	77.2
Max. Day + FF (217 L/s)	344.99	106.78	139.98	20.3

The hydraulic model found that the flow and pressure available from the municipal watermain adjacent to the site is adequate to service the Institutional/Park block. Since the proposed water demands are less than the design flows modelled in the Design Brief (2025), the proposed Site Plan can be adequately serviced by the existing infrastructure.

3.4 Proposed Watermain Servicing and Layout

Onsite (private) watermain are proposed to connect to the 200mm diameter watermain on Jessie Chenevert Walk. The onsite network provides connections to the proposed library building, community centre building and splash pad vault, and are sized to 150mm diameter PVC CL.150. One new onsite fire



hydrant is proposed, as shown in Drawing SSP-1. The onsite hydrant is to be situated within 45m of the proposed fire department connection. The proposed onsite watermain is to provide required domestic and fire flows.

3.4.1 Fire Hydrant Coverage

There are three existing fire hydrants adjacent to the site within the municipal ROWs and one proposed onsite (private) hydrant. All hydrants are situated within 60m of the building. According to the NFPA 1 Table 18.5.4.3 in Appendix I of the City of Ottawa Technical Bulletin ISTB-2018-02, a hydrant situated less than 76 m away from a building can supply a maximum capacity of 5,678 L/min. Hence, any combination of the two hydrants can provide the site's required fire flow of 6,000L/min.



4 Wastewater Servicing

4.1 Background

The proposed development within Phase 7A of the Riverside South community will be serviced by the previously installed 150mm diameter service connection to the 250mm sanitary sewer in Jessie Chenevert Walk, which directs flows from the subdivision to the trunk sewer within Portico Way / Canyon Walk to the north.

4.2 Design Criteria

As outlined in the City of Ottawa Sewer Design Guidelines and the MECP Design Guidelines for Sewage Works, the following criteria are used to calculate the estimated wastewater flow rates and to determine the size and location of the sanitary service laterals:

- Minimum velocity = 0.6 m/s (0.8 m/s for upstream sections)
- Maximum velocity = 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes = 0.013
- Minimum size of sanitary sewer service = 135 mm
- Minimum grade of sanitary sewer service = 1.0 % (2.0 % preferred)
- Average institutional wastewater generation = 28,000 L/ha/day (per City Design Guidelines)
- Peak Factor = 1.5 (institutional)
- Harmon correction factor = 0.8
- Infiltration allowance = 0.33 L/s/ha (per City Design Guidelines)
- Minimum cover for sewer service connections = 2.0 m

4.3 Proposed Servicing

Phase 1 of the site will be serviced via 150mm diameter gravity service lines, which will direct wastewater peak flows (approximately 0.8 L/s with allowance for infiltration) to the existing 250 mm diameter PVC sanitary sewer in Jessie Chenevert Walk. The receiving sewers within Jessie Chenevert and downstream have been sized to accommodate wastewater from the proposed development. The sanitary sewer design sheet for the proposed sanitary service within the site plan development and the sanitary design sheet and sanitary drainage area plan for the Riverside South Phase 7A subdivision are included in **Appendix B.2**.

Phase 1 of the site plan is represented by Area ID MH5C on the subdivision sanitary design sheet. As part of the subdivision design, it was assumed that the eastern portion of the block (approximately 1.47ha) would be developed with institutional uses totalling approximately 0.34ha of floor area, and that western portions of the block (the remaining 9.40ha) would only consider an infiltration allowance with respect to peak flow contribution. As Phase 1 is now proposed as 1.74ha with a building measuring approximately 0.39ha, design flows for the development block are marginally higher than the peak flows assumed at the time of subdivision design. As the design sheets for the subdivision sanitary sewer identify available



capacity downstream of at least 19L/s, the marginal increase in sanitary flow from the site plan block can easily be accommodated in the downstream system.

Full port backwater valves are to be installed on all sanitary services within the site to prevent any surcharge from the downstream sewer mains from impacting the proposed site.



5 Stormwater Management

The following section describes the stormwater management (SWM) design for the proposed site plan in accordance with the background documents and governing criteria for Phase 7A of the Riverside South subdivision established in the Design Brief Phase 7A report (Arcadis, 2025).

5.1 Proposed Conditions

The development site is located within the southwest corner of Phase 7A of the Riverside South subdivision, and comprises a total area of approximately 10.88ha, and includes a 0.39ha institutional building and associated parking areas. Phase 1 of the development includes approximately 1.74ha of the overall site at the block's eastern boundary. Two connections to existing storm sewers have been provided for the site. The connection for Phase 1 discharges to the existing 1350mm diameter storm sewer within Jessie Chenevert Walk, whereas Phase 2 is ultimately to discharge to the existing storm sewers on Portico Way (see **Drawing SD-1**).

Stormwater collected from the development block and the overall Phase 7A subdivision is ultimately discharged to Mosquito Creek via Riverside South Pond 2. The pond has been sized to provide quantity and quality control (80 % TSS Removal) of runoff inclusive of the development site before discharge.

5.2 Criteria and Constraints

The overall approach for storm servicing and stormwater management for the proposed development is outlined in the Design Brief Phase 7A report by Arcadis (2025), excerpts of which can be found in **Appendix C.3**. The following summarizes the SWM criteria and constraints that will govern the detailed design of the proposed site as per the latest revision of the City of Ottawa Sewer Design Guidelines as well as the conclusions made in the Phase 7A Report.

- Design using the dual drainage principle. (City of Ottawa SDG)
- Minor system capture rate from Phase 1 up to the 100-year storm is to be restricted to **113 L/s**. (Design Brief Phase 7A – see discharge from area MH15C)
- Minor system capture rate from Phase 2 up to the 100-year storm is to be restricted to **579 L/s**. (Design Brief Phase 7A – see discharge from area MH34A)
- The hydraulic grade line in the storm sewer must remain at least 0.3m below the underside of adjacent building footings during the 100-year storm event or a minimum clearance of 0.3m from the pipe obvert. (City)
- Where there is footing drainage connected to the storm collection system, maximum 'climate change' HGL to be lower than proposed basement elevations. (City)
- Total maximum depth of flow under static and dynamic conditions shall be less than 0.35 m. (City)
- Design storm sewers along local roadways to convey the 2-year peak flow respectively under free-flow conditions using 2004 City of Ottawa I-D-F parameters and an inlet time of 10 minutes. (City)
- Assess impact of 2-year storm, and the worst case 100-year storm events, on the major and minor drainage system. (City)



- Building openings to be above the 100-year water level. (City)
- There must be at least 30 cm of vertical clearance between the spill elevation on the private street/parking and the lowest building opening that is in the proximity of the flow route or ponding area. (City)
- Provide adequate emergency overflow conveyance off-site. (City)

5.3 Design Methodology

The design methodology for the SWM component of the development is as follows:

- Create a Modified Rational Method model to quantify runoff from the development and identify surface and subsurface volume storage required to restrict site outflows to the allowable release rate.
- Size inlet control devices for the proposed catch basins to avoid surface ponding during the 2-year storm while meeting the required 0.3 m 100-year HGL to USF clearance and the minor system allowable release rate in the 100-year storm.
- Ensure that total dynamic and static surface ponding depths do not exceed 0.35 m during the 100-year storm scenario.

The site is designed using the “dual drainage” principle, whereby the minor (pipe) system is designed to convey the peak rate of runoff from the 2-year design storm and runoff from larger events is conveyed by both minor (pipe) and major (overland) channels, such as roadways and walkways, safely to the appropriate outlet without impacting proposed or existing downstream properties.

In keeping with the minor system target peak outflow, Inlet Control Devices (ICDs) or orifice plates have been specified for all catch basins to limit the inflow to the minor system. Restricted inlet rates to the sewer are necessary to meet the target peak outflows.

Drawing SD-1 outlines the proposed storm sewer alignment, ICD locations, drainage divides, and labels. The storm sewer design sheet is included in **Appendix C.1**.

5.4 Modeling Rationale

The Modified Rational Method was employed to assess the rate of runoff generated during post-development conditions. A time of concentration for the post-development areas (10 minutes) was assigned based on the relatively small site and its proximity to the existing drainage outlet for the site. Surface storage estimates were based on the final grading plan design (see **Drawing GP-1**). Peak flow rates to sewers have been calculated using the rational method as follows:

$$Q = 2.78 (C)(I)(A)$$

Where:

Q = peak flow rate, L/s

C = site runoff coefficient

I = rainfall intensity, mm/hr (per City of Ottawa IDF curves)

A = drainage area, ha



5.4.1 Storage Requirements

The site requires quantity control measures to meet the restrictive stormwater release criteria. The use of controlled surface and subsurface storage volumes within a proposed underground storage facility (Cultec unit) are proposed to reduce site peak outflow to the allowable target release rate. As per City of Ottawa criteria, no surface ponding is permitted within the site during the 2-year storm event. Refer to **Appendix C** for the Modified Rational Method calculations which demonstrate that no surface storage is required in the 2-year event.

It is proposed to detain stormwater on the surface in parking lot areas using inlet control devices (ICDs) in associated catch basins. Additional runoff from storms in excess of the 100-year storm event that exceed available on-site storage will be directed overland towards the Jessie Chenevert ROW at the northern boundary of Phase 1. Phase 2 is expected to maintain an overland flow route to the west towards Portico Way per results of the Phase 7A design brief.

The Modified Rational Method was employed to determine the peak volume stored in the catch basins and surface storage areas. The site was subdivided into subcatchments (subareas) as defined by the proposed grades and the location, nature, or presence/absence of inlet control devices (ICDs). Each subcatchment was assigned a runoff coefficient based on the proposed finished surface. Further details can be found in **Appendix C**, while **Drawing SD-1** illustrates the proposed subcatchments. The inlet control devices were sized based on the available target release rate from the site during the 2-year storm event. Storage volume and controlled release rates from the on-site catch basins during the 2 and 100-year events are summarized in the table below.

Table 5.1 2-Year and 100-Year Peak Surface Volume and Controlled Discharge Summary

Area ID	ICD (Circular Orifice)	2-Year Event			100-Year Event		
		Release Rate (L/s)	V _{required} (m ³)	V _{available} (m ³)	Release Rate (L/s)	V _{required} (m ³)	V _{available} (m ³)
L103A	127 mm	22.4	0.0	18.0	39.5	15.4	18.0
L104A	83 mm	15.9	0.5	25.1	17.3	18.7	25.1
L105A	95 mm	20.7	0.2	33.1	22.4	19.3	33.1
L107C	83 mm	7.2	0.0	60.0	17.5	2.1	60.0
L200A	102 mm	13.5	0.0	7.7	24.9	5.9	7.7
L201A	95 mm	13.5	0.0	10.4	22.0	7.6	10.4
L202A	95 mm	12.7	0.0	9.8	22.9	7.1	9.8

Runoff captured by on-site catch basins is routed to the subsurface storage system (Cultec SWM Chambers Model 902HD) for further volume retention prior to discharge to sewers within Jessie Chenevert Walk. The proposed storage chambers are detailed on **Drawing DS-1** and are anticipated to provide an overall storage volume of approximately 400m³ between the bottom of stone elevation of 89.57 and top of storage system elevation of 91.33. Outflows from the system are controlled via in-line ICD within manhole



STM101 at an invert of 89.51. The manhole is to incorporate a weir wall between the invert of 89.51 and the top of storage elevation of 91.33 to allow additional site outflow in the event the primary orifice becomes blocked. Inverts to building connections have been set above this elevation to ensure 100yr storage elevations do not impact building services, and meet sufficient freeboard to anticipated building USF elevations as shown on **Drawing GP-1**. The table below identifies the required storage and anticipated controlled discharge rates from the Cultec unit during the 2 and 100-year storm events.

Table 5.2 2-Year and 100-Year Peak Subsurface Volume and Controlled Discharge Summary

Area ID	ICD (Circular Orifice)	2-Year Event			100-Year Event		
		Release Rate (L/s)	V _{required} (m ³)	V _{available} (m ³)	Release Rate (L/s)	V _{required} (m ³)	V _{available} (m ³)
Tank	200 mm	60.0	122.7	400.0	111.3	384.7	400.0

5.4.2 Uncontrolled Areas

Due to grading restrictions, four subcatchment areas have been designed without a storage component. Areas UNC1-2 are located at the perimeter of the site where tie-ins to existing property line grades cannot permit capture of runoff to the minor system. Peak discharges from uncontrolled areas are not anticipated to have deleterious effects on downstream systems based on available ponding volumes within downstream rights-of-way as identified in the Phase 7A Design Brief.

Table 5.3 summarizes the 2 and 100-year uncontrolled release rates from the proposed development.

Table 5.3 Peak Uncontrolled 2-Year and 100-Year Release Rates

Storm Return Period	Area ID	Area (ha)	Runoff 'C'	Tc (min)	Q _{release} (L/s)
2-year	UNC-1	0.02	0.55	10	2.3
	UNC-2	0.06	0.85	10	10.9
100-year	UNC-1	0.02	0.69	10	6.8
	UNC-2	0.06	1.00	10	29.8

5.5 Results and Discussion

The following section summarizes the key hydrologic and hydraulic model results for the proposed site and demonstrates the proposed stormwater management plan meets target peak rates established in the background studies. For detailed model results or inputs please refer to **Appendix C.3**.



Table 5.4: Target and Resultant Major and Minor System Release Rates

Storm event	Target Release Rate per Subdivision Design (L/s)	Phase 1 Minor System Release Rate (L/s)	Phase 1 Uncontrolled Release Rate (L/s)
2-year, 3-hour Chicago	113.0	60.0	13.2
100-year, 3-hour Chicago		111.3	36.6

5.6 Quality Control

Riverside South Pond 2 is a stormwater management pond located to the northeast of the adjacent Phase 7A subdivision. The pond has been designed to provide quality control treatment for the entirety of the subdivision including the proposed site plan block (Block 7). As outlined in the Design Brief for Phase 7A, Block 7 was assumed to have two runoff subcatchments with an associated area and runoff coefficient assigned for Phase 1 and Phase 2 of 1.47ha at C=0.75 and 9.40ha at C=0.40. The current design considers Phase 1 area of 1.47 at a runoff C=0.68, and the remaining lands for Phase 2 of 9.14ha at an assumed C=0.40. Given that the overall imperviousness of the two subcatchments has been reduced for the proposed design (rough AxC of 4.84 in the proposed condition vs 4.86 as contemplated in the Arcadis design brief), no impacts to the downstream stormwater management pond with respect to provision of adequate quality control for the block is expected.

6 Grading

The proposed RSCC&L Phase 1 development site measures approximately 1.74 ha in area. The topography across the site under existing conditions is relatively flat, and slopes gently toward the north. The objective of the grading design strategy is to satisfy the stormwater management requirements, adhere to permissible grade raise restrictions, and provide for minimum cover requirements for sewers, where possible. The grading design also follows the recommendations outlined in the Design Brief (2025).

The finished floor elevation (FFE) is consistent (93.65) between the two parts of the building (Library and Community Centre). Site grading has been established to provide barrier-free/accessible building entrances and direct most of the emergency overland drainage towards the Site Access at Jessie Chenevert Walk, in accordance with the City of Ottawa requirements. The top of foundation (TOF) elevation (93.85m) and all foundation openings are consistently a minimum of 150mm above the adjacent finished surface grades. The underside of footing (USF) elevation (91.95m) is provided with a minimum of 1.5m soil cover to protect against frost action as recommended in the geotechnical report (Paterson Group, 2024). Due to the limited grade change across the site and to adjacent sites, no retaining walls are proposed or anticipated. Some terracing is proposed along the back of curbs at the perimeter of the parking lot.

The grading plan (**Drawing GP-1**) was prepared considering the grade raise restrictions identified in the geotechnical report (Paterson Group, 2024). The permissible grade raise restrictions are shown in Drawing No., PG4958-20 in the Paterson Report, and included in the **Appendix D** excerpts. Due to the presence of the silty clay deposit in the RSCC&L Phase 1 Site, grade raise restrictions vary between up to 1.0m to 2.5m, as shown in Drawing No., PG4958-20.

7 Utilities

As the subject site lies within new residential community under development, electrical, gas, and telecommunications servicing for the proposed site will be readily available within the neighbouring rights-of-way. Exact size, location and routing of utilities will be established with the utility providers.

8 Approvals

Reporting on the Environmental Activity and Sector Registry (EASR) may be required for the site as some of the proposed works may be below the groundwater elevation shown in the geotechnical report. The geotechnical consultant shall determine whether EASR reporting is required prior to construction.

Per O.Reg. 525/98, the proposed stormwater management works service a single parcel of land, discharge into a separated storm sewer in Jessie Chenevert Walk which is included in the City's existing CLI ECA, and the site is not industrial, nor does it service industrial land. As such, the proposed works for the RSCC&L development are likely exempt from Environmental Compliance Approval requirements.



9 Erosion Control

To protect downstream water quality and prevent sediment build-up in catch basins and storm sewers, erosion and sediment control measures must be implemented during construction. The following recommendations will be included in the contract documents and communicated to the Contractor.

Drawing ECDS-1 provides an erosion and sediment control (ESC) concept plan. Development and implementation of a detailed erosion and sediment control plan will be the responsibility of the Contractor.

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit the extent of the exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with geotextiles, geogrid, or synthetic mulches.
6. Install silt barriers/fencing around the perimeter of the site to prevent the migration of sediment offsite.
7. Install track out control mats (mud mats) at the entrance/egress as shown in **Drawing ECDS-1** to prevent migration of sediment into the public ROW.
8. Provide sediment traps and basins during dewatering works.
9. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
10. Schedule the construction works at times which avoid flooding due to seasonal rains.

The Contractor will also be required to complete inspections and guarantee the proper performance of their erosion and sediment control measures at least after every rainfall. The inspections are to include:

- Verification that water is not flowing under silt barriers.
- Cleaning and changing the sediment traps placed on catch basins.

Refer to **Drawing ECDS-1** for the proposed location of silt fences, sediment traps, and other erosion control measures.



10 Geotechnical Investigation

A geotechnical report for the development was completed by Paterson Group Inc. in June 2024. The report summarizes the existing soil conditions within RSCC&L Phase 1 Site and construction recommendations. For details which are not summarized below, please see the Paterson report included in the submission package.

Subsurface soil conditions within the site were determined through field investigations conducted for the Riverside South Town Centre Phase 7A between April 2-8, 2024, in addition to information available from several previous investigations carried out by Paterson and others within the subject site and in adjacent sites. The current investigation included 19 boreholes, five of which were fitted with groundwater monitoring wells.

In general, soil stratigraphy consisted of a thin topsoil layer overlying a silty sand to sandy silt deposit (0.7 to 2.4 m thick) overlaying a silty clay deposit. In some locations the topsoil layer was underlain by a thin layer of silty clay and overlaying the silty sand layer.

Bedrock was estimated to occur at auger refusal between depths of 2.9 to 14.3m. Based on groundwater level readings measured in the monitoring wells and piezometers, the groundwater levels varied between 0.18 mbgs to 2.18 mbgs (93.17m to 90.32m elevation) as measured in April 2024. As groundwater levels fluctuate seasonally, these levels could vary at the time of construction, and the long-term groundwater levels were estimated to be at 2.5 to 3.5 mbgs. It should be noted that a post-development groundwater lowering of 0.5 m was considered in the permissible grade raise calculations and recommendations by Paterson. In addition, clay seals are recommended to be provided in the service trenches to reduce long-term lowering of the groundwater level at this site. Refer to the Paterson report (2024) for details.

Based on the results of the geotechnical investigation, the subject site is suitable for the proposed development, and it is recommended that the proposed building be founded on conventional spread footings placed on a suitable bearing surface. The report provides alternative foundation options if a suitable bearing surface is not encountered.

The geotechnical report recommends the pavement structure presented in

Table 10.1 below.

Table 10.1: Recommended Pavement Structure

Material	Driveways and Car-only Parking Areas	Local Residential Roadways
Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete	50 mm	40 mm
Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete	-	50 mm
BASE – OPSS Granular A Crushed Stone	150 mm	



SUBBASE – OPSS Granular B Type II	300 mm	400 mm
--------------------------------------	--------	--------

The geotechnical report also makes the following construction recommendations for other items pertaining to the civil servicing scope:

- Excavation, backfill, and engineered fill requirements.
- Pipe bedding and backfill requirements.
- Tree planting considerations and setbacks in sensitive marine clay soils.
- Foundation perimeter drains are recommended.
- All excess soil must be handled as per Ontario Regulation 406/19: On-Site and Excess Soil Management.



11 Conclusions and Recommendations

This Site Servicing and Stormwater Management report assessed the servicing conditions within the RSCC&L Phase 1 project site and evaluated the feasibility of new stormwater management infrastructure to meet LID infiltration targets and flow rate restrictions to the minor system. Other SPC design requirements appeared in this report including:

- potable water and fire flow servicing,
- sanitary servicing,
- stormwater servicing,
- grading and drainage,
- geotechnical considerations,
- utilities considerations,
- best management practices for erosion and sediment control during construction, and
- approvals considerations.

We trust that the information, servicing strategies, and the proposed stormwater management systems presented in this report are adequate to support the Site Plan Control Application for this development.

Please contact the author or engineer of record of this report if you have any questions or concerns.



Appendix A Water Servicing

A.1 Domestic Water Demands



Riverside South Community Centre and Library, Ottawa, ON - Domestic Water Demand Estimates

Site Plan provided by Diamond Schmitt (2026-03-16)

Stantec Project No. 160402150

City File No.

Revision: 1

Date Created: 6-Jul-2026

Created by: AG

Checked by: DT

Date Checked: 7-Jul-2026

**Demand conversion factors per Per Table 4.2
City Ottawa Water Distribution Design Guideline (Dec 2025):**

Commercial/Institutional	28000	L/gross ha/day
Maximum Daily Peaking Factor	1.5	x AVDY
Peak Hour Peaking Factor	1.8	x MXDY

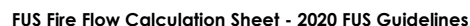
Unit Type	Institutional Area (m ²)	Average Day Demand (AVDY)		Maximum Day Demand (MXDY)		Peak Hour Demand (PKHR)	
		(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Institutional							
Library and Community Centre	3934.09	7.65	0.13	11.47	0.19	20.65	0.34
Splash Pad ¹	-	189.3	3.16	189.3	3.16	189.30	3.16
Total	3934.09	196.9	3.3	200.8	3.3	210.0	3.5

Notes:

1. Assume the splash pad water play features are timed to demand a maximum flow rate of US 50GPM (189.3 L/min).

A.2 FUS Calculation Sheets





Fire Flow Calculation #: 1

Date Checked: 10-Jul-2026

Notes: A Community Centre and Library, principally one storey building. Information from Site Plan by Diamond Schmitt dated March 16, 2026. Sprinklered with floor assemblies / load bearing walls as 1hr rated assemblies per OBC 3.2.2.52

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type II - Noncombustible Construction / Type IV-A - Mass Timber Construction										0.8	-
2	Determine Effective Floor Area	Sum of Largest Floor + 25% of Two Additional Floors					Vertical Openings Protected?					YES	-
		3934										3934.09	-
3	Determine Required Fire Flow	(F = 220 x C x A ^{1/2}). Round to nearest 1000 L/min										-	11000
4	Determine Occupancy Charge	Limited Combustible										-15%	9350
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-4675
		Standard Water Supply										-10%	
		Fully Supervised										-10%	
		% Coverage of Sprinkler System										100%	
6	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	Firewall / Sprinklered ?	-	-			
		North	> 30	0	1	0-20	Type I-II - Unprotected Openings	NO	0%	935			
		East	20.1 to 30	74	2	> 100	Type V	NO	10%				
		South	> 30	0	1	0-20	Type I-II - Unprotected Openings	NO	0%				
		West	> 30	0	1	0-20	Type I-II - Unprotected Openings	NO	0%				
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min										6000	
		Total Required Fire Flow in L/s										100.0	
		Required Duration of Fire Flow (hrs)										2.00	
		Required Volume of Fire Flow (m ³)										720	

A.3 Water Modelling Background Report Excerpts



2 Water Supply

2.1 Existing Conditions

As noted in Section 1.5 there is an existing 406 mm watermain on Earl Armstrong Road, a 305 mm watermain on Portico Way and a 610 mm watermain on Limebank Road which has been constructed but not commissioned at the time of this report, connections to all of these watermains are proposed to service the site. **Figure 1.3** shows the location of the existing watermains.

2.2 Riverside South Community Infrastructure Servicing Study Update Phase 1 Mosquito Creek Study Area (2023 IBI Group)

The Infrastructure Servicing Study Update provided a macro level water servicing plan for the Phase 1 lands of the Mosquito Creek Study Area (Phase 1 ISSU). Major watermains in the Phase 1 lands are shown on Figure 3-2 Potable Water Servicing Plan, a copy is included in **Appendix B**. The figure shows the extension south of the 305 mm watermain on Portico Way and the 610 mm watermain on Limebank Road in the vicinity of the 980 Earl Armstrong & 4700 Limebank Road development.

2.3 Design Criteria

2.3.1 Water Demands

Water demands have been calculated for the site based on per unit population density and consumption rates taken from Tables 4.1 and 4.2 of the City of Ottawa Design Guidelines – Water Distribution and are summarized as follows:

• Single Family	3.4 person per unit
• Townhouse and Semi-Detached	2.7 person per unit
• Average Apartment	1.8 person per unit
• Residential Average Day Demand	280 l/cap/day
• Residential Peak Daily Demand	700 l/cap/day
• Residential Peak Hour Demand	1,540 l/cap/day
• ICI Average Day Demand	28,000 l/ha/day
• ICI Peak Daily Demand	42,000 l/ha/day
• ICI Peak Hour Demand	75,600 l/ha/day
• Employment Average Day Demand	35,000 l/ha/day
• Employment Peak Daily Demand	52,500 l/ha/day
• Employment Peak Hour Demand	94,500 l/ha/day

A water demand was calculated using the Draft Plan per **Figure 1.3** in **Appendix A**. The land use for Blocks 1 to 4, 6, 11, 13 and 14 is designated Town Center, a plan showing unit allocation within the Town Center is included

in **Appendix C** and summarized in Table 3-2 in Section 3.3. There is a small area of commercial land which has the Employment sanitary flow rate of 35,000 l/ha/day, Block 7 the District Park uses the institutional rate of 28,000 l/ha/day. The total water demands are as follows;

- Average Day 14.77 l/s
- Maximum Day 30.59 l/s
- Peak Hour 65.14 l/s

2.3.2 System Pressure

The Ottawa Design Guidelines – Water Distribution (WDG001), July 2010, City of Ottawa, Clause 4.2.2 states that the preferred practice for design of a new distribution system is to have normal operating pressures range between 345 kPa (50 psi) and 552 kPa (80 psi) under maximum daily flow conditions. Other pressure criteria identified in Clause 4.2.2 of the guidelines are as follows:

Minimum Pressure	Minimum system pressure under peak hour demand conditions shall not be less than 276 kPa (40 psi)
Fire Flow	During the period of maximum day demand, the system pressure shall not be less than 140 kPa (20 psi) during a fire flow event.
Maximum Pressure	Maximum pressure at any point in the distribution system shall not exceed 689 kPa (100 psi). In accordance with the Ontario Building/Plumbing Code, the maximum pressure should not exceed 552 kPa (80 psi). Pressure reduction controls will be required for buildings where it is not possible/feasible to maintain the system pressure below 552 kPa.

2.3.3 Fire Flow Rates

There are no building designs for this development at this time so that a fire flow calculation using the Underwriters Survey (FUS) method cannot be done at this time. Boundary conditions have been provided for a 13,000 l/min fire flow which was the fire flow used in the Phase 1 ISSU water analysis. A boundary condition with a 10,000 l/min fire flow has also been provided as non-combustible and fire resistant multi-story buildings with protected vertical openings typically have fire flow rates less than 10,00 l/min. For this report the fire flow analysis is run with a 13,000 l/min fire flow.

2.3.4 Boundary Conditions

The City of Ottawa has provided four boundary conditions at the watermain connection locations at Portico Way (Connection 1 and 5), two connections on Earl Armstrong (Connections 2 and 3) and at Limebank Road (Connection 4). Boundary conditions are provided for the existing pressure zone and for the SUC Zone Reconstruction. A copy of the boundary condition is included in **Appendix B** and summarized as follows;

Table 2-1 Water Supply Boundary Conditions

	Max HGL (Basic Day)	Peak Hour	Max Day + Fire (10,000 l/min Fire Flow)	Max Day + Fire (13,000 l/min Fire Flow)
Connection 1 Existing Conditions	132.1 m	124.9 m	124.9 m	123.9 m
Connection 1 Future SUC Zone	146.8 m	143.5 m	142.8 m	141.3 m
Connection 2 Existing Conditions	132.1 m	124.9 m	124.9 m	124.9 m
Connection 2 Future SUC Zone	146.8 m	143.5 m	143.5 m	142.4 m
Connection 3 Existing Conditions	132.1 m	124.9 m	124.9 m	123.9 m
Connection 3 Future SUC Zone	146.8 m	143.4 m	142.8 m	141.3 m
Connection 4 Existing Conditions	132.1 m	124.9 m	124.7 m	122.5 m
Connection 4 Future SUC Zone	146.8 m	143.4 m	141.9 m	139.9 m
Connection 5 Existing Conditions	132.1 m	124.9 m	124.8 m	122.6 m
Connection 5 Future SUC Zone	146.8 m	143.4 m	142.0 m	140.0 m

2.3.5 Hydraulic Model

A computer model has been created for the subject site using the InfoWater 12.4 program. The model includes the hydraulic boundary conditions at the connections to existing watermain.

2.4 Proposed Water Plan

2.4.1 Modeling Results

The hydraulic model was run under basic day, maximum day with fire flows and under peak hour conditions. Water pipes are sized to provide sufficient pressure and to deliver the required fire flows.

Results of the hydraulic model are included in **Appendix B** and summarized as follows:

Table 2-2 Hydraulic model results

Scenario	Existing Zone	SUC Zone Reconfiguration
Basic Day (Max HGL) Pressure Range	373.3 to 391.0 kPa	517.4 to 535.0 kPa
Peak Hour Pressure Range	302.8 to 320.4 kPa	484.1 to 502.4 kPa
Max Day + 13,000 l/min Fire Flow Minimum Residual Pressure	220.5 kPa	314.3 kPa

A comparison of the results and design criteria is summarized as follows:

Maximum Pressure	All nodes under both pressure zone scenarios have basic day pressures under 552 kPa (80 psi), pressure reducing control is not required for this development.
Minimum Pressure	All nodes under both scenarios pressure zone scenarios exceed the minimum value of 276 kPa (40 psi).
Fire Flow	Under the existing pressure zone condition with a 13,000 l/min fire all nodes have residual pressures above the minimum required pressure of 140 kPa (20 psi). Under the Post SUC Zone Reconfirmation the residual pressure increases to 327.8 kPa for the 13,000 l/min fire. All nodes can accommodate a 13,000 l/min fire flow, the required fire flow will be determined when building layouts are available.

2.4.2 Watermain Layout

As per the Phase 1 ISSU a 305mm watermain is extended south on Portico Way to the transit corridor. A 250 mm main is proposed on Town Square Street for the district park as per item 29 b) in the pre-consultation meeting as discussed in **Section 1.6**. All other watermains on Flavescent Street, Jessie Chenevert Walk and Springbriar Street in the development are proposed to be 204 mm diameter with the main on Flavescent Street connecting to an existing 305mm watermain stub at Earl Armstrong Road. **Figure 2.1, Conceptual Water Plan** from the Assessment of Adequacy of Public Services report is included in **Appendix B**

As mentioned in **Section 1.4** the northern portion of the development will be developed in advance of the southern portion which will result un-looped mains on Town Square Street and Portico Way. On Town Square Street there are two hydrants on an un-looped main in front of the District Park, hydrant flows can be confirmed using the methodology of Technical Bulletin ISTB-2018-02 Appendix I – Guideline on Coordination of Hydrant Placement with Required Fire Flow when there is a site plan for the District Park. Fire protection for the District Park is also provided by hydrants on Jessie Chenevert Walk. On Town Square Street a connection to the existing watermain servicing the LRT Limebank Station can be constructed as shown on **Figure 2.1 in Appendix B** if required. The 305 mm watermain on Portico Way will not need to be extended until the southern portion of the site is developed which is not proposed at this time, should Portico Way be constructed in advance of the southern development the watermain can be installed but not commissioned until it extended south across the

Design Brief Phase 7A
980 Earl Armstrong Road & 4700 Limebank Road, Riverside South

transit corridor. The spacing of isolation valves and fire hydrants in Phase 7A are per Tables 4.8 and 4.9 on the Ottawa Design Guidelines – Water Distribution

* Proposed watermains added to distribution model

Results

Existing Condition (Pre- SUC Pressure Zone Reconfiguration)

Connection 1 - Portico Way North

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	132.1	57.2
Peak Hour	124.9	47.0
Max Day plus Fire Flow #1	124.9	47.0
Max Day plus Fire Flow #2	123.9	45.6

¹ Ground Elevation = 91.9 m

Connection 2 - Earl Armstrong Road West

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	132.1	56.6
Peak Hour	124.9	46.4
Max Day plus Fire Flow #1	124.9	46.4
Max Day plus Fire Flow #2	124.9	46.4

¹ Ground Elevation = 92.3 m

Connection 3 - Earl Armstrong Road East

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	132.1	55.1
Peak Hour	124.9	45.0
Max Day plus Fire Flow #1	124.9	44.9
Max Day plus Fire Flow #2	123.9	43.6

¹ Ground Elevation = 93.3 m

Connection 4 - Limebank Road

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	132.1	55.4
Peak Hour	124.9	45.2
Max Day plus Fire Flow #1	124.7	44.9
Max Day plus Fire Flow #2	122.5	41.8

¹ Ground Elevation = 93.1 m

Connection 5 - Portico Way South

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	132.1	57.3
Peak Hour	124.9	47.1
Max Day plus Fire Flow #1	124.8	47.0
Max Day plus Fire Flow #2	122.6	43.9

¹ Ground Elevation = 91.8 m

Future Condition (Post- SUC Pressure Zone Reconfiguration)

Connection 1 - Portico Way

Demand Scenario	Head (m)	Pressure ¹ (psi)
Maximum HGL	146.8	78.2
Peak Hour	143.5	73.3
Max Day plus Fire Flow #1	142.8	72.4
Max Day plus Fire Flow #2	141.3	70.3

¹ Ground Elevation = 91.9 m

Connection 2 - Earl Armstrong Road West

Demand Scenario	Head (m)	Pressure ¹ (psi)
Maximum HGL	146.8	77.6
Peak Hour	143.5	72.8
Max Day plus Fire Flow #1	143.5	72.8
Max Day plus Fire Flow #2	142.4	71.3

¹ Ground Elevation = 92.3 m

Connection 3 - Earl Armstrong Road East

Demand Scenario	Head (m)	Pressure ¹ (psi)
Maximum HGL	146.8	76.1
Peak Hour	143.4	71.3
Max Day plus Fire Flow #1	142.8	70.4
Max Day plus Fire Flow #2	141.3	68.2

¹ Ground Elevation = 93.3 m

Connection 4 - Limebank Road

Demand Scenario	Head (m)	Pressure ¹ (psi)
Maximum HGL	146.8	76.4
Peak Hour	143.4	71.5
Max Day plus Fire Flow #1	141.9	69.3
Max Day plus Fire Flow #2	139.9	66.5

¹ Ground Elevation = 93.1 m

Connection 5 - Portico Way South

Demand Scenario	Head (m)	Pressure ¹ (psi)
Maximum HGL	146.8	78.3
Peak Hour	143.4	73.5
Max Day plus Fire Flow #1	142.0	71.4
Max Day plus Fire Flow #2	140.0	68.6

¹ Ground Elevation = 91.8 m

Disclaimer

The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.



Arcadis
333 PRESTON STREET
OTTAWA, ON
K1S 5N4

IBI GROUP

WATERMAIN DEMAND CALCULATION SHEET

PROJECT : RSS TOWN CENTER - PHASE 7A
LOCATION : CITY OF OTTAWA
DEVELOPER : RIVERSIDE SOUTH DEVELOPMENT CORPORATION

FILE: 144320
DATE PRINTED: 06-Jun-24
DESIGN: LE
PAGE: 1 OF 1

NODE	RESIDENTIAL					NON-RESIDENTIAL			AVERAGE DAILY DEMAND (l/s)			MAXIMUM DAILY DEMAND (l/s)			MAXIMUM HOURLY DEMAND (l/s)			FIRE DEMAND
	UNITS				POP'N	INDTRL (ha.)	EMP (ha.)	INST. (ha.)	Res.	Non-res.	Total	Res.	Non-res.	Total	Res.	Non-res.	Total	(l/min)
	SF	SD & TH	APT	TC (ha)														
C05			45		81				0.26	0.00	0.26	0.66	0.00	0.66	1.44	0.00	1.44	13,000
C12			350		630		0.19		2.04	0.08	2.12	5.10	0.12	5.22	11.23	0.21	11.44	13,000
C14			350		630		0.19		2.04	0.08	2.12	5.10	0.12	5.22	11.23	0.21	11.44	13,000
C16			175		315				1.02	0.00	2.12	2.55	0.00	2.55	5.61	0.00	5.61	13,000
C18			235		423				1.37	0.00	1.37	3.43	0.00	3.43	7.54	0.00	7.54	13,000
C22								10.60	0.00	3.44	3.44	0.00	5.15	5.15	0.00	9.28	9.28	13,000
C32		120			324				1.05	0.00	1.05	2.63	0.00	2.63	5.78	0.00	5.78	13,000
C33		48			130				0.42	0.00	0.42	1.05	0.00	1.05	2.31	0.00	2.31	13,000
C35		205			554				1.79	0.00	1.79	4.48	0.00	4.48	9.87	0.00	9.87	13,000
C36		9			24				0.08	0.00	0.08	0.20	0.00	0.20	0.43	0.00	0.43	13,000
TOTALS					3,110		0.38	10.60			14.77			30.59			65.14	

ASSUMPTIONS

RESIDENTIAL DENSITIES

- Single Family (SF)
- Semi Detached (SD) & Townhouse (TH)
- Apartment (APT)
- Town Centre Area (TC)

3.4 p / p / u
2.7 p / p / u
1.8 p / p / u
122.4 p / p / ha

AVG. DAILY DEMAND

- Residential
- Employment
- INST

280 l / cap / day
35,000 l / ha / day
28,000 l / ha / day

MAX. DAILY DEMAND

- Residential
- Employment
- INST

700 l / cap / day
52,500 l / ha / day
42,000 l / ha / day

MAX. HOURLY DEMAND

- Residential
- Employment
- INST

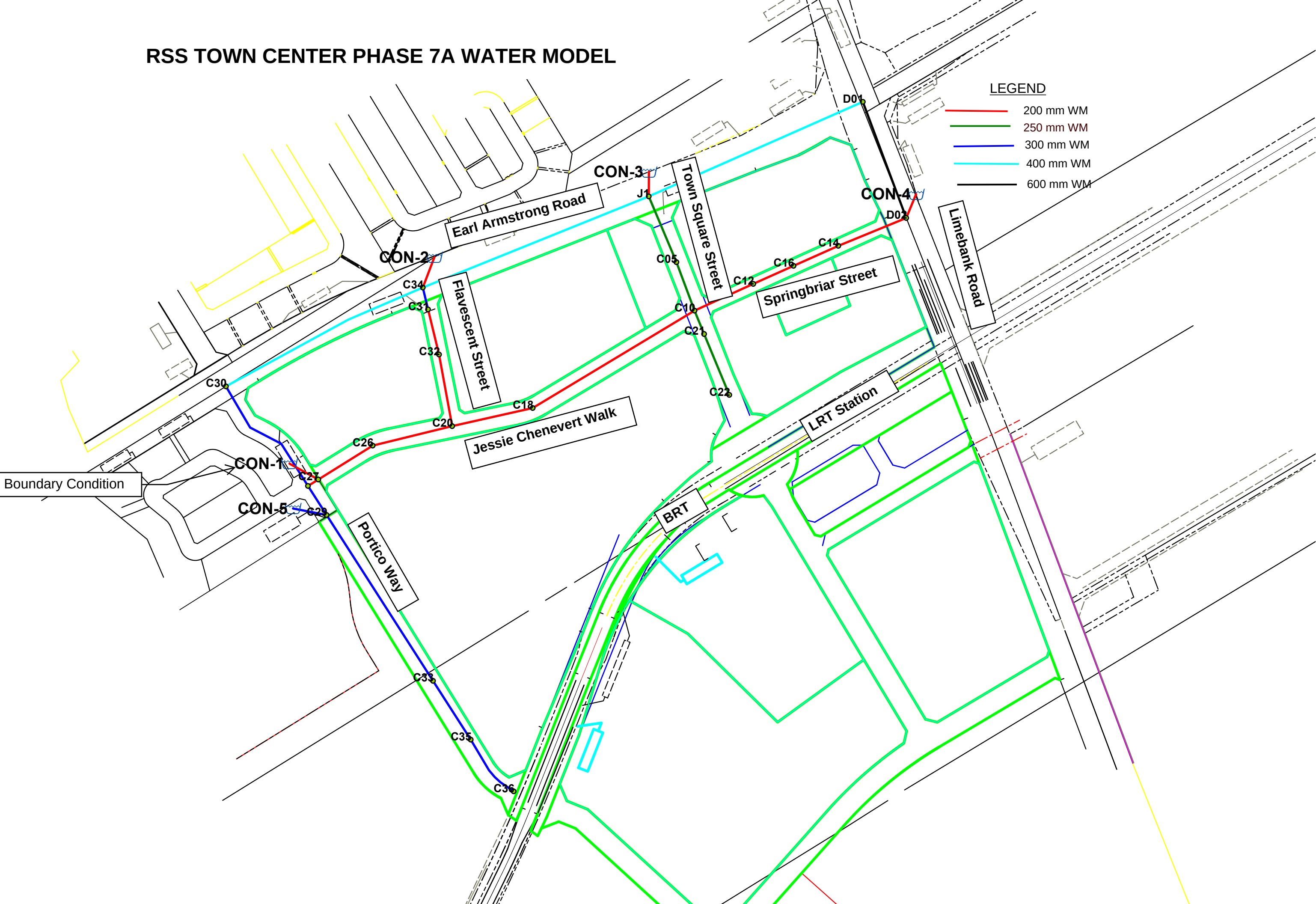
1,540 l / cap / day
94,500 l / ha / day
75,600 l / ha / day

FIRE FLOW

- SF, SD, TH & ST
- ICI

10,000 l / min
13,000 l / min

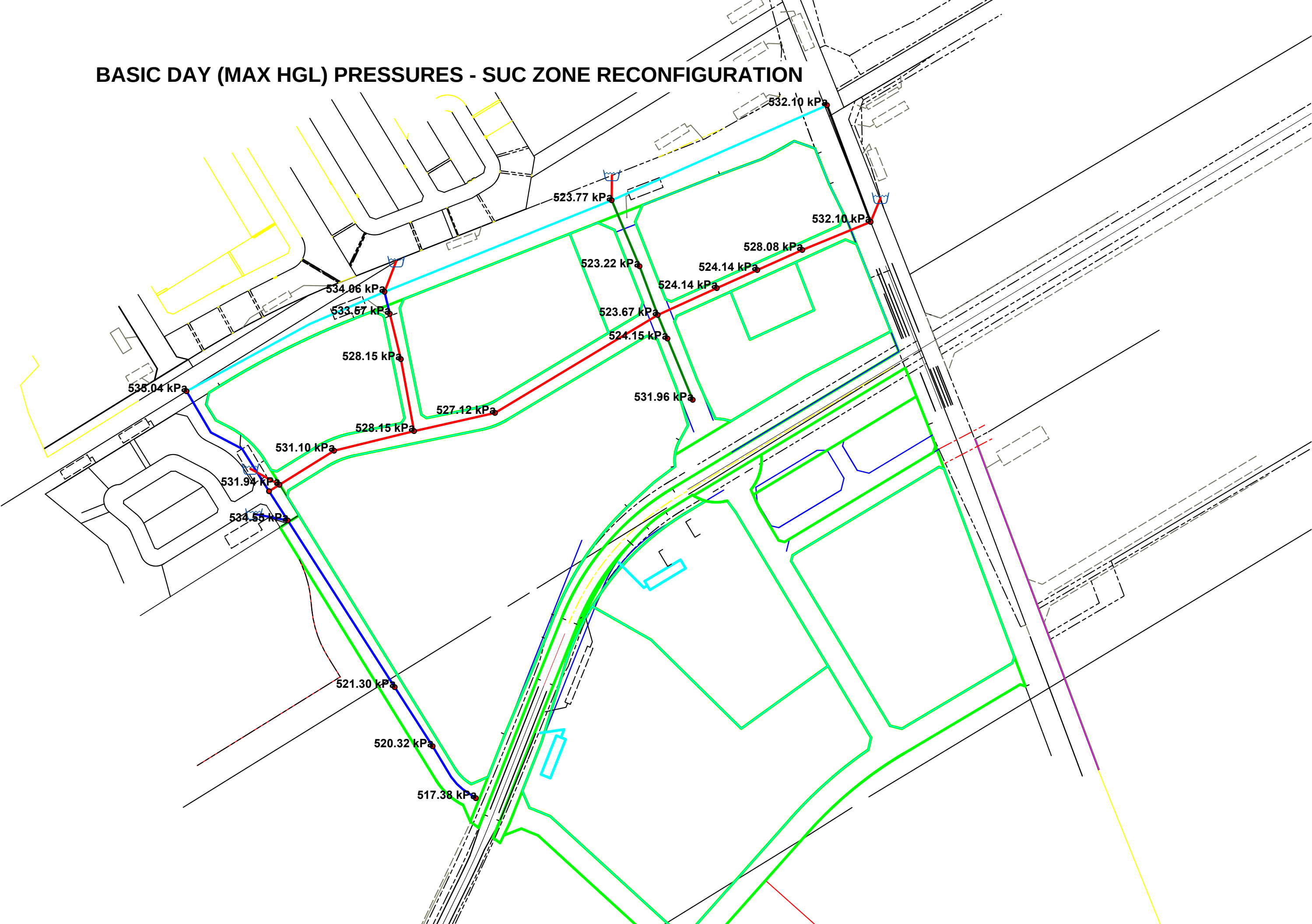
RSS TOWN CENTER PHASE 7A WATER MODEL



LEGEND

- 200 mm WM
- 250 mm WM
- 300 mm WM
- 400 mm WM
- 600 mm WM

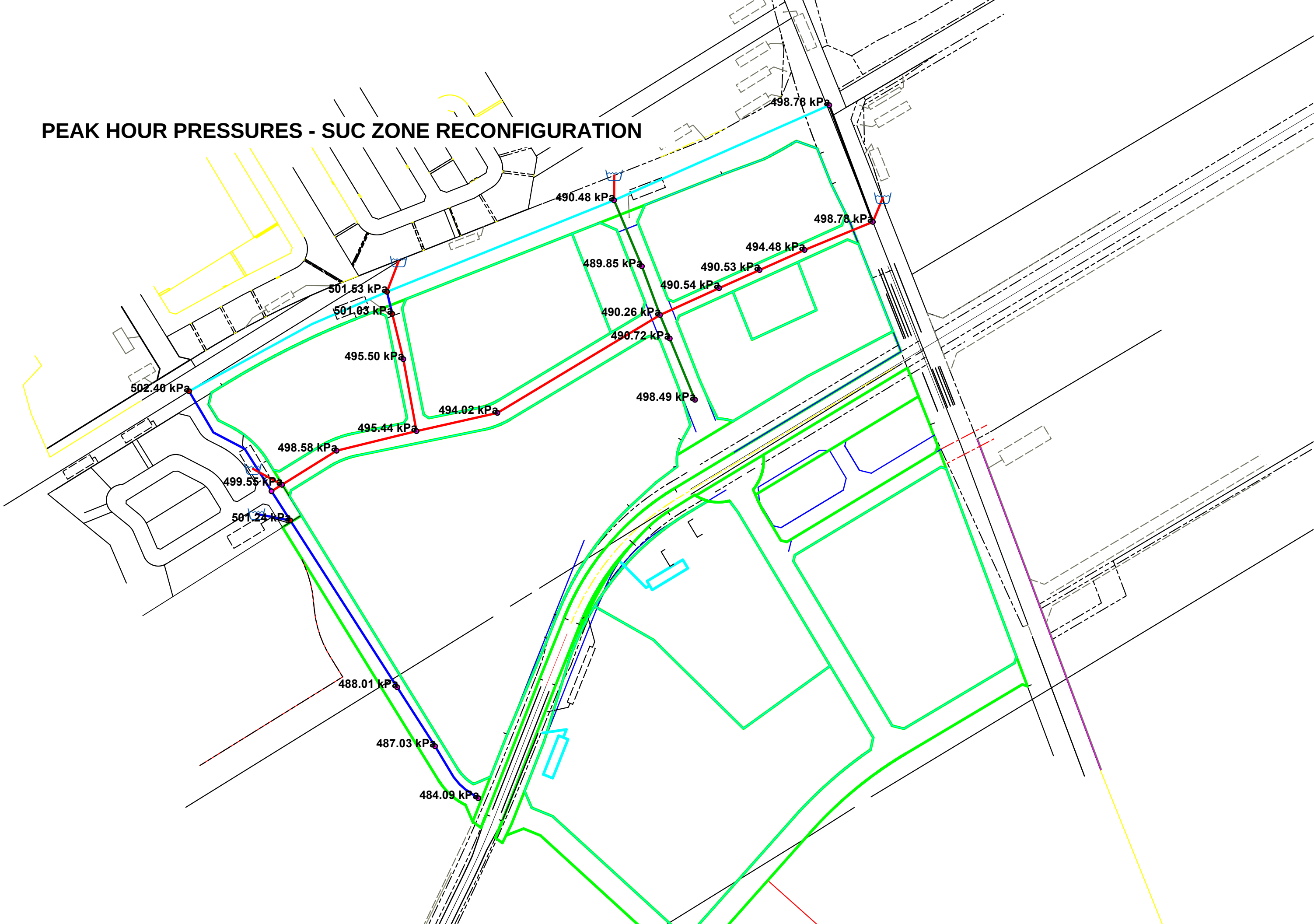
BASIC DAY (MAX HGL) PRESSURES - SUC ZONE RECONFIGURATION



Basic Day (Max HGL) - SUC Zone - Junction Report

		ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (kPa)
1	☐	C05	0.26	93.40	146.79	523.22
2	☐	C10	0.00	93.35	146.79	523.67
3	☐	C12	2.12	93.30	146.79	524.14
4	☐	C14	2.12	92.90	146.79	528.08
5	☐	C16	1.02	93.30	146.79	524.14
6	☐	C18	1.37	93.00	146.79	527.12
7	☐	C20	0.00	92.90	146.80	528.15
8	☐	C21	0.00	93.30	146.79	524.15
9	☐	C22	3.44	92.50	146.79	531.96
10	☐	C26	0.00	92.60	146.80	531.10
11	☐	C27	0.00	92.52	146.80	531.94
12	☐	C28	0.00	92.60	146.80	531.12
13	☐	C29	0.00	92.25	146.80	534.55
14	☐	C30	0.00	92.20	146.80	535.04
15	☐	C31	0.00	92.35	146.80	533.57
16	☐	C32	1.05	92.90	146.80	528.15
17	☐	C33	0.42	93.60	146.80	521.30
18	☐	C34	0.00	92.30	146.80	534.06
19	☐	C35	1.79	93.70	146.80	520.32
20	☐	C36	0.08	94.00	146.80	517.38
21	☐	D01	0.00	92.50	146.80	532.10
22	☐	D02	0.00	92.50	146.80	532.10
23	☐	J1	0.00	93.35	146.80	523.77

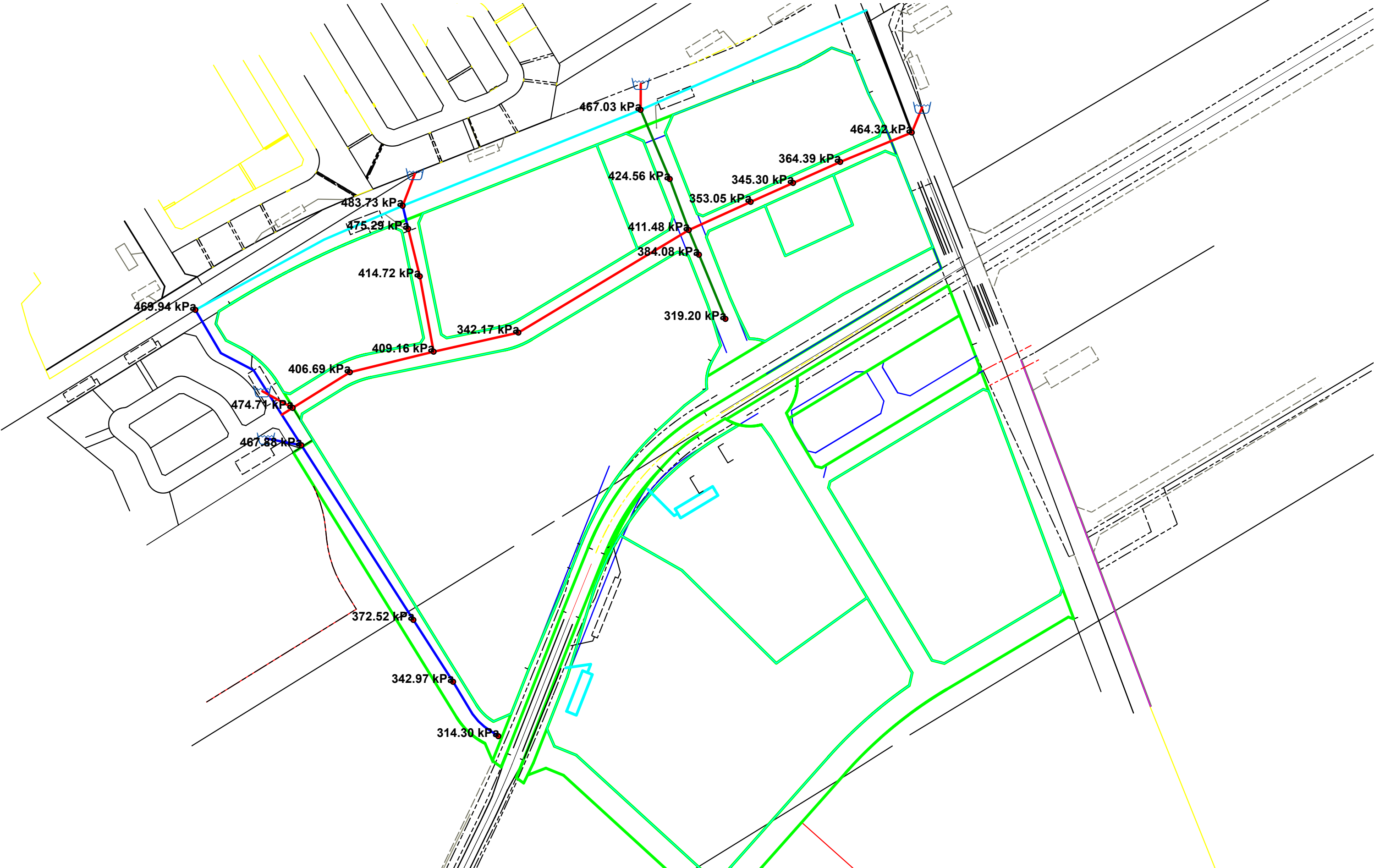
PEAK HOUR PRESSURES - SUC ZONE RECONFIGURATION



Peak Hour - SUC Zone - Junction Report

		ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (kPa)
1	☐	C05	0.66	93.40	143.39	489.85
2	☐	C10	0.00	93.35	143.38	490.26
3	☐	C12	5.22	93.30	143.36	490.54
4	☐	C14	5.22	92.90	143.36	494.48
5	☐	C16	2.55	93.30	143.36	490.53
6	☐	C18	3.43	93.00	143.41	494.02
7	☐	C20	0.00	92.90	143.46	495.44
8	☐	C21	0.00	93.30	143.38	490.72
9	☐	C22	5.15	92.50	143.37	498.49
10	☐	C26	0.00	92.60	143.48	498.58
11	☐	C27	0.00	92.52	143.49	499.55
12	☐	C28	0.00	92.60	143.44	498.24
13	☐	C29	0.00	92.25	143.40	501.24
14	☐	C30	0.00	92.20	143.47	502.40
15	☐	C31	0.00	92.35	143.48	501.03
16	☐	C32	2.63	92.90	143.47	495.50
17	☐	C33	1.05	93.60	143.40	488.01
18	☐	C34	4.48	92.30	143.48	501.53
19	☐	C35	0.20	93.70	143.40	487.03
20	☐	C36	0.00	94.00	143.40	484.09
21	☐	D01	0.00	92.50	143.40	498.78
22	☐	D02	0.00	92.50	143.40	498.78
23	☐	J1	0.00	93.35	143.40	490.48

MAX DAY + FIRE (13,000 l/min) RESIDUAL PRESSURES - SUC ZONE RECONFIGURATION



Max Day + Fire (13,000 l/min) - SUC Zone - Fireflow Design Report

		ID	Total Demand (L/s)	Available Flow at Hydrant (L/s)	Critical Node ID	Critical Node Pressure (kPa)	Critical Node Head (m)	Design Flow (L/s)	Design Pressure (kPa)	Design Fire Node Pressure (kPa)
1	☐	C05	218.11	707.09	C05	139.96	107.68	707.09	139.96	139.96
2	☐	C10	216.67	612.51	C10	139.96	107.63	612.51	139.96	139.96
3	☐	C12	228.11	416.33	C12	139.96	107.58	416.34	139.96	140.02
4	☐	C14	228.11	438.92	C14	139.96	107.18	438.92	139.96	140.00
5	☐	C16	222.28	395.64	C16	139.96	107.58	395.64	139.96	140.07
6	☐	C18	224.21	373.92	C18	139.96	107.28	373.92	139.96	139.97
7	☐	C20	216.67	535.55	C20	139.96	107.18	535.55	139.96	140.06
8	☐	C21	216.67	478.15	C21	139.96	107.58	478.15	139.96	140.35
9	☐	C22	225.95	344.99	C22	139.96	106.78	344.99	139.96	139.98
10	☐	C26	216.67	509.81	C26	139.96	106.88	509.81	139.96	139.98
11	☐	C27	216.67	4,005.32	C27	140.02	106.81	4,005.71	139.96	139.61
12	☐	C29	216.67	9,975.58	C36	122.71	106.52	9,689.13	139.96	157.75
13	☐	C30	216.67	1,518.26	C30	139.97	106.48	1,518.28	139.96	139.96
14	☐	C31	216.67	1,662.51	C31	139.97	106.63	1,662.54	139.96	139.96
15	☐	C32	222.45	558.30	C32	139.96	107.18	558.30	139.96	140.01
16	☐	C33	218.98	462.77	C36	135.95	107.87	459.50	139.96	144.05
17	☐	C34	216.67	4,758.11	C34	140.05	106.59	4,758.74	139.96	139.96
18	☐	C35	226.54	398.95	C36	137.02	107.98	396.92	139.96	142.90
19	☐	C36	217.10	343.67	C36	139.96	108.28	343.67	139.96	140.00
20	☐	D02	216.67	4,096.07	D02	140.02	106.79	4,096.50	139.96	139.95
21	☐	J1	216.67	5,007.51	J1	140.06	107.64	5,008.29	139.96	139.96

Appendix B Wastewater Servicing

B.1 Sanitary Sewer Design Sheet



	SUBDIVISION:		SANITARY SEWER DESIGN SHEET (City of Ottawa)										DESIGN PARAMETERS																								
	Riverside South Community Centre and Library												MAX PEAK FACTOR (RES.)= 4.0								AVG. DAILY FLOW / PERSON 280 l/p/day				MINIMUM VELOCITY 0.60 m/s												
	DATE: 7/10/2026												MIN PEAK FACTOR (RES.)= 2.0								COMMERCIAL 28,000 l/ha/day				MAXIMUM VELOCITY 3.00 m/s												
	REVISION: 1												PEAKING FACTOR (INDUSTRIAL): 2.4								INDUSTRIAL (HEAVY) 55,000 l/ha/day				MANNINGS n 0.013												
	DESIGNED BY: JP												PEAKING FACTOR (ICI >20%): 1.5								INDUSTRIAL (LIGHT) 35,000 l/ha/day				BEDDING CLASS B												
CHECKED BY: DT		FILE NUMBER: 160402150										PERSONS / SINGLE 3.4				INSTITUTIONAL 28,000 l/ha/day				MINIMUM COVER 2.50 m																	
												PERSONS / TOWNHOME 2.7				INFILTRATION 0.33 l/s/Ha				HARMON CORRECTION FACTOR 0.8																	
												PERSONS / APARTMENT 1.8																									
LOCATION			RESIDENTIAL AREA AND POPULATION										COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+H	INFILTRATION			TOTAL	PIPE									
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT	POP.	CUMULATIVE AREA POP. (ha)	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)					
	1B	1	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.00	0.00	0.00	0.00	0.20	0.00	0.00	0.1	0.20	0.20	0.1	0.2	20.6	150	PVC	DR 28	1.00	15.3	1.06%	0.86	0.23				
	1A	1	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.00	0.00	0.00	0.00	0.19	0.19	0.00	0.00	0.1	0.19	0.19	0.1	0.2	8.4	150	PVC	DR 28	1.00	15.3	1.01%	0.86	0.23			
	1	11	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.00	0.00	0.00	0.00	0.39	1.35	1.35	0.2	1.35	1.74	0.6	0.8	61.1	150	PVC	DR 28	1.00	15.3	4.98%	0.86	0.38				
																										150											

B.2 Sanitary Design Background Report Excerpts



2025-04-03 3:24 PM

Appendix C Stormwater Management and Servicing

C.1 Storm Sewer Design Sheet



C.2 Modified Rational Method (MRM) Design Sheet



Stormwater Management Calculations

File No: **PROJECT # 160402150**

Project: **Riverside South Community Centre and Library**

Date: **10-Jul-26**

SWM Approach:
Post-development to Predefined Release Rate

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Runoff Coefficient Table							
Catchment Type	Sub-catchment Area	ID / Description	Area (ha) "A"	Runoff Coefficient "C"	"A x C"	Overall Runoff Coefficient	
Uncontrolled - Non-Tributary	UNC-2	Hard	0.056	0.9	0.050	0.051	0.850
		Soft	0.004		0.001		
		Subtotal		0.06			
Uncontrolled - Non-Tributary	UNC-1	Hard	0.010	0.9	0.009	0.011	0.550
		Soft	0.010		0.002		
		Subtotal		0.02			
Uncontrolled - Tributary	L105B	Hard	0.020	0.9	0.018	0.034	0.340
		Soft	0.080		0.016		
		Subtotal		0.1			
Controlled - Tributary	L105A	Hard	0.110	0.9	0.099	0.099	0.900
		Soft	0.000		0.000		
		Subtotal		0.11			
Controlled - Tributary	L200A	Hard	0.070	0.9	0.063	0.063	0.900
		Soft	0.000		0.000		
		Subtotal		0.07			
Controlled - Tributary	L202A	Hard	0.065	0.9	0.059	0.0595	0.850
		Soft	0.005		0.001		
		Subtotal		0.07			
Controlled - Tributary	L201A	Hard	0.070	0.9	0.063	0.063	0.900
		Soft	0.000		0.000		
		Subtotal		0.07			
Controlled - Tributary	L103A	Hard	0.110	0.9	0.099	0.105	0.750
		Soft	0.030		0.006		
		Subtotal		0.14			
Controlled - Tributary	L107C	Hard	0.011	0.9	0.010	0.0338	0.260
		Soft	0.119		0.024		
		Subtotal		0.13			
Uncontrolled - Tributary	L107A	Hard	0.199	0.9	0.179	0.2352	0.490
		Soft	0.281		0.056		
		Subtotal		0.48			
Uncontrolled - Tributary	L104C	Hard	0.200	0.9	0.180	0.18	0.900
		Soft	0.000		0.000		
		Subtotal		0.2			
Uncontrolled - Tributary	L104B	Hard	0.190	0.9	0.171	0.171	0.900
		Soft	0.000		0.000		
		Subtotal		0.19			
Controlled - Tributary	TANK	Hard	0.000	0.9	0.000	0	0.000
		Soft	0.000		0.000		
		Subtotal		0			
Controlled - Tributary	L104A	Hard	0.083	0.9	0.075	0.078	0.780
		Soft	0.017		0.003		
		Subtotal		0.1			
Total				1.740	1.184		
Overall Runoff Coefficient= C:							0.68

Total Controlled Roof Areas

0.000 ha

Total Tributary Surface Areas (Controlled and Uncontrolled)

1.660 ha

Total Tributary Area to Outlet

1.660 ha

Total Uncontrolled Areas (Non-Tributary)

0.080 ha

Total Site

1.740 ha

Stormwater Management Calculations

Project #PROJECT # 160402150, Riverside South Community Centre and Library
Modified Rational Method Calculations for Storage

2 yr Intensity City of Ottawa	$I = a/(t + b)^c$	a = 732.951	t (min)	I (mm/hr)
		b = 6.199	10	76.81
		c = 0.81	20	52.03
			30	40.04
			40	32.86
			50	28.04
			60	24.56
			70	21.91
			80	19.83
			90	18.14
			100	16.75
			110	15.57
			120	14.56

2 YEAR Post-Development Target Release from Portion of Site				
Subdrainage Area: Predevelopment Tributary Area to Outlet				
Area (ha):	1.7400			
C:	0.75			
Q:	113.0 L/s			

2 YEAR Modified Rational Method for Entire Site				
Subdrainage Area: UNC-2 Uncontrolled - Non-Tributary				
Area (ha):	0.06			
C:	0.85			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	10.9	10.9		
20	52.0	7.4	7.4		
30	40.0	5.7	5.7		
40	32.9	4.7	4.7		
50	28.0	4.0	4.0		
60	24.6	3.5	3.5		
70	21.9	3.1	3.1		
80	19.8	2.8	2.8		
90	18.1	2.6	2.6		
100	16.7	2.4	2.4		
110	15.6	2.2	2.2		
120	14.6	2.1	2.1		

Subdrainage Area: UNC-1 Uncontrolled - Non-Tributary				
Area (ha):	0.02			
C:	0.55			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	2.3	2.3		
20	52.0	1.6	1.6		
30	40.0	1.2	1.2		
40	32.9	1.0	1.0		
50	28.0	0.9	0.9		
60	24.6	0.8	0.8		
70	21.9	0.7	0.7		
80	19.8	0.6	0.6		
90	18.1	0.6	0.6		
100	16.7	0.5	0.5		
110	15.6	0.5	0.5		
120	14.6	0.4	0.4		

Subdrainage Area: L105B Uncontrolled - Tributary				
Area (ha):	0.10			
C:	0.34			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	7.3	7.3		
20	52.0	4.9	4.9		
30	40.0	3.8	3.8		
40	32.9	3.1	3.1		
50	28.0	2.7	2.7		
60	24.6	2.3	2.3		
70	21.9	2.1	2.1		
80	19.8	1.9	1.9		
90	18.1	1.7	1.7		
100	16.7	1.6	1.6		
110	15.6	1.5	1.5		
120	14.6	1.4	1.4		

Project #PROJECT # 160402150, Riverside South Community Centre and Library
Modified Rational Method Calculations for Storage

100 yr Intensity City of Ottawa	$I = a/(t + b)^c$	a = 1735.688	t (min)	I (mm/hr)
		b = 6.014	10	178.56
		c = 0.820	20	119.95
			30	91.87
			40	75.15
			50	63.95
			60	55.89
			70	49.79
			80	44.99
			90	41.11
			100	37.90
			110	35.20
			120	32.89

100 YEAR Post-Development Target Release from Portion of Site				
Subdrainage Area: Predevelopment Tributary Area to Outlet				
Area (ha):	1.7400			
C:	0.75			
Q:	113.0 L/s			

100 YEAR Modified Rational Method for Entire Site				
Subdrainage Area: UNC-2 Uncontrolled - Non-Tributary				
Area (ha):	0.06			
C:	1.00			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	29.8	29.8		
20	120.0	20.0	20.0		
30	91.9	15.3	15.3		
40	75.1	12.5	12.5		
50	64.0	10.7	10.7		
60	55.9	9.3	9.3		
70	49.8	8.3	8.3		
80	45.0	7.5	7.5		
90	41.1	6.9	6.9		
100	37.9	6.3	6.3		
110	35.2	5.9	5.9		
120	32.9	5.5	5.5		

Subdrainage Area: UNC-1 Uncontrolled - Non-Tributary				
Area (ha):	0.02			
C:	0.69			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	6.8	6.8		
20	120.0	4.6	4.6		
30	91.9	3.5	3.5		
40	75.1	2.9	2.9		
50	64.0	2.4	2.4		
60	55.9	2.1	2.1		
70	49.8	1.9	1.9		
80	45.0	1.7	1.7		
90	41.1	1.6	1.6		
100	37.9	1.4	1.4		
110	35.2	1.3	1.3		
120	32.9	1.3	1.3		

Subdrainage Area: L105B Uncontrolled - Tributary				
Area (ha):	0.10			
C:	0.43			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	21.1	21.1		
20	120.0	14.2	14.2		
30	91.9	10.9	10.9		
40	75.1	8.9	8.9		
50	64.0	7.6	7.6		
60	55.9	6.6	6.6		
70	49.8	5.9	5.9		
80	45.0	5.3	5.3		
90	41.1	4.9	4.9		
100	37.9	4.5	4.5		
110	35.2	4.2	4.2		
120	32.9	3.9	3.9		

Stormwater Management Calculations

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L105A		Controlled - Tributary				
Area (ha): 0.11						
C: 0.90						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	21.1	20.7	0.4	0.2	
20	52.0	14.3	14.3	0.0	0.0	
30	40.0	11.0	11.0	0.0	0.0	
40	32.9	9.0	9.0	0.0	0.0	
50	28.0	7.7	7.7	0.0	0.0	
60	24.6	6.8	6.8	0.0	0.0	
70	21.9	6.0	6.0	0.0	0.0	
80	19.8	5.5	5.5	0.0	0.0	
90	18.1	5.0	5.0	0.0	0.0	
100	16.7	4.6	4.6	0.0	0.0	
110	15.6	4.3	4.3	0.0	0.0	
120	14.6	4.0	4.0	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 95 mm				Vol within CB = 0.5m³		
Invert Elevation: 91.67 m						
T/G Elevation: 93.05 m						
Max Ponding Depth: 0.00 m						
Downstream W/L: 91.33 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	93.05	1.33	20.7	0.2	33.1	OK

Subdrainage Area: L200A		Controlled - Tributary				
Area (ha): 0.07						
C: 0.90						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	13.5	13.5	0.0	0.0	
20	52.0	9.1	9.1	0.0	0.0	
30	40.0	7.0	7.0	0.0	0.0	
40	32.9	5.8	5.8	0.0	0.0	
50	28.0	4.9	4.9	0.0	0.0	
60	24.6	4.3	4.3	0.0	0.0	
70	21.9	3.8	3.8	0.0	0.0	
80	19.8	3.5	3.5	0.0	0.0	
90	18.1	3.2	3.2	0.0	0.0	
100	16.7	2.9	2.9	0.0	0.0	
110	15.6	2.7	2.7	0.0	0.0	
120	14.6	2.6	2.6	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 102 mm						
Invert Elevation: 91.68 m						
T/G Elevation: 93.06 m						
Max Ponding Depth: 0.00 m						
Downstream W/L: 0.00 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	92.16	0.42	13.5	0.0	7.7	OK

Subdrainage Area: L202A		Controlled - Tributary				
Area (ha): 0.07						
C: 0.85						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	12.7	12.7	0.0	0.0	
20	52.0	8.6	8.6	0.0	0.0	
30	40.0	6.6	6.6	0.0	0.0	
40	32.9	5.4	5.4	0.0	0.0	
50	28.0	4.6	4.6	0.0	0.0	
60	24.6	4.1	4.1	0.0	0.0	
70	21.9	3.6	3.6	0.0	0.0	
80	19.8	3.3	3.3	0.0	0.0	
90	18.1	3.0	3.0	0.0	0.0	
100	16.7	2.8	2.8	0.0	0.0	
110	15.6	2.6	2.6	0.0	0.0	
120	14.6	2.4	2.4	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 95 mm						
Invert Elevation: 91.61 m						
T/G Elevation: 92.99 m						
Max Ponding Depth: 0.00 m						
Downstream W/L: 91.33 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	92.16	0.50	12.7	0.0	9.8	OK

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L105A		Controlled - Tributary				
Area (ha): 0.11						
C: 1.00						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	54.6	22.4	32.2	19.3	
20	120.0	36.7	22.4	14.2	17.1	
30	91.9	28.1	22.4	5.6	10.2	
40	75.1	23.0	22.4	0.5	1.3	
50	64.0	19.6	19.6	0.0	0.0	
60	55.9	17.1	17.1	0.0	0.0	
70	49.8	15.2	15.2	0.0	0.0	
80	45.0	13.8	13.8	0.0	0.0	
90	41.1	12.6	12.6	0.0	0.0	
100	37.9	11.6	11.6	0.0	0.0	
110	35.2	10.8	10.8	0.0	0.0	
120	32.9	10.1	10.1	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 95 mm						
Invert Elevation: 91.67 m						
T/G Elevation: 93.05 m						
Max Ponding Depth: 0.23 m						
Downstream W/L: 91.33 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.28	1.56	22.4	19.3	33.1	OK
13.81						

Subdrainage Area: L200A		Controlled - Tributary				
Area (ha): 0.07						
C: 1.00						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	34.7	24.9	9.8	5.9	
20	120.0	23.3	23.3	0.0	0.0	
30	91.9	17.9	17.9	0.0	0.0	
40	75.1	14.6	14.6	0.0	0.0	
50	64.0	12.4	12.4	0.0	0.0	
60	55.9	10.9	10.9	0.0	0.0	
70	49.8	9.7	9.7	0.0	0.0	
80	45.0	8.8	8.8	0.0	0.0	
90	41.1	8.0	8.0	0.0	0.0	
100	37.9	7.4	7.4	0.0	0.0	
110	35.2	6.9	6.9	0.0	0.0	
120	32.9	6.4	6.4	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 102 mm						
Invert Elevation: 91.68 m						
T/G Elevation: 93.06 m						
Max Ponding Depth: 0.12 m						
Downstream W/L: 0.00 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.18	1.45	24.9	5.9	7.7	OK
1.80						

Subdrainage Area: L202A		Controlled - Tributary				
Area (ha): 0.07						
C: 1.00						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	34.7	22.9	11.8	7.1	
20	120.0	23.3	22.9	0.4	0.5	
30	91.9	17.9	17.9	0.0	0.0	
40	75.1	14.6	14.6	0.0	0.0	
50	64.0	12.4	12.4	0.0	0.0	
60	55.9	10.9	10.9	0.0	0.0	
70	49.8	9.7	9.7	0.0	0.0	
80	45.0	8.8	8.8	0.0	0.0	
90	41.1	8.0	8.0	0.0	0.0	
100	37.9	7.4	7.4	0.0	0.0	
110	35.2	6.9	6.9	0.0	0.0	
120	32.9	6.4	6.4	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C = 0.57				
Orifice Diameter: 95 mm						
Invert Elevation: 91.61 m						
T/G Elevation: 92.99 m						
Max Ponding Depth: 0.30 m						
Downstream W/L: 91.33 m						
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.29	1.63	22.9	7.1	9.8	OK
2.72						

Stormwater Management Calculations

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L201A		Controlled - Tributary				
Area (ha): 0.07						
C: 0.90						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	13.5	13.5	0.0	0.0	
20	52.0	9.1	9.1	0.0	0.0	
30	40.0	7.0	7.0	0.0	0.0	
40	32.9	5.8	5.8	0.0	0.0	
50	28.0	4.9	4.9	0.0	0.0	
60	24.6	4.3	4.3	0.0	0.0	
70	21.9	3.8	3.8	0.0	0.0	
80	19.8	3.5	3.5	0.0	0.0	
90	18.1	3.2	3.2	0.0	0.0	
100	16.7	2.9	2.9	0.0	0.0	
110	15.6	2.7	2.7	0.0	0.0	
120	14.6	2.6	2.6	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		95 mm				
Invert Elevation		91.70 m				
T/G Elevation		93.08 m				
Max Ponding Depth		0.00 m				
Downstream W/L		91.33 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	92.31	0.56	13.5	0.0	10.4	OK

Subdrainage Area: L103A		Controlled - Tributary				
Area (ha): 0.14						
C: 0.75						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	22.4	22.4	0.0	0.0	
20	52.0	15.2	15.2	0.0	0.0	
30	40.0	11.7	11.7	0.0	0.0	
40	32.9	9.6	9.6	0.0	0.0	
50	28.0	8.2	8.2	0.0	0.0	
60	24.6	7.2	7.2	0.0	0.0	
70	21.9	6.4	6.4	0.0	0.0	
80	19.8	5.8	5.8	0.0	0.0	
90	18.1	5.3	5.3	0.0	0.0	
100	16.7	4.9	4.9	0.0	0.0	
110	15.6	4.5	4.5	0.0	0.0	
120	14.6	4.3	4.3	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		127 mm				
Invert Elevation		91.62 m				
T/G Elevation		93.00 m				
Max Ponding Depth		0.00 m				
Downstream W/L		91.33 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	92.17	0.49	22.4	0.0	18.0	OK

Subdrainage Area: L107C		Controlled - Tributary				
Area (ha): 0.13						
C: 0.26						
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	76.8	7.2	7.2	0.0	0.0	
20	52.0	4.9	4.9	0.0	0.0	
30	40.0	3.8	3.8	0.0	0.0	
40	32.9	3.1	3.1	0.0	0.0	
50	28.0	2.6	2.6	0.0	0.0	
60	24.6	2.3	2.3	0.0	0.0	
70	21.9	2.1	2.1	0.0	0.0	
80	19.8	1.9	1.9	0.0	0.0	
90	18.1	1.7	1.7	0.0	0.0	
100	16.7	1.6	1.6	0.0	0.0	
110	15.6	1.5	1.5	0.0	0.0	
120	14.6	1.4	1.4	0.0	0.0	
Storage: Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		83 mm				
Invert Elevation		91.72 m				
T/G Elevation		93.10 m				
Max Ponding Depth		0.00 m				
Downstream W/L		0.00 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
5-year Water Level	92.04	0.28	7.2	0.0	60.0	OK

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L201A		Controlled - Tributary				
Area (ha): 0.07						
C: 1.00						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	34.7	22.0	12.7	7.6	
20	120.0	23.3	22.0	1.3	1.6	
30	91.9	17.9	17.9	0.0	0.0	
40	75.1	14.6	14.6	0.0	0.0	
50	64.0	12.4	12.4	0.0	0.0	
60	55.9	10.9	10.9	0.0	0.0	
70	49.8	9.7	9.7	0.0	0.0	
80	45.0	8.8	8.8	0.0	0.0	
90	41.1	8.0	8.0	0.0	0.0	
100	37.9	7.4	7.4	0.0	0.0	
110	35.2	6.9	6.9	0.0	0.0	
120	32.9	6.4	6.4	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		95 mm				
Invert Elevation		91.70 m				
T/G Elevation		93.08 m				
Max Ponding Depth		0.17 m				
Downstream W/L		91.33 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.25	1.50	22.0	7.6	10.4	OK
2.76						

Subdrainage Area: L103A		Controlled - Tributary				
Area (ha): 0.14						
C: 0.94						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	65.2	39.5	25.6	15.4	
20	120.0	43.8	39.5	4.2	5.1	
30	91.9	33.5	33.5	0.0	0.0	
40	75.1	27.4	27.4	0.0	0.0	
50	64.0	23.3	23.3	0.0	0.0	
60	55.9	20.4	20.4	0.0	0.0	
70	49.8	18.2	18.2	0.0	0.0	
80	45.0	16.4	16.4	0.0	0.0	
90	41.1	15.0	15.0	0.0	0.0	
100	37.9	13.8	13.8	0.0	0.0	
110	35.2	12.8	12.8	0.0	0.0	
120	32.9	12.0	12.0	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		127 mm				
Invert Elevation		91.62 m				
T/G Elevation		93.00 m				
Max Ponding Depth		0.20 m				
Downstream W/L		91.33 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.20	1.52	39.5	15.4	18.0	OK
2.62						

Subdrainage Area: L107C		Controlled - Tributary				
Area (ha): 0.13						
C: 0.33						
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	
10	178.6	21.0	17.5	3.4	2.1	
20	120.0	14.1	14.1	0.0	0.0	
30	91.9	10.8	10.8	0.0	0.0	
40	75.1	8.8	8.8	0.0	0.0	
50	64.0	7.5	7.5	0.0	0.0	
60	55.9	6.6	6.6	0.0	0.0	
70	49.8	5.8	5.8	0.0	0.0	
80	45.0	5.3	5.3	0.0	0.0	
90	41.1	4.8	4.8	0.0	0.0	
100	37.9	4.5	4.5	0.0	0.0	
110	35.2	4.1	4.1	0.0	0.0	
120	32.9	3.9	3.9	0.0	0.0	
Storage: Surface Storage Above CB						
Orifice Equation: $Q = CdA(2gh)^{0.5}$		Where C =		0.57		
Orifice Diameter:		83 mm				
Invert Elevation		91.72 m				
T/G Elevation		93.10 m				
Max Ponding Depth		0.30 m				
Downstream W/L		0.00 m				
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check	
100-year Water Level	93.40	1.64	17.5	2.1	60.0	OK
57.94						

Stormwater Management Calculations

Project #PROJECT # 160402150, Riverside South Community Centre and Library
Modified Rational Method Calculations for Storage

Subdrainage Area: L107A Uncontrolled - Tributary					
Area (ha): 0.48					
C: 0.49					
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	50.2	50.2		
20	52.0	34.0	34.0		
30	40.0	26.2	26.2		
40	32.9	21.5	21.5		
50	28.0	18.3	18.3		
60	24.6	16.1	16.1		
70	21.9	14.3	14.3		
80	19.8	13.0	13.0		
90	18.1	11.9	11.9		
100	16.7	10.9	10.9		
110	15.6	10.2	10.2		
120	14.6	9.5	9.5		

Subdrainage Area: L104C Uncontrolled - Tributary					
Area (ha): 0.20					
C: 0.90					
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	38.4	38.4		
20	52.0	26.0	26.0		
30	40.0	20.0	20.0		
40	32.9	16.4	16.4		
50	28.0	14.0	14.0		
60	24.6	12.3	12.3		
70	21.9	11.0	11.0		
80	19.8	9.9	9.9		
90	18.1	9.1	9.1		
100	16.7	8.4	8.4		
110	15.6	7.8	7.8		
120	14.6	7.3	7.3		

Subdrainage Area: L104B Uncontrolled - Tributary					
Area (ha): 0.19					
C: 0.90					
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	36.5	36.5		
20	52.0	24.7	24.7		
30	40.0	19.0	19.0		
40	32.9	15.6	15.6		
50	28.0	13.3	13.3		
60	24.6	11.7	11.7		
70	21.9	10.4	10.4		
80	19.8	9.4	9.4		
90	18.1	8.6	8.6		
100	16.7	8.0	8.0		
110	15.6	7.4	7.4		
120	14.6	6.9	6.9		

Subdrainage Area: TANK Controlled - Tributary					
Area (ha): 0.00					
C: 0.00 *Includes runoff from all upstream tributary areas					
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	238.3	60.0	178.3	107.0
20	52.0	162.2	60.0	102.2	122.7
30	40.0	124.8	60.0	64.8	116.7
40	32.9	102.5	60.0	42.5	101.9
50	28.0	87.4	60.0	27.4	82.3
60	24.6	76.6	60.0	16.6	59.6
70	21.9	68.3	60.0	8.3	34.9
80	19.8	61.8	60.0	1.8	8.7
90	18.1	56.6	56.6	0.0	0.0
100	16.7	52.2	52.2	0.0	0.0
110	15.6	48.5	48.5	0.0	0.0
120	14.6	45.4	45.4	0.0	0.0

Storage: Underground Tank

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.61

Orifice Diameter: 200 mm

Invert Elevation: 89.51 m

T/G Elevation: 93.37 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	90.11	0.50	60.0	122.7	OK

Project #PROJECT # 160402150, Riverside South Community Centre and Library
Modified Rational Method Calculations for Storage

Subdrainage Area: L107A Uncontrolled - Tributary					
Area (ha): 0.48					
C: 0.61					
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	145.9	145.9		
20	120.0	98.0	98.0		
30	91.9	75.1	75.1		
40	75.1	61.4	61.4		
50	64.0	52.3	52.3		
60	55.9	45.7	45.7		
70	49.8	40.7	40.7		
80	45.0	36.8	36.8		
90	41.1	33.6	33.6		
100	37.9	31.0	31.0		
110	35.2	28.8	28.8		
120	32.9	26.9	26.9		

Subdrainage Area: L104C Uncontrolled - Tributary					
Area (ha): 0.20					
C: 1.00					
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	99.3	99.3		
20	120.0	66.7	66.7		
30	91.9	51.1	51.1		
40	75.1	41.8	41.8		
50	64.0	35.6	35.6		
60	55.9	31.1	31.1		
70	49.8	27.7	27.7		
80	45.0	25.0	25.0		
90	41.1	22.9	22.9		
100	37.9	21.1	21.1		
110	35.2	19.6	19.6		
120	32.9	18.3	18.3		

Subdrainage Area: L104B Uncontrolled - Tributary					
Area (ha): 0.19					
C: 1.00					
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	94.3	94.3		
20	120.0	63.4	63.4		
30	91.9	48.5	48.5		
40	75.1	39.7	39.7		
50	64.0	33.8	33.8		
60	55.9	29.5	29.5		
70	49.8	26.3	26.3		
80	45.0	23.8	23.8		
90	41.1	21.7	21.7		
100	37.9	20.0	20.0		
110	35.2	18.6	18.6		
120	32.9	17.4	17.4		

Subdrainage Area: TANK Controlled - Tributary					
Area (ha): 0.00					
C: 0.00 *Includes runoff from all upstream tributary areas					
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	527.3	111.3	416.0	249.6
20	120.0	403.9	111.3	292.6	351.1
30	91.9	323.2	111.3	211.9	381.4
40	75.1	271.6	111.3	160.3	384.7
50	64.0	234.2	111.3	122.9	368.6
60	55.9	204.7	111.3	93.4	336.2
70	49.8	182.4	111.3	71.0	298.4
80	45.0	164.8	111.3	53.5	256.6
90	41.1	150.6	111.3	39.2	211.9
100	37.9	138.8	111.3	27.5	165.0
110	35.2	128.9	111.3	17.6	116.2
120	32.9	120.5	111.3	9.2	65.9

Storage: Underground Tank

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.61

Orifice Diameter: 200 mm

Invert Elevation: 89.51 m

T/G Elevation: 93.37 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	91.33	1.72	111.3	384.7	OK

Stormwater Management Calculations

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L104A		Controlled - Tributary	
Area (ha): 0.10			
C: 0.78			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	16.7	15.9	0.8	0.5
20	52.0	11.3	11.3	0.0	0.0
30	40.0	8.7	8.7	0.0	0.0
40	32.9	7.1	7.1	0.0	0.0
50	28.0	6.1	6.1	0.0	0.0
60	24.6	5.3	5.3	0.0	0.0
70	21.9	4.8	4.8	0.0	0.0
80	19.8	4.3	4.3	0.0	0.0
90	18.1	3.9	3.9	0.0	0.0
100	16.7	3.6	3.6	0.0	0.0
110	15.6	3.4	3.4	0.0	0.0
120	14.6	3.2	3.2	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.57
 Orifice Diameter: 83 mm
 Invert Elevation: 91.64 m
 T/G Elevation: 93.02 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Vol within CB = 0.5m³

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	93.02	1.34	15.9	0.5	25.1 OK

SUMMARY TO OUTLET					
		Vrequired	Vavailable*		
Tributary Area	1.660 ha				
Total 2yr Flow to Sewer	60.0 L/s	123	287 m³		
Non-Tributary Area	0.080 ha				
Total 2yr Flow Uncontrolled	13.2 L/s				
Total Area	1.740 ha				
Total 2yr Flow	60.0 L/s				
Target	113.0 L/s				

Project #PROJECT # 160402150, Riverside South Community Centre and Library Modified Rational Method Calculations for Storage

Subdrainage Area: L104A		Controlled - Tributary	
Area (ha): 0.10			
C: 0.98			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	48.4	17.3	31.1	18.7
20	120.0	32.5	17.3	15.2	18.3
30	91.9	24.9	17.3	7.6	13.7
40	75.1	20.4	17.3	3.1	7.4
50	64.0	17.3	17.3	0.1	0.2
60	55.9	15.2	15.2	0.0	0.0
70	49.8	13.5	13.5	0.0	0.0
80	45.0	12.2	12.2	0.0	0.0
90	41.1	11.1	11.1	0.0	0.0
100	37.9	10.3	10.3	0.0	0.0
110	35.2	9.5	9.5	0.0	0.0
120	32.9	8.9	8.9	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.57
 Orifice Diameter: 83 mm
 Invert Elevation: 91.64 m
 T/G Elevation: 93.02 m
 Max Ponding Depth: 0.25 m
 Downstream W/L: 91.33 m

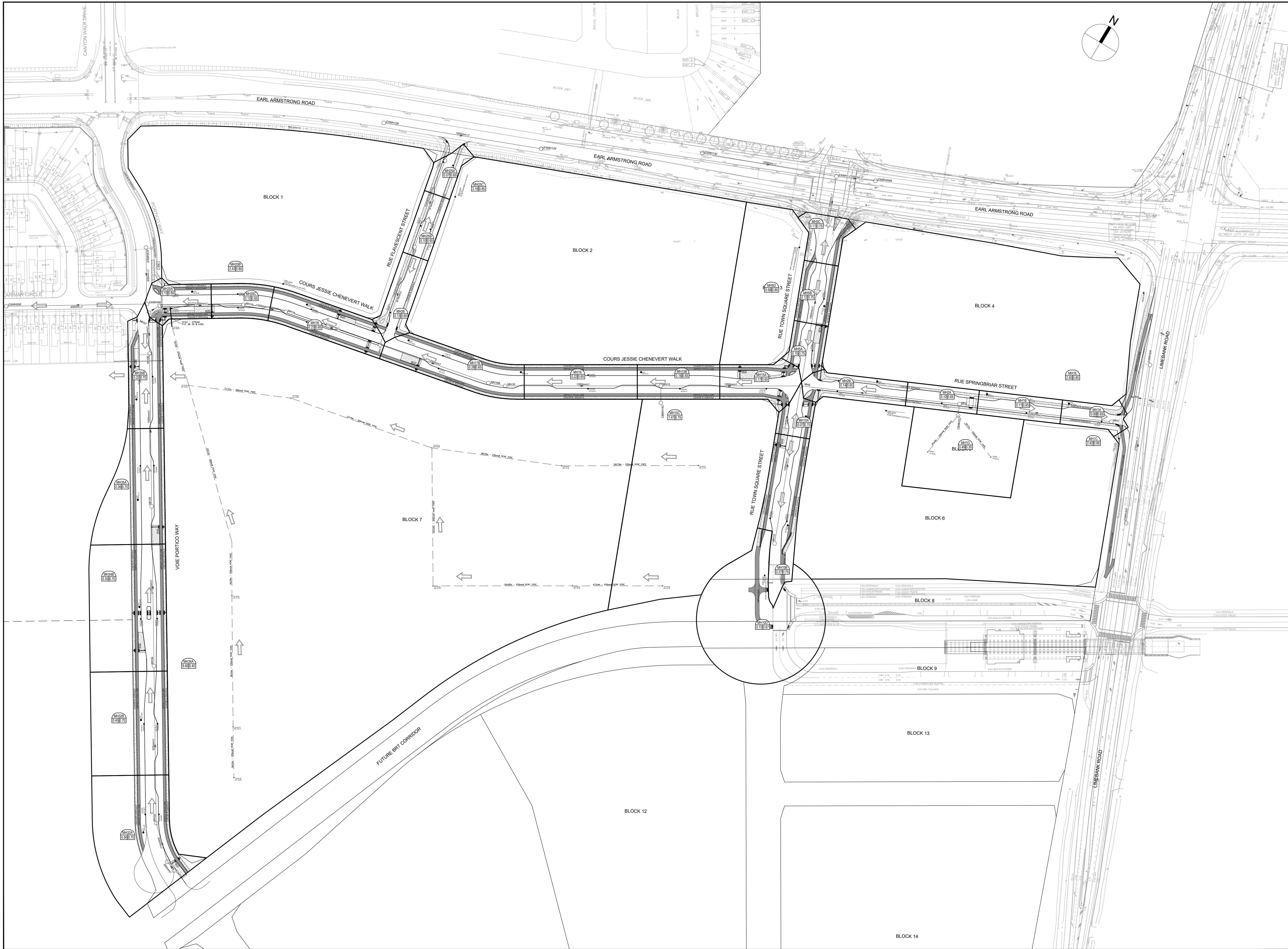
Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	93.27	1.59	17.3	18.7	25.1 OK

6.43

SUMMARY TO OUTLET					
		Vrequired	Vavailable*		
Tributary Area	1.660 ha				
Total 100yr Flow to Sewer	111.3 L/s	461	564 m³		
Non-Tributary Area	0.080 ha				
Total 100yr Flow Uncontrolled	36.6 L/s				
Total Area	1.740 ha				
Total 100yr Flow	111.3 L/s				
Target	113.0 L/s				

C.3 Storm Design Background Report Excerpts





CLIENT

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Arcadis Professional Services (Canada) Inc.
formerly B|G Group Professional Services (Canada) Inc.

ISSUES

No.	DESCRIPTION	DATE
1	SUBMISSION 1 FOR CITY REVIEW	2024-07-19
2	SUBMISSION 2 FOR CITY REVIEW	2024-10-29
3	SUBMISSION 3 FOR CITY REVIEW	2025-01-17

SEE 010, 011 FOR NOTES, LEGEND, CB TABLE, STREET SECTIONS AND DETAILS

KEY PLAN

1:1250

SEAL

PRIME CONSULTANT

333 Preston Street - Suite 500
Ottawa ON K1S 5N4 Canada
tel 613 225 1311
www.arcadis.com

PROJECT

RSS TOWN CENTER
980 EARL ARMSTRONG & 4700
LIMEBANK ROAD
PHASE 7A

PROJECT NO:
144320

DRAWN BY:
C.C

CHECKED BY:
L.E.

PROJECT MGR:
L.E.

APPROVED BY:
L.E.

SHEET TITLE

STORM DRAINAGE AREA PLAN

SHEET NUMBER

500

ISSUE

3

LOCATION				AREA (Ha)										RATIONAL DESIGN FLOW																				SEWER DATA																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
STREET	AREA ID	FROM	TO	2 YEAR										5 YEAR			100 YEAR		2 YEAR		5 YEAR		100 YEAR		INLET (min)	TIME IN PIPE	TOTAL (min)	i (2) (mm/hr)	i (5) (mm/hr)	i (10) (mm/hr)	i (100) (mm/hr)	2yr PEAK FLOW (L/s)	5yr PEAK FLOW (L/s)	10yr PEAK FLOW (L/s)	100yr PEAK FLOW (L/s)	FIXED FLOW		DESIGN FLOW (L/s)	CAPACITY (L/s)	LENGTH (m)	PIPE SIZE (mm)			SLOPE (%)	VELOCITY (m/s)	AVAIL CAP (2yr)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
				C=0.20	C=0.30	C=0.40	C=0.65	C=0.70	C=0.75	C=0.85	C=0.90	C=0.65	C=0.70	C=0.75	C=0.67	C=0.75	IND 2.78AC	CUM 2.78AC	IND 2.78AC	CUM 2.78AC	IND 2.78AC	CUM 2.78AC	IND	CUM												DIA	W				H	(L/s)	(%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
Portico Way		CAP30	MH31												0.00	0.00	0.00	0.00					10.00	0.17	10.17	76.81	104.19	122.14	178.56	0.00	0.00	0.00	0.00	0.00	0.00	59.68	8.25	300			0.35	0.818	59.68	100.00%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		MH31	MH32												0.00	0.00	0.00	0.00					10.17	0.61	10.78	167.04	103.31	121.10	177.04	0.00	0.00	0.00	0.00	0.00	0.00	59.68	30.13	300			0.35	0.818	59.68	100.00%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		MH32A&B	MH32	MH34											0.00	0.00	1.97	1.97					10.78	1.96	12.74	73.93	100.24	117.48	171.71	0.00	197.01	0.00	0.00	0.00	197.01	224.33	118.12	525			0.25	1.004	27.32	12.18%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		MH34A&B	MH34	MH35											10.45	10.45	1.07	3.04					12.74	1.62	14.36	67.66	91.64	107.36	156.85	707.27	178.19	0.00	0.00	0.00	985.46	1,103.33	120.00	1050			0.15	1.234	117.87	10.68%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		MH35A,B&C	MH35	MH36											0.00	10.45	1.13	4.16					14.36	1.55	15.91	63.31	85.67	100.34	146.54	661.78	356.77	0.00	0.00	0.00	1,018.56	1,103.33	114.85	1050			0.15	1.234	84.78	7.68%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
			MH36	MH37											0.00	10.45	0.00	4.16					15.91	0.47	16.38	59.69	80.72	94.51	137.99	623.93	336.13	0.00	0.00	0.00	960.07	1,286.19	31.10	1200			0.10	1.102	326.13	25.36%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
			MH37	MH20											0.00	10.45	0.00	4.16					16.38	0.14	16.52	58.68	79.33	92.89	135.61	613.38	330.38	0.00	0.00	0.00	943.77	1,286.19	9.16	1200			0.10	1.102	342.43	26.62%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
Jessie Chenevert Walk		MH19	MH18												5.74	16.19	0.45	4.62					16.52	0.80	17.33	58.39	78.94	92.42	134.92	945.64	364.39	0.00	0.00	0.00	1,310.03	1,928.87	63.00	1350			0.12	1.305	618.84	32.08%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		MH18	MH17												0.00	16.19	0.00	4.62					17.33	0.41	17.74	56.77	76.72	89.81	131.10	919.37	354.16	0.00	0.00	0.00	1,273.52	1,928.87	32.43	1350			0.12	1.305	655.35	33.98%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
															0.00	16.19	0.42	5.03					17.74	0.96	18.70	55.97	75.63	88.53	129.22	906.47	380.57	0.00	0.00	0.00	1,287.03	1,928.87	75.16	1350			0.12	1.305	641.84	33.28%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
Flavescent Street															7.77	7.77							10.00	1.19	11.19	76.81	104.19	122.14	178.56	597.10	0.00	0.00	0.00	0.00	0.00	622.60	739.33	68.64	975			0.10	0.959	116.72	15.79%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
		MH26	MH26	MH17											0.23	1.36	12.55						11.19	1.36	12.55	72.51	98.29	115.19	168.35	580.75	0.00	0.00	0.00	0.00	0.00	622.60	739.33	78.00	975			0.10	0.959	116.72	15.79%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
Jessie Chenevert Walk															0.00	24.20		0.51	5.54					18.70	1.10	19.80	54.22	73.24	85.72	125.10	1,312.32	405.57	0.00	0.00	0.00	0.00	0.00	1,717.88	1,928.87	86.35	1350			0.12	1.305	210.99	10.94%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
		MH16B	MH16	MH16											0.00	24.20	0.00	5.54					19.80	0.16	19.97	52.35	70.68	82.72	120.70	1,267.06	391.44	0.00	0.00	0.00	0.00	0.00	1,658.50	1,928.87	12.85	1350			0.12	1.305	270.37	14.02%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
		MH16	MH16	MH15											0.00	24.20	0.40	5.94					19.97	1.49	21.46	52.08	70.32	82.29	120.07	1,260.63	417.38	0.00	0.00	0.00	0.00	0.00	1,678.01	1,928.87	117.00	1350			0.12	1.305	250.87	13.01%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
		MH15A,B&C	MH15	MH5											3.06	27.27		0.60	6.53					21.46	1.34	22.80	49.79	67.20	78.62	114.69	1,357.81	438.91	0.00	0.00	0.00	0.00	0.00	1,796.72	1,928.87	105.20	1350			0.12	1.305	132.15	6.85%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
Springbriar Street															12.40	12.40							10.00	1.55	11.55	76.81	104.19	122.14	178.56	952.08	0.00	0.00	0.00	0.00	0.00	952.08	1,103.33	114.99	1050			0.15	1.234	151.26	13.71%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
		MH2A&B	MH2	MH5											0.43	12.83							11.55	1.62	13.17	71.32	96.65	113.26	165.51	914.97	0.00	0.00	0.00	0.00	0.00	914.97	1,103.33	120.00	1050			0.15	1.234	188.36	17.07%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
Town Square Street															0.00	0.00	0.15	6.68	1.90	1.90			10.00	1.82	11.82	76.81	104.19	122.14	178.56	0.00	695.76	0.00	0.00	339.98	0.00	0.00	1,035.74	1,286.19	120.00	1200			0.10	1.102	250.46	19.47%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
Town Square Street															1.37	41.47	0.69	7.37	0.00	1.90			22.80	0.77	23.58	47.92	64.64	75.62	110.29	1,987.08	476.12	0.00	210.00	0.00	0.00	2,673.20	8,130.98	122.53	1950			0.30	2.638	5457.77	67.12%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
															0.00	41.47	0.00	7.37	0.00	1.90			23.58	0.40	23.98	46.91	63.26	74.00	107.92	1,945.13	465.97	0.00	205.48	0.00	0.00	2,616.58	4,694.42	37.00	1950			0.10	1.523	2077.84	44.26%																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																

Design Brief Phase 7A

980 Earl Armstrong Road & 4700 Limebank Road, Riverside South

DRAINAGE AREA ID	CONTINUOUS/ SAG	ROAD TYPE	MINOR SYSTEM DESIGN TARGET (BASED ON ROAD TYPE)		100 YEAR CAPTURED FLOW (L/S) (3 HOUR CHICAGO STORM)	ICD ORIFICE SIZE (MM DIA.) (TWO ICDs PER DRAINAGE AREA)	
			MINOR SYSTEM DESIGN STORM	GENERATED FLOW ON INDIVIDUAL SEGMENT SIMULATED (L/S)			
S2B-Ph7A	Sag	26mROW-9.4mAsphalt	5	26.9	33.7	83	83
S32A-Ph7A	Sag	26mROW-9.4mAsphalt	5	127.0	141.6	178	152
S32B-Ph7A	Sag	26mROW-9.4mAsphalt	5	102.8	119.3	152	152
S20C-Ph7A	Sag	26mROW-9.4mAsphalt	5	21.8	33.8	83	83
S34B-Ph7A	Sag	26mROW-9.4mAsphalt	5	125.6	143.4	178	152
S35A-Ph7A	Sag	26mROW-9.4mAsphalt	5	82.8	134.0	178	152
S35B-Ph7A	Sag	26mROW-9.4mAsphalt	5	50.1	51.6	102	102
S5A-Ph7A	Sag	26mROW-12mAsphalt	2	16.2	33.5	83	83
S5B-Ph7A	Sag	26mROW-12mAsphalt	2	20.0	34.4	83	83
S5C-Ph7A	Sag	26mROW-12mAsphalt	2	18.3	42.4	94	94
Development Blocks							
B20B-Ph7A	Block	Rear Yard	2	478.8	479.0	n/a	
B25C-Ph7A	Block	Rear Yard	2	621.7	622.0	n/a	
B1C-Ph7A	Block	Rear Yard	2	476.2	477.0	n/a	
B1D-Ph7A	Block	Rear Yard	2	15.4	16.0	n/a	
B1E-Ph7A	Block	Rear Yard	2	492.6	493.0	n/a	
B15C-Ph7A	Block	Rear Yard	2	113.0	113.0	n/a	
B34A-Ph7A	Block	Rear Yard	2	578.7	579.0	n/a	
B5D-Ph7A	Block	Rear Yard	2	114.6	115.0	n/a	
EXT-Ph7A	External	External	100	307.4	307	n/a	

The available on-site storage and the results of the PCSWMM evaluation for Town Center Phase 7A are presented in **Table 4-**. The ponding plan is presented on **Drawing 144320-200** to **Drawing 144320-202**.

Appendix D Geotechnical Report Excerpts



3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out between April 2 to April 8, 2024, and consisted of advancing a total of 19 boreholes to a maximum depth of 11.3 m below the existing ground surface. In addition, a number of previous investigations have been carried out by Paterson and others within the subject site and on adjacent sites in the last several years.

The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations from the current investigation and relevant boreholes from previous investigations are shown on Drawing PG4958-19 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a track-mounted power auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The testing procedure consisted of augering to the required depth at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sampler. All samples were visually inspected and initially classified on site, placed in sealed plastic bags, and transported to our laboratory for further examination and classification. The depths at which the auger and split-spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at boreholes BH 3-24, BH 11-24, and BH 18-24. The DCPT testing consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Boreholes BH 1-24, BH 7-24, BH 8-24, BH 13-24, and BH 17-24 were fitted with 51 mm diameter PVC groundwater monitoring wells. The other boreholes were fitted with flexible piezometers to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ☐ 1.5 m or less, as needed depending on the soil profile, of slotted 51 mm diameter PVC screen at specified intervals within the borehole column.
- ☐ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ☐ No. 3 silica sand backfill within annular space around the screen.
- ☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
- ☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

3.2 Field Survey

The borehole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG4958-19 - Test Hole Location Plan in Appendix 2.

4.0 Observations

4.1 Surface Conditions

The subject site is located on the south-west quadrant of the intersection of Limebank Road and Earl Armstrong Road. The site is currently largely undeveloped and predominately consists of agricultural land with some tree covered areas and an asphalt access lane located on the north-east side of the site. Construction on the future O-Train network extension is currently underway running east-west through the central portion of the site.

The subject site is bordered by Earl Armstrong Road followed by a residential development and a currently under construction commercial development to the north. Further, the site is bordered by Limebank Road followed by agricultural lands to the east, agricultural lands to the south, and by a densely treed area followed by a residential development to the west. The ground surface across the subject site gradually slopes down towards the north at approximate geodetic elevations of 94.0 to 91.5 m. The site is located below grade of surrounding roadways.

4.2 Subsurface Profile

Overburden

The subsurface profile at the test hole locations consists of a thin topsoil layer underlain by alternating layers of a loose to compact silty sand to sandy silt, silty clay, and clayey silt. Generally, the soil profile consists of an approximately 0.7 to 2.4 m thick silty sand to sandy silt deposit overlaying a silty clay deposit. In some locations a thin layer of silty clay was encountered directly below the topsoil and overlaying the silty sand layer.

The silty clay deposit was encountered at all test hole locations and generally consisted of a very stiff to stiff brown silty clay crust overlaying a stiff to firm grey silty clay deposit. The presence of the brown silty clay crust layer directly overlaying the grey silty clay was absent at the location of boreholes BH 2-24, BH 5-24, BH 6-24, BH 8-24, and BH 9-24. The silty clay deposit was found to extend as deep as over 11.3 m below the existing ground surface and was generally observed to be underlain by a thin deposit of clayey silt across the majority of the site. At the location of boreholes BH 12-24, BH 12A-24, and BH 12B-24 the silty clay deposit was observed at ground surface and directly overlaid the glacial till deposit at a depth of approximately 2.9 m below the existing ground surface.

A deposit of glacial till was observed at depths of approximately 2.6 to 9.6 m below the existing ground surface in BH 1-24, BH 3-24, BH 6-24, BH 7-24, BH 8-24, BH 12-24, BH 12A-24, and BH 13-24. The glacial till deposit generally consists of compact to dense, grey clayey silt, grey silty sand to sandy silt, or grey silty clay with variable amounts of gravel, cobbles, and boulders.

Practical refusal to augering or DCPT testing was encountered at BH 3-24, BH 11-24, BH 12-24, BH 12A-24, BH 12B-24, and BH 18-24 at depths ranging from 2.9 to 14.3 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the local bedrock consists of interbedded sandstone and dolomite of the March Formation. The overburden thickness is generally anticipated to range between approximately 5 to 15 m.

Atterberg Limits Testing

Atterberg limits testing was completed on select silty clay samples recovered from BH 1- 24, BH 3-24, BH 4-24, BH 5-24, BH 6-24, BH 7-24, BH 8-24, BH 11-24, BH 14-24, BH 15-24, BH 17-24, and BH 18-24. The results of the Atterberg limits tests is presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 4 – Summary of Groundwater Level Readings				
Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-24	93.09	0.43	92.66	April 18, 2024
BH 2-24	92.31	0.08	92.23	
BH 3-24	93.60	0.67	92.93	
BH 4-24	93.12	0.28	92.84	
BH 5-24	92.60	1.03	91.57	
BH 6-24	92.55	1.84	90.71	
BH 7-24	92.31	0.40	91.91	
BH 8-24	92.50	2.18	90.32	
BH 9-24	91.69	0.72	90.97	
BH 10-24	92.32	0.10	92.22	
BH 11-24	92.31	1.05	91.26	
BH 12B-24	93.47	0.30	93.17	
BH 13-24	93.05	1.26	91.79	
BH 14-24	92.32	0.37	91.95	
BH 15-24	92.21	0.41	91.8	
BH 16-24	92.30	1.36	90.94	
BH 17-24	91.89	1.48	90.41	
BH 18-24	91.96	0.35	91.61	
BH 19-24	91.62	0.18	91.44	
Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS and are referenced to a geodetic datum.				

It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations.

The Long-term groundwater levels can also be estimated based on the observed color, consistency, and moisture content of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately **2.5 to 3.5 m** below the existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

A series of well points may be required to control groundwater in-flow for excavations that extend below the water table. It is assumed that the excavations will be carried out within the confines of a fully braced trench box or other acceptable shoring system designed by a qualified engineer to resist the design lateral earth pressures and potential basal heave issues.

Due to the presence of a silty clay deposit, the proposed grading throughout the subject site will be subjected to a permissible grade restriction. Permissible grade raise recommendations are discussed in Subsection 5.3. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Bedrock removal may be required to complete the excavations for basement levels of proposed high-rise buildings. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. All contractors should be prepared for bedrock and oversized boulder removal. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

The above and other considerations are discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in sub-excavation of the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Fill Placement

Fill placed for grading throughout the building footprints of low to mid-rise buildings, or below the floor slab of high-rise buildings should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials should be spread in a maximum of 300 mm thick loose lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in a maximum 300 mm thick loose lift to at least 95% of the material's SPMDD. The placement of subgrade material should be reviewed at the time of placement by Paterson personnel.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system.

Bedrock Removal

If required, bedrock removal can be accomplished by hoe ramming where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing surrounding services, buildings, LRT, and other structures should be addressed. A pre-blast or pre-construction survey of the existing adjacent structures located in the proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries or claims related to the blasting operations. It should be noted that a Vibration Monitoring Control Plan (VMCP) may be required to address vibration monitoring for the LRT.

As a general guideline, peak particle velocities (measured at the structures) should not exceed the below noted vibration limits during the blasting program to reduce the risks of damage to the existing surrounding structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using near vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing of the overburden. The 1 m horizontal ledge setback can be eliminated with a shoring program which has drilled piles extending below the proposed founding elevation.

Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated into the construction operations to maintain, as much as possible, a cooperative environment with the surrounding residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 20 mm/s for frequencies equal to or less than 40 Hz, and below 50 mm/s above a frequency of 40 Hz. These guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to be completed to minimize the risks of claims during or following the construction of the proposed buildings. It should be noted that lower peak particle velocity limits may be required for construction activities near the LRT.

Depending on the looseness and degree of saturation at the time of construction, other measures (additional compaction, dewatering, mud-slab, sub-excavation and reinstatement of crushed stone fill) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of construction by Paterson on a footing-specific basis.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction is recommended. The recommended grade raise restrictions are shown on Drawing PG4958-20 – Permissible Grade Raise Plan included in Appendix 2. A post-development groundwater lowering of 0.5 m was considered in our permissible grade raise calculations.

It should be noted that a previous test fill pile program was carried out at the site from December 2019 to December 2020. The program consisted of two 30 x 30 m pads each containing two settlement plates, the height of the fill pads ranged from approximately 2.1 to 2.4 m above the existing ground surface. The location of the test fill pads is shown on drawing PG4958-19 – Test Hole Location Plan and the results are presented in Figure 2 – Test Fill Pile Settlement Monitoring Program Area B in Appendix 2.

If greater permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D** for foundations founded upon a silty sand or silty clay bearing medium and as **Class C** for foundations founded upon a glacial till or bedrock bearing medium for foundations considered at the subject site.

Higher site classes such as Class A or Class B may be provided for buildings founded upon or within 3 m of the bedrock surface. However, they would have to be confirmed by site specific shear wave velocity testing. Such testing may be considered once more detailed plans are available for the proposed development. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.7 Pavement Design

For design purposes, the pavement structures presented in the following tables could be used for the design of driveways, local residential streets, and roadways with bus traffic.

It should be noted that for residential driveways and car only parking areas, an Ontario Traffic Category A is applicable. For local roadways, an Ontario Traffic Category B should be used for design purposes.

Table 7 - Recommended Pavement Structure - Driveways	
Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in-situ soil, or OPSS Granular B Type I or II material over in-situ soil.	

Table 8 - Recommended Pavement Structure - Local Residential Roadways	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Wear Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
400	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in-situ soil, or OPSS Granular B Type I or II material over in-situ soil.	

Table 9 - Recommended Pavement Structure - Arterial Roadways with Bus Traffic	
Thickness (mm)	Material Description
40	Wear Course - Superpave 12.5 Asphaltic Concrete
50	Upper Binder Course - Superpave 19.0 Asphaltic Concrete
50	Lower Binder Course - Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
600	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either in-situ soil, or OPSS Granular B Type I or II material over in-situ soil.	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on the contact zone between the subgrade material and the base stone remaining in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level, and the subgrade surface should be crowned to promote water flow to drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be designed for the future structures. The system should consist of a 150 mm diameter, geotextile wrapped, perforated, corrugated, plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Buildings proposed throughout the development of the subject site whose basement levels are founded below the long-term groundwater table should be provided a groundwater suppression system. The groundwater suppression system would consist of installing a waterproofing membrane over a drainage geocomposite installed on the exterior portion of the foundation wall. The waterproofing membrane is recommended to extend between the bottom of the foundation and up to a minimum of 1 m above the long-term groundwater level. A groundwater suppression system would also be recommended for structures located below the buildings foundations (i.e.- elevator shafts, sump pits, etc).

Due to the preliminary nature of the development, the requirement for groundwater suppression systems will be assessed once the number of proposed basement levels for the future mid and high-rise buildings is known. Details pertaining to the groundwater suppression system may also be provided at that time.

Backfill against the exterior sides of the foundation walls should consist of free-draining non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Backfill material below sidewalk or asphalt paved subgrade areas or other settlement sensitive structures should consist of free draining, non-frost susceptible material placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

6.2 Protection of Footings Against Frost Action

Perimeter footings, raft slabs, pile caps, and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

Other exterior unheated footings are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover (or insulation equivalent).

Where footings are founded directly on clean, surface-sounded bedrock with no near-surface cracks or fissures and is approved by Paterson personnel at the time of the excavation, the minimum soil cover, listed above, is not required.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structures are backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly a Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain a safe working distance from the excavation sides.

Excavation side slopes carried out for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff where shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

where:

N_b - stability factor dependent upon the geometry of the excavation and given in Figure 1 on the following page.

s_u - undrained shear strength of the soil below the base level

σ_z - total overburden and surcharge pressures at the bottom of the excavation

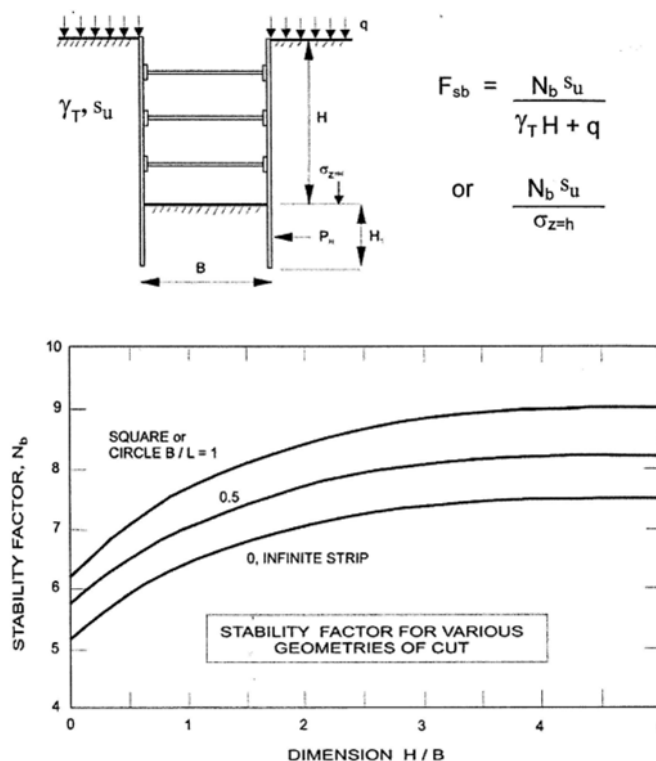


Figure 1 – Stability Factor for Various Geometries of Cut

A safety factor of 2 is generally recommended to be considered for excavation base stability.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and backfilling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

The backfill material within the frost zone (about 1.5 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub-bedding, and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD using a sheepsfoot roller. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

Due to the relatively impervious nature of the silty clay materials, it is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbances to the founding medium.

Due to the permeable glacial till deposit encountered within the groundwater table at the subject site, it is anticipated that conventional pumping with open sumps may be difficult to control the groundwater influx through servicing trench and deep basement excavations. If excavations well below groundwater level and within the permeable sand or glacial till deposits are considered, dewatering from outside the excavations using deep wells, well points or other means may be required.

Above the permeable sand and glacial till deposit, it is anticipated that groundwater infiltration into the excavations should be low to moderate, if encountered, and controllable using open sumps. Similar conditions may arise where excavations are undertaken below the depth of a silty clay deposit and into more permeable soils.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a non-aggressive to moderately aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Considerations

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed review of the soils at the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. The results of our Atterberg limits, shrinkage, and hydrometer testing are presented in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% for all the tested silty clay samples. In addition, based on the moisture levels and consistency of the encountered clay, the silty clay deposit encountered across the subject site is considered to be a low to medium sensitivity silty clay deposit.

The following tree planting setbacks are recommended for low to medium sensitivity silty clay deposits throughout the subject site.

Large trees (mature tree height over 14 m) can be planted at the subject site provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g., in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature height up to 7.5 m) and medium size trees (mature height 7.5 to 14 m), provided that the conditions noted below are met:

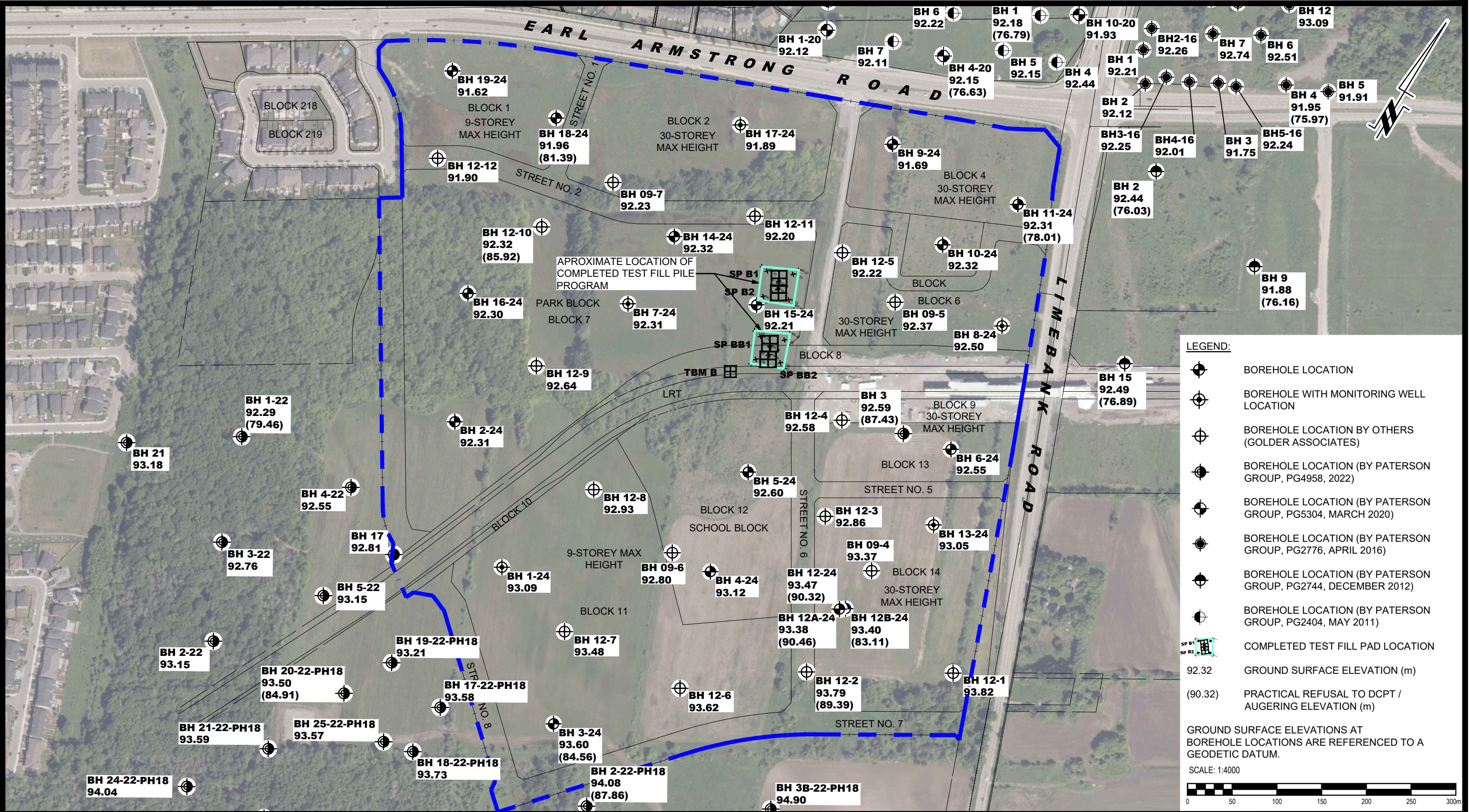
- ❑ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.
- ❑ A small tree must be provided with a minimum 25 m³ of available soil volume while a medium tree must be provided with a minimum of 30 m³ of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.
- ❑ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15 mm bars in the foundation wall).
- ❑ Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

Above-Ground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer's requirements.

Additional grading around the hot tub should not exceed permissible grade raise restrictions. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer's specifications.

Additional grading around proposed decks or additions should not exceed permissible grade raises restrictions. Otherwise, standard construction practices are considered acceptable.



LEGEND:

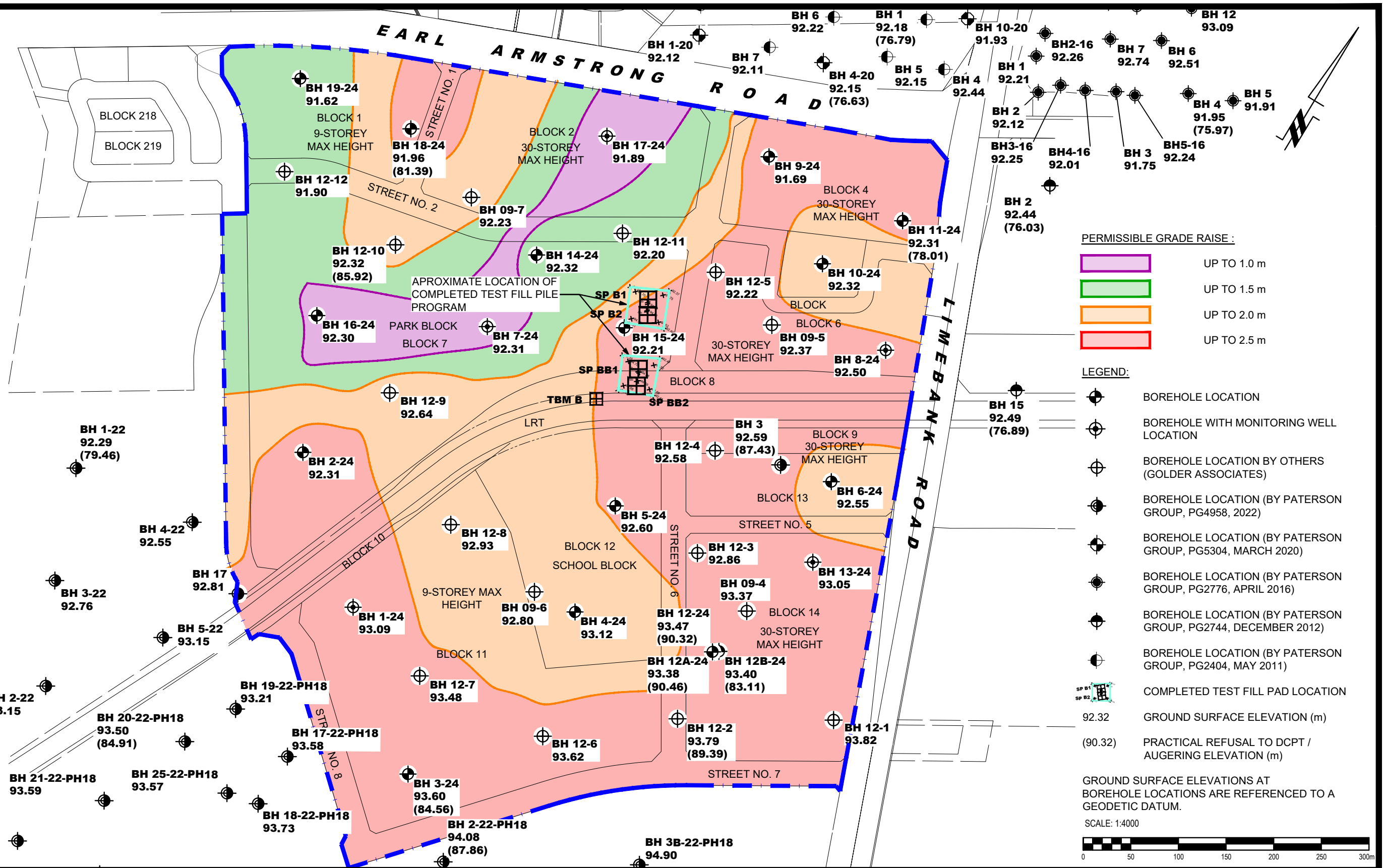
- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- BOREHOLE LOCATION BY OTHERS (GOLDER ASSOCIATES)
- BOREHOLE LOCATION (BY PATERSON GROUP, PG4958, 2022)
- BOREHOLE LOCATION (BY PATERSON GROUP, PG5304, MARCH 2020)
- BOREHOLE LOCATION (BY PATERSON GROUP, PG2776, APRIL 2016)
- BOREHOLE LOCATION (BY PATERSON GROUP, PG2744, DECEMBER 2012)
- BOREHOLE LOCATION (BY PATERSON GROUP, PG2404, MAY 2011)
- COMPLETED TEST FILL PAD LOCATION
- 92.32 GROUND SURFACE ELEVATION (m)
- (90.32) PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

0 50 100 150 200 250 300m

 9 AURIGA DRIVE OTTAWA, ON K2E 7T9 TEL: (613) 226-7381				RIVERSIDE SOUTH LIMITED PARTNERSHIP GEOTECHNICAL INVESTIGATION PROPOSED MIXED-USE DEVELOPMENT RIVERSIDE SOUTH - TOWN CENTER PHASE 7A ONTARIO			Scale: 1:4000	Date: 06/2024
				OTTAWA, ONTARIO			Drawn by: ZS	Report No.: PG4958-6
							Checked by: NP	Dwg. No.: PG4958-19
							Approved by: DJG	Revision No.:
NO.	REVISIONS	DATE	INITIAL	TEST HOLE LOCATION PLAN				



PERMISSIBLE GRADE RAISE :

[Purple Box]	UP TO 1.0 m
[Green Box]	UP TO 1.5 m
[Orange Box]	UP TO 2.0 m
[Red Box]	UP TO 2.5 m

LEGEND:

[Symbol]	BOREHOLE LOCATION
[Symbol]	BOREHOLE WITH MONITORING WELL LOCATION
[Symbol]	BOREHOLE LOCATION BY OTHERS (GOLDER ASSOCIATES)
[Symbol]	BOREHOLE LOCATION (BY PATERSON GROUP, PG4958, 2022)
[Symbol]	BOREHOLE LOCATION (BY PATERSON GROUP, PG5304, MARCH 2020)
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GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

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PROPOSED MIXED-USE DEVELOPMENT
RIVERSIDE SOUTH - TOWN CENTER PHASE 7A
OTTAWA, ONTARIO

Title: **PERMISSIBLE GRADE RAISE PLAN**

Scale:	1:4000	Date:	06/2024
Drawn by:	ZS	Report No.:	PG4958-6
Checked by:	NP	Dwg. No.:	PG4958-20
Approved by:	DJG	Revision No.:	

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