

Stormwater Management Report
Picton Cement Plant Improvements – Heidelberg Materials
Picton, ON

Revised

March 18, 2026

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Revision Summary

March 18, 2026

Stormwater Management Report issued for Site Plan Control application.

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1 Introduction and Background

Heidelberg Materials (HM) operates Picton Cement Plant & Quarry, a cement manufacturing facility, located at 1370 County Rd (CR) 49 in Picton ON (Figure 1-1).



Figure 1-1: Project site location (Google, Maxar Tech 2020)

The site is bounded by Picton Bay to the east and Elmbrook Rd and CR 5 to the west. CR 49 runs in a north-south direction through the centre of the site. There are agricultural lands immediately south and west of the site, and both agricultural and industrial (quarry) lands to the north. Small clusters of residential dwellings also exist within each of these areas. There are additional industrial developments (a shipping terminal and solar farm) further south and west respectively (Figure 1-2).



Figure 1-2: Project site and surrounding area (Google, Maxar Tech 2023)

HM is proposing an improvement to the cement plant, comprising the addition of a new 0.05 ha truck docking station and 0.73 ha paved staging area. HM is seeking approval through a Site Plan Control application. Jewell Engineering Inc. (Jewell) has been retained by HM to review the existing stormwater management system and determine any upgrades required to support the proposed improvements.

2 Existing Conditions

The project site has a total area of ~420 ha, and is generally gently sloped towards Picton Bay to the east. The project site comprises:

- An active quarry, located in the centre-west and southwest
- Picton Cement Plant (the Plant), located in the centre-east
- A decommissioned quarry (converted to a lagoon system), located in the southeast
- Agricultural lands, located in the northwest
- Undeveloped/heavily-treed lands, located in the northeast and along site boundaries.

The surfaces of the quarries are primarily exposed bedrock, and the surfaces of the Plant are primarily granular material and shallow, disturbed soil. There are some areas of concrete paving (parking areas and pathways) throughout the site (Figure 1-2).

There is an existing storm sewer system, which drains to the lagoon system in the southeast. The storm sewers were designed to convey the 10-yr storm event in addition to process waters contributed by the Plant and potentially some intercepted groundwater. It should be noted that, due to heavily disturbed site conditions, several sections of the storm sewer have previously been unable to be mapped or investigated and thus introduces some uncertainty.

The site operates a water recycling process, with the lagoon system functioning as a water management pond (WMP). This water recycling system includes stormwater runoff that is all directed to the WMP where it is clarified by settling.

The Plant holds two permits to take water (PTTW). The first is for dewatering of the active quarry, the water from which is mixed with Plant process waters, stormwater runoff, and groundwater as they are conveyed to the WMP by the storm sewer system. The water is then clarified within the WMP. The second PTTW allows withdrawal of fresh water from Picton Bay. However, Plant operators have not needed to draw water from Picton Bay in the past 10 years. Clarified water from the WMP is discharged infrequently to Picton Bay.

2.1 Soils, Bedrock & Groundwater

Soils were reviewed from the Soil Survey of Prince Edward County (Ontario Ministry of Agriculture and Food, 1948) and National Soil Database (Government of Ontario, 2019). The site consists of hydrologic soil group (HSG) B and HSG C soil (Figure 2-1) (Figure 2-2).

Soils found on the site include:

- Ameliasburg Loam
 - These soils are naturally well drained, and the material is comprised primarily of sand ($\geq 44\%$) and silt ($\geq 30\%$). Ameliasburg Loam is classified as HSG B.
- Farmington Loam
 - These soils are naturally well drained, and the material is comprised primarily of sand ($\geq 44\%$) and silt ($\geq 36\%$). Farmington Loam is classified as HSG B.
- Rockland
 - This is a true non-soil, with exposed or extremely shallow bedrock. Drainage and material composition are not applicable. Rockland is classified as HSG B.
- South Bay Clay Loam
 - These soils are naturally moderately well drained, and the material primarily comprises clay ($\geq 52\%$) and silt ($\geq 34\%$). South Bay Clay Loam is classified as HSG C.
- Elmbrook Clay / Clay Loam
 - These soils are naturally imperfectly drained. The composition of both materials is known to be similar, consisting primarily of clay ($\geq 50\%$) and silt ($\geq 40\%$). Elmbrook Clay and Elmbrook Clay Loam are classified as HSG C.

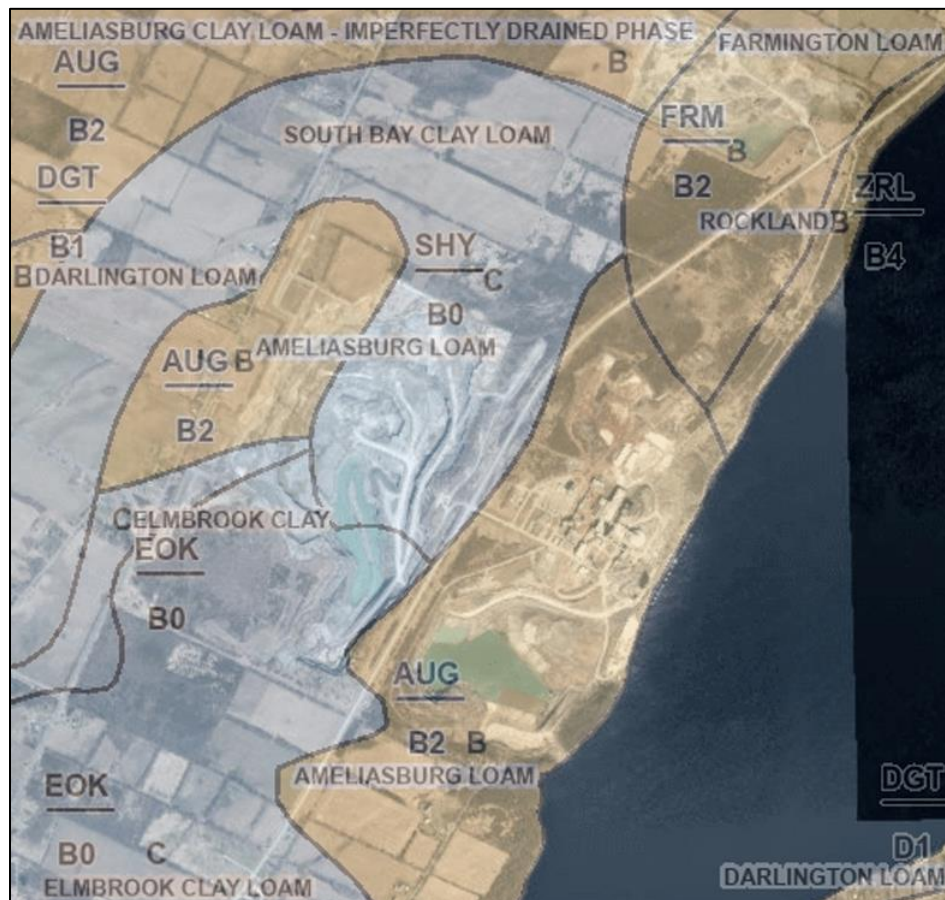


Figure 2-1: Project site soils (AgMaps, Kings Printer for Ontario 2026)

Hydrologic Soil Group

The hydrologic soil group is used to classify soils into groups of various runoff potential.

The Soil Conservation Service (SCS) classifies bare thoroughly wet soils into four hydrologic soil groups (A, B, C and D). SCS descriptions of the four groups, modified slightly to suit Ontario conditions, are as follows: (Design Chart 1.09)

- A: High infiltration and transmission rates when thoroughly wet, eg. deep, well drained to excessively-drained sands and gravels. These soils have a low runoff potential.
- B: Moderate infiltration and transmission rates when thoroughly wet, such as moderately deep to deep open textured loam.
- C: Slow infiltration and transmission rates when thoroughly wet, eg. fine to moderately fine-textured soils such as silty clay loam.
- D: Very slow infiltration and transmission rates when thoroughly wet, eg. clay loams with a high swelling potential. These soils have the highest runoff potential.

In Ontario, soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC, CD as appropriate, such as Guelph loam, which is classified as BC.

Figure 2-2: MTO Drainage Management Manual – Description of Hydrologic Soils Groups

The surfaces of the quarries consist primarily of exposed limestone bedrock. The Rockland soil group also indicates the presence of exposed or extremely shallow bedrock. From this, it can be inferred that bedrock is very shallow throughout the site and that the remaining areas of soil are a thin surface layer. This aligns with the bedrock conditions detailed in the 2018 SWM Report by Cornerstone Builders Ltd. (CBL), which has been included in Appendix F.

The 2018 CBL Report also specified that the site storm sewer system includes a gravel bedding that intercepts groundwater. The 2025 Subsurface Utility Plan (included in Appendix D) shows that the inverts of most storm sewer segments are 2-3 m below the top of grate of their connecting structures, with several only 1 m below. Thus, it can be inferred that groundwater is present at a depth of ~1-3 m throughout the site. This aligns with expectations for high-elevation base flows adjacent to Picton Bay.

3 Purpose of this Report

This report provides a drainage design for the proposed site improvements. The existing stormwater management system (the storm sewers and WMP) and its modelling from the 2018 CBL Report were reviewed. The model was updated to incorporate the proposed improvements and any works completed since 2018. The system's performance under the updated conditions was analyzed to determine any upgrades required to support the proposed improvements.

3.1 Proposed Improvements

The Plot Plan for the project is included in Appendix A.

The proposed improvements comprise the addition of a new 0.05 ha truck docking station and 0.73 ha paved staging area. This will improve the ease of loading and unloading trucks, and help control dust and improve runoff quality through easier maintenance. The improvements will be located in the northern extent of the Plant, north of the clinker storage hall and east of the corrugated steel shelters, replacing an area of granular material (Figure 3-1).

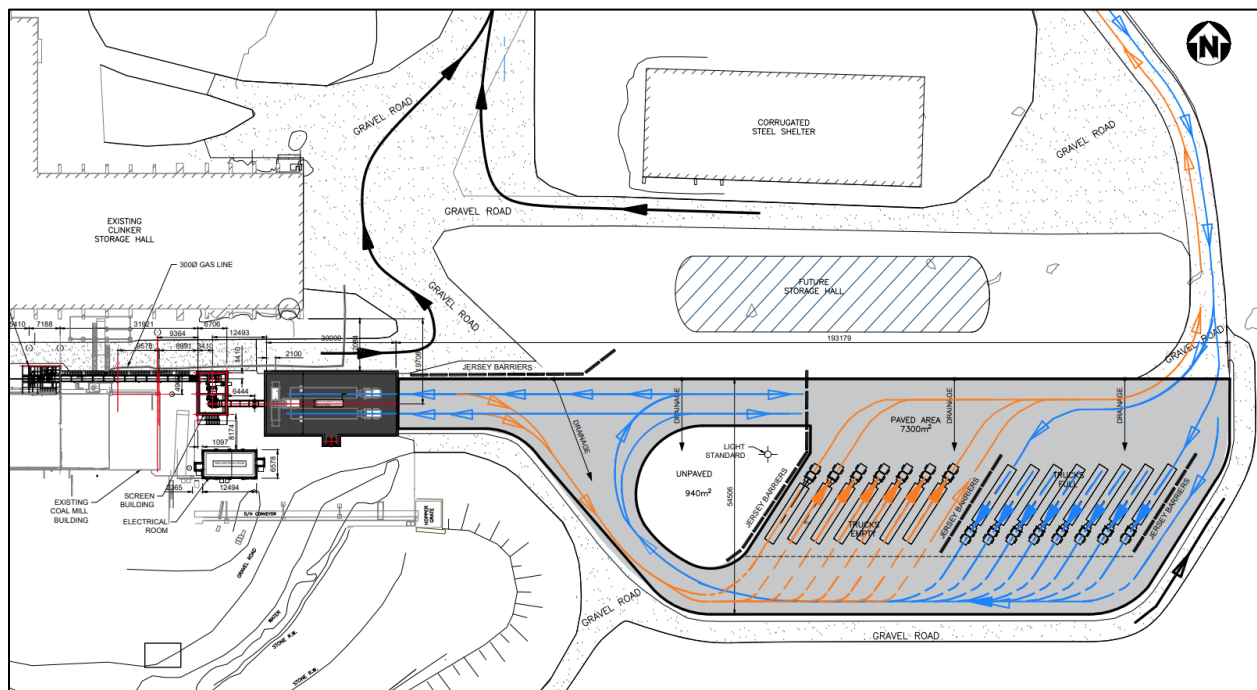


Figure 3-1: Project Plot Plan excerpt (rCon Engineers Inc., 2026)

4 Hydrologic Modelling

A hydrologic model of the site was prepared by CBL in their 2018 SWM Report. No major changes to the land use, topography, or other site conditions have occurred since; the only alteration has been the addition of the corrugated steel shelters to the north of the Plant. As such, the 2018 modelling remains applicable, and only requires updating to incorporate the corrugated steel shelters and proposed improvements.

4.1 Improvement Impacts

The 2018 CBL Report modelled the Plant site using 16 catchment areas based on LiDAR topographic data. Both the corrugated steel shelters and proposed improvements are located in Catchment Area (CA) 104, which is 9.75 ha in total (Figure 4-1).

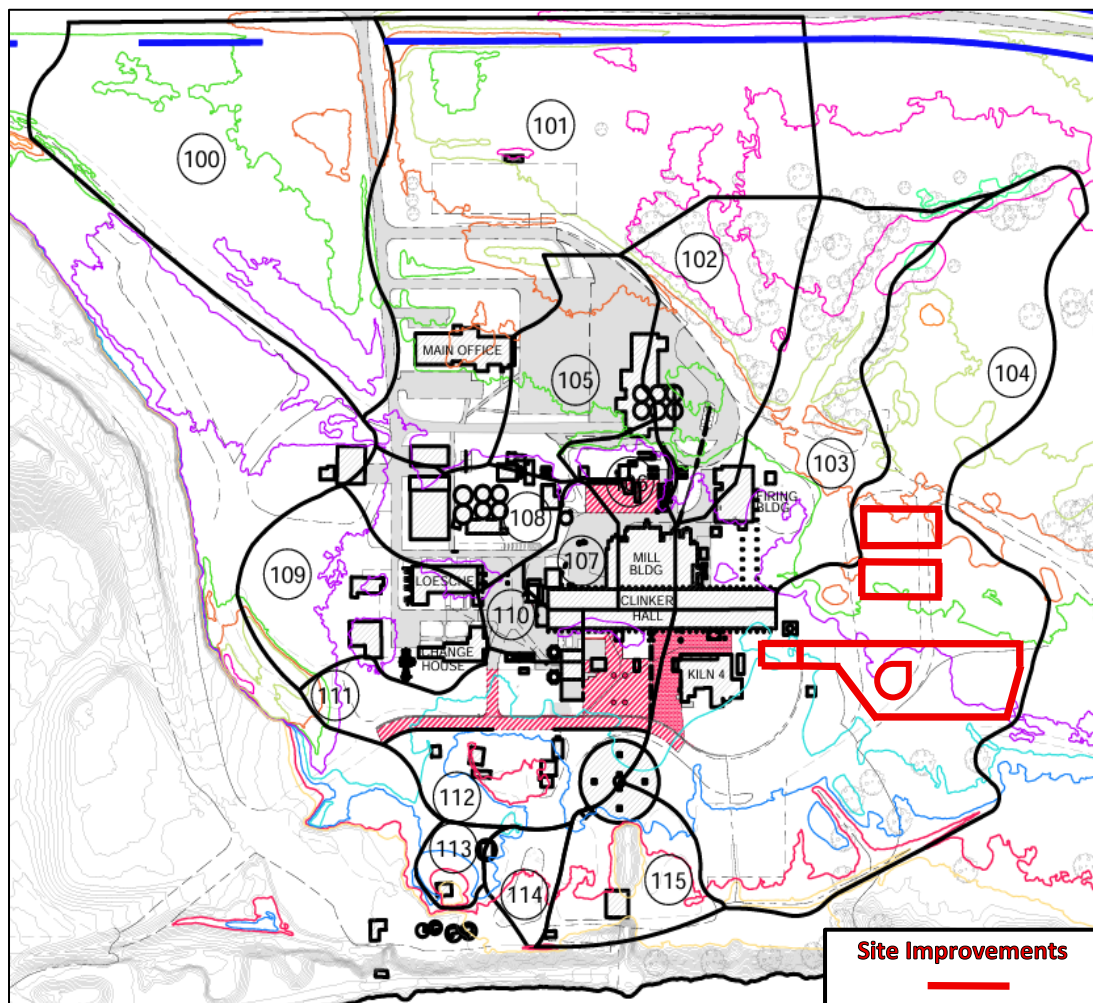


Figure 4-1: Storm Drainage and Catchment Plan excerpt showing the location of later site improvements (CBL, 2018)

The updates will result in changes to the runoff coefficient and time of concentration CBL previously calculated for CA 104. Equation 1 shows the calculation method used to determine the weighted runoff coefficient, and Equation 2 shows the Airport Method used to determine the time of concentration.

Equation 1: Weighted Runoff Coefficient

$$C_w = \frac{C_1 \times A_1 + C_2 \times A_2 + \dots C_n \times A_n}{A_1 + A_2 + \dots A_n}$$

Where:

C_i = Runoff coefficient for A_i
 A_i = Land area (ha) of cover type

Equation 2: Airport Method of Determining Time of Concentration

$$T_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{0.33}}$$

Where:

T_c = Time of concentration (min)
 C = Runoff coefficient
 L = Length of flow path (m)
 S_w = Slope of the watershed (%)

It should be noted that, while the maximum runoff coefficient of 0.4 recommended for use of the Airport Method was often exceeded, the Bransby-Williams method recommended for higher coefficients is based on highly channelized flow. As the project site is dominated by overland flow, this would not accurately represent site conditions and produce inaccurate results; the Airport Method was more suitable.

The site improvement updates comprise:

- 0.73 ha paved truck staging area
 - Previous cover: gravel (runoff coefficient = 0.6 per 2018 CBL Report)
 - Proposed cover: concrete (runoff coefficient = 0.9 per 2018 CBL Report)
- 0.05 ha truck docking station
 - Previous cover: gravel (runoff coefficient = 0.6 per 2018 CBL Report)
 - Proposed cover: building (runoff coefficient = 0.9 per 2018 CBL Report)
- 0.34 ha corrugated steel shelters
 - Previous cover: gravel (runoff coefficient = 0.6 per 2018 CBL Report)
 - Current cover: building (runoff coefficient = 0.9 per 2018 CBL Report)

4.1.1 Runoff Coefficient

The runoff coefficient for CA 104 was calculated as 0.60 in the 2018 CBL Report. The updated runoff coefficient was calculated as 0.63. The calculation process is shown below.

$$0.60 = \frac{8.63 (C_{unupdated}) + 1.12 (0.60)}{9.75}$$

$$C_{unupdated} = 0.60$$

$$C_{(2026)} = \frac{8.63 (0.6) + 1.12 (0.9)}{9.75}$$

$$C_{(2026)} = \mathbf{0.63}$$

4.1.2 Time of Concentration

CA 104 and CA 115 are the two catchment areas contributing to a segment of the storm sewer system. As such, they were treated as one 10.95 ha catchment (runoff coefficient = 0.60) in the 2018 CBL analysis, with an initial time of concentration calculated as 20 min. The updated initial time of concentration was calculated as 18.8 min. The calculation process is shown below.

$$C_{Combined(2026)} = \frac{9.75 (0.63) + 1.2 (0.60)}{10.95}$$

$$C_{Combined(2026)} = 0.63$$

$$20 \text{ min} = 3.26 (1.1 - 0.60) \left(\frac{L^{0.5}}{S_W^{0.33}} \right)$$

$$\left(\frac{L^{0.5}}{S_W^{0.33}} \right) = 12.27$$

$$T_{C(2026)} = 3.26 (1.1 - 0.63) (12.27)$$

$$T_{C(2026)} = \mathbf{18.8 \text{ min}}$$

It should be noted that the 34.5 min initial time of concentration for CA 101 was decreased to 30 min for conservatism; a similar process may or may not have been applied to CA 104. As such, calculation based on the 20 min time of concentration will either be accurate or conservative. The length and slope of the flow path remain applicable as the site topography remains unchanged.

4.2 Updated Model

The catchment area properties used in the updated modelling, including values previously determined in the 2018 CBL Report and those updated in this report, are summarized in Table 4-1

Table 4-1: Updated catchment area properties (*Updated in 2026) (CBL, 2018)

Catchment Area	Area (ha)	Runoff Coefficient
100	5.71	0.25
101	7.67	0.4
102	2.31	0.60
103	3.31	0.60
104	9.75	*0.63
105	1.57	0.9
106	0.78	0.9
107	0.53	0.9
108	1.21	0.9
109	2.04	0.4
110	0.40	0.9
111	1.62	0.9
112	1.26	0.9
113	0.31	0.9
114	0.43	0.9
115	1.20	0.6

The 2018 CBL report determined times of concentration for the catchment(s) contributing to each section of storm sewer. 20 min and 30 min times of concentration were used for the initial sewer sections, with times of concentration at downstream pipes increased based on the time of flow in the previous section. The times of concentration for the contributing catchment(s) used in the updated modelling, including those previously determined by CBL and those updated in this report, are summarized in Table 4-2.

Table 4-2: Updated catchment area times of concentration (*Updated in 2026) (CBL, 2018)

Catchment Area(s)	Runoff Coefficient(s)	Time of Concentration (min)
102, 103	0.6	20
106	0.9	20.4
100, 101, 105	0.25, 0.4, 0.9	30
108	0.9	30.4
107	0.9	30.5
110	0.9	30.8
109	0.4	20
111	0.9	31
112	0.9	20
113, 114	0.9	32.2
104, 115	*0.63	*18.8

5 Storm Sewer Design

The storm sewer design sheet (SSDS) developed in the 2018 CBL Report was updated to incorporate the corrugated steel shelters and proposed improvements. The SSDS from the 2018 CBL Report and the newly-updated SSDS's are included in Appendix C.

It should be noted that due to the heavily-disturbed site conditions, both the 2018 CBL Report and 2025 Subsurface Utility Plan indicate that several sections of the storm sewer system are unknown/could not be investigated. As such, minor uncertainties are associated with this modelling update, however, the findings and conclusions remain valid.

5.1 Hydraulic Modelling

Storm sewer design sheets are prepared by comparing peak flow rates calculated using the Rational Method to pipe capacity calculated using Manning's Equation. The Rational Method is included as Equation 3, and Manning's Equation is included as Equation 4.

Equation 3: Rational Method

$$Q = \frac{1}{360} * C * i * A$$

Where:

Q = Peak flow (m³/s)

C = Runoff coefficient

I = Rainfall Intensity (mm/hr)

A = Area (ha)

Equation 4: Manning's Equation

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

Where:

N = Manning's roughness coefficient

A = Pipe area (m²)

R = Hydraulic radius (A/P)

P = Wetted perimeter (m)

S = Slope (m/m)

The inputs to the updated SSDS are discussed below.

5.1.1 Area

The areas of each catchment are summarized in Table 4-1.

5.1.2 Runoff Coefficient

The runoff coefficients of each catchment are summarized in Table 4-1.

5.1.3 Intensity

Precipitation records from Environment Canada for Belleville station 6150689 were used, as Belleville is the closest station to the project site. The most recent V3.3 intensity-duration-frequency (IDF) curves were released October 2022, and contain data from 1960-2016. These IDF curves are included in Appendix B.

Rainfall intensity was calculated using the Rainfall Intensity Equation included as Equation 5.

Equation 5: Rainfall Intensity Equation

$$I = A * T_c^B$$

Where:

I = Rainfall Intensity in mm/hr

A = Rainfall Coefficient

B = Rainfall Exponent

T_c = Time of Concentration in hrs

The rainfall coefficients for the 10-yr storm event are:

- A = 30.5
- B = -0.678

5.1.4 Time of Concentration

The times of concentration for the catchment(s) contributing to each section of storm sewer are summarized in Table 4-2.

5.1.5 Manning's Roughness Coefficient

All pipes were assigned the standard coefficients of 0.013 for PVC/RCP pipes and 0.024 for CSP pipes, as determined from the SSDS in the 2018 CBL Report.

5.1.6 Pipe Properties

Pipe lengths, slopes, and sizes were initially set as those from the SSDS in the 2018 CBL Report, including the recommended upgrades. Pipe size was then increased as required to satisfy capacity constraints.

5.2 Results

The 10-yr storm sewer design sheets for the 2026 updated conditions and proposed storm sewer upgrades are included in Appendix C.

The following storm sewer sections are undersized:

- **900 CSP Pipe – Gate Chamber**
- **Pipe Between Gate Chamber – Holding Pond**
- **Pipe between MH11 – MH12**
- **Pipe between MH12 – Gate Chamber**

It should also be noted that there is a section of unknown storm sewer running east from the location of the proposed improvements to the well-defined ditch that contributes to the 900 mm CSP pipe. It is not known if this pipe receives flows, or what its capacity is. However, as runoff will flow overland to the ditch if it is inaccessible or undersized, the condition of this pipe will not impact the drainage design of the proposed improvements.

5.2.1 900 CSP – Gate Chamber

The 900 mm CSP pipe has a q/Q (flow/capacity) ratio of 1.85 under the updated conditions, increased from the 1.69 ratio in the 2018 CBL Report. These ratios are based on the assumed pipe slope of 0.5% from the 2018 CBL Report. However, even if the pipe is undersized, flows in excess of the capacity will be temporarily retained in the well-defined ditch north of the pipe.

If the pipe was upsized to 1050 (1066) mm, the q/Q ratio would decrease to 1.18. This would still impose storage in the ditch, but it would be significantly lower than that of the current design. The pipe would need to be upsized to 1200 mm to convey all flows without retention.

5.2.2 Gate Chamber – Holding Pond

The 1050 (1066) mm RCP pipe between the Gate Chamber and Holding Pond has an updated q/Q ratio of 1.46, increased from the 1.44 ratio in the 2018 CBL Report. However, these ratios are based on the pipe properties from the 2018 CBL Report, which noted that the pipe may be either 1050 mm or 1200 mm.

If the pipe is 1200 mm, the q/Q ratio will decrease to 1.07, which is the maximum q/Q that pipes are able to convey (per Design Chart 2.30 of the MTO Drainage Management Manual).

5.2.3 MH11 – MH12 & MH12 – Gate Chamber

The 1050 (1066) mm RCP pipes between MH11-MH12 and MH12-Gate Chamber have updated q/Q ratios of 1.03 and 1.02 respectively. These values are the same as those from the 2018 CBL Report, as the contributing catchments were not impacted by the model updates. As the q/Q ratios are less than 1.07, the pipes remain sufficient to convey flows.

5.3 Recommendations

To support the proposed site improvements, Jewell recommends:

- **Ongoing monitoring of the 900 mm CSP pipe and the contributing ditch**
 - If ditch storage is sufficient: no upgrades recommended
 - If ditch storage is insufficient, that is, if stormwater is not contained within the ditch and spills to Picton Bay: upsizing from 900 mm – 1050 mm recommended
- **Investigation of the RCP Gate Chamber – Holding Pond pipe**
 - If the pipe is 1200 mm: no upgrades recommended
 - If the pipe is 1050 mm: upsizing from 1050 mm - 1200 mm recommended

Investigation (and potential upsizing) of the RCP Gate Chamber – Holding Pond pipe should be completed in conjunction with the proposed improvements. Upsizing of the 900 mm CSP pipe should be completed if concerns regarding the ditch storage are identified.

6 Water Management Facility

The 1996 Water Management Plan prepared by Water and Earth Science Associated Ltd. (WESA) estimated the WMP to have a residence time of ~97 days. The WESA Report also referenced effluent settling tests by Beak Engineering Ltd. (Beak) in 1991. Beak found that a settling time of 24 hrs was sufficient to achieve the TSS limits set for the site under the Municipal/Industrial Strategy for Abatement program. The WMP provides a settling period greatly exceeding the requirement. Excerpts from the WESA Report are included in Appendix E.

The WMP would also satisfy the *Enhanced* treatment (80% TSS removal) target of the 2006 Bay of Quinte Remedial Action Plan Stormwater Management Design Guidelines. Per Table 3.2 of the 2003 Stormwater Management Planning and Design Manual, a site with 85% imperviousness requires a wet pond with 250 m³/ha of storage. At ~420 ha, the project site would require 105,000 m³ of storage if it were at this level of imperviousness. However, the WMP provides 446,720 m³ of storage based on the 2025 Lagoon Survey (included in Appendix D). This greatly exceeds the storage volume required to provide *Enhanced* treatment.

Stormwater generated by the proposed improvements will not impact WMP quality.

The area contributing to the WMP has not changed. However, the corrugated steel shelters and proposed improvements cause surface hardening, which increases runoff volume (e.g. peak flows of CA 104 increased by 0.116 m³/s). This increased runoff can be accommodated by the excess storage of the WMP, or discharged to Picton Bay with other excess water from the WMP if required.

Stormwater generated by the proposed improvements will not impact WMP storage.

Under normal conditions, the stormwater is in a closed loop and does not discharge into the natural environment. If a discharge event is triggered, only treated water will be pumped to Picton Bay. The treated water has been clarified for approximately 97 days.

The site improvements are expected to have no impact on the natural environment.

7 Conclusions & Recommendations

Heidelberg Materials is proposing the following improvements to the Picton Cement Plant & Quarry:

- 0.05 ha truck docking station
- 0.73 ha paved truck staging area

Modelling of the stormwater management system was previously completed by Cornerstone Builders Ltd. in their 2018 SWM Report. This modelling was updated to account for the impact of the proposed improvements and the works completed since 2018.

Jewell found the 2026 proposed improvements will result in a small increase in peak flows that will be conveyed to the existing water management facility through the existing storm sewer network. Since a full inventory of the existing storm sewer system is not available, there is some uncertainty in the capacity of the system to convey the expected flows to the water management facility.

Despite uncertain knowledge of the storm sewer system, the Plant operators have provided observations of flows and did not report any surcharges of stormwater runoff.

Storm Sewer System

To support the proposed improvements, Jewell recommends:

- **Ongoing monitoring of the 900 mm CSP pipe and the contributing ditch**
 - If ditch storage is not exceeded, that is no spills occur: no upgrades recommended
 - If ditch storage is insufficient and spills out of the ditch are observed: upsizing of the 900 mm pipe to a 1050 mm pipe would be recommended
- **Investigation of the RCP Gate Chamber – Holding Pond pipe**
 - If the pipe is 1200 mm: no upgrades recommended
 - If the pipe is 1050 mm: upsizing to a 1200 mm pipe is recommended

Recommendations should be completed in conjunction with the proposed improvements.

Water Management Facility

No changes to the water management facility are required.

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Reviewed by



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Jewell Engineering Inc.

8 References

Cornerstone Builders Ltd. (2018). *Stormwater Management Report*

Government of Ontario. (2024). *National Soil Database*.

Ministry of the Environment. (2003). *Stormwater Management Planning and Design Manual*.

Ministry of Transportation of Ontario. (1997). *MTO Drainage Management Manual*.

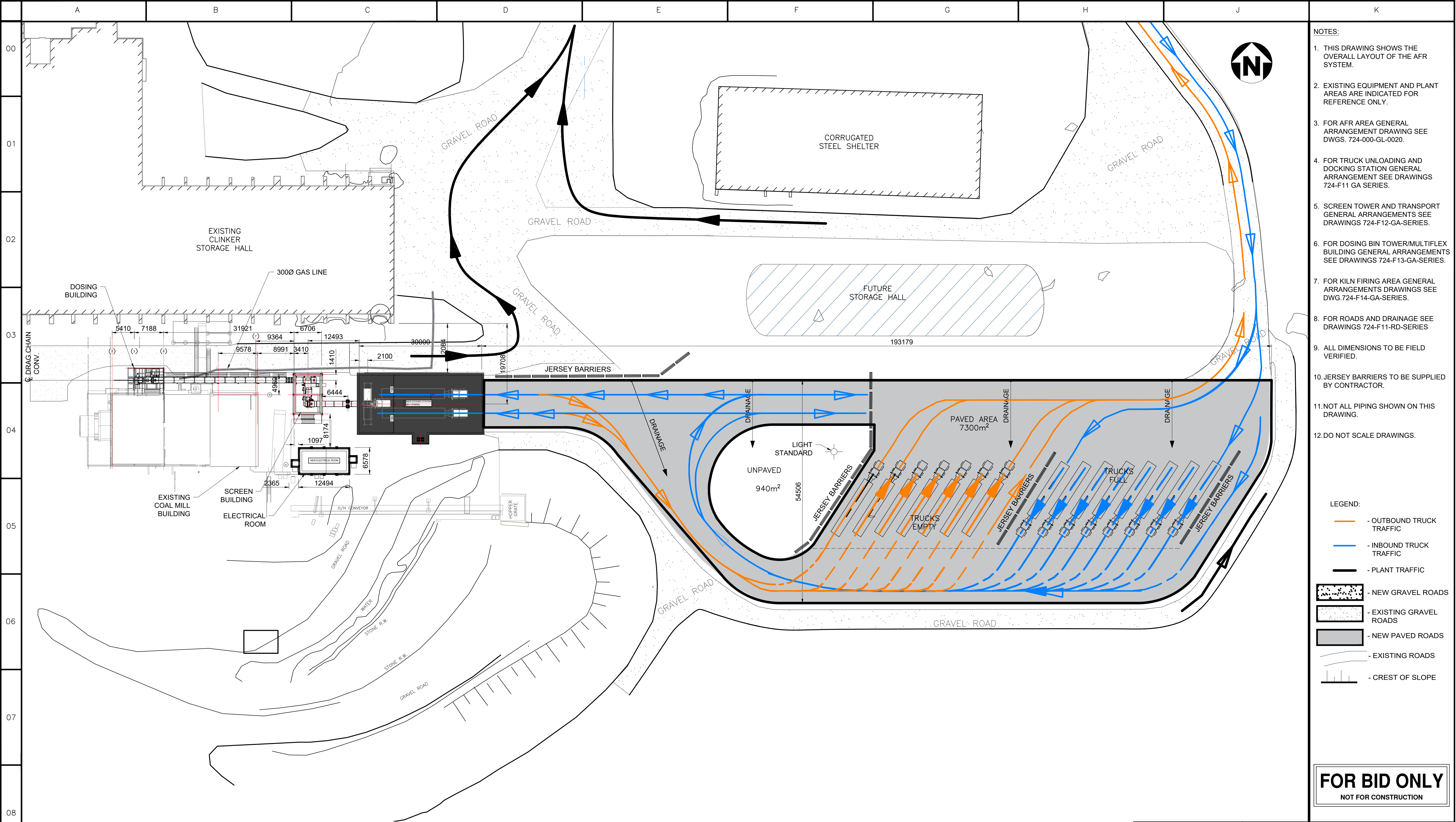
Ontario Ministry of Agriculture and Food. (1948). *Soil Survey of Prince Edward County*.

Quinte Conservation Authority. (2006). *Bay of Quinte Remedial Action Plan Stormwater Management Design Guidelines*.

Water and Earth Science Associated Ltd. (1996). *Water Management Plan*

APPENDIX A

Project Plot Plan



- NOTES:
1. THIS DRAWING SHOWS THE OVERALL LAYOUT OF THE AFR SYSTEM.
 2. EXISTING EQUIPMENT AND PLANT AREAS ARE INDICATED FOR REFERENCE ONLY.
 3. FOR AFR AREA GENERAL ARRANGEMENT DRAWING SEE DWGS. 724-000-GL-0020.
 4. FOR TRUCK UNLOADING AND DOCKING STATION GENERAL ARRANGEMENT SEE DRAWINGS 724-F11 GA SERIES.
 5. SCREEN TOWER AND TRANSPORT GENERAL ARRANGEMENTS SEE DRAWINGS 724-F12-GA-SERIES.
 6. FOR DOSING BIN TOWER/MULTIFLEX BUILDING GENERAL ARRANGEMENTS SEE DRAWINGS 724-F13-GA-SERIES.
 7. FOR KILN FIRING AREA GENERAL ARRANGEMENTS DRAWINGS SEE DWG.724-F14-GA-SERIES.
 8. FOR ROADS AND DRAINAGE SEE DRAWINGS 724-F11-RD-SERIES
 9. ALL DIMENSIONS TO BE FIELD VERIFIED.
 10. JERSEY BARRIERS TO BE SUPPLIED BY CONTRACTOR.
 11. NOT ALL PIPING SHOWN ON THIS DRAWING.
 12. DO NOT SCALE DRAWINGS.

- LEGEND:
- OUTBOUND TRUCK TRAFFIC
 - INBOUND TRUCK TRAFFIC
 - PLANT TRAFFIC
 - NEW GRAVEL ROADS
 - EXISTING GRAVEL ROADS
 - NEW PAVED ROADS
 - EXISTING ROADS
 - CREST OF SLOPE

FOR BID ONLY
NOT FOR CONSTRUCTION

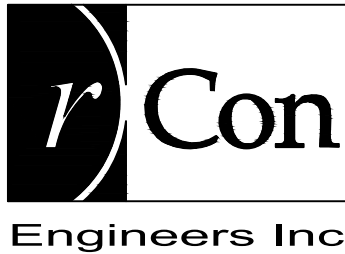
OVERALL AFR PLOT PLAN

DWG. NO.	TITLE	REV:	-	DATE	-	REV:	-	DATE	-	REV:	A	DATE	ISSUED FOR BIDS
		DWN:	-	-	-	DWN:	WP	03-FEB-26	-	DWN:	WP	03-FEB-26	-
		CHD:	-	-	-	CHD:	BB	03-FEB-26	-	CHD:	BB	03-FEB-26	-
		APPR:	-	-	-	APPR:	IK	03-FEB-26	-	APPR:	IK	03-FEB-26	-

Heidelberg Materials
Competence Center Cement NAM
7660 Imperial Way
Allentown, PA 18195

PICTON AFR PROJECT
GENERAL ARRANGEMENT
OVERALL PLOT PLAN

SEAL	DATE	DWG. TYPE	CIVIL	SCALE 1:400
DRAWN	WP	FEB/2026	PLANT	PICTON
CHD.	BB	FEB/2026	DWG. NO. 724-000-GL-0010	REV. A
APPR.	IK	FEB/2026		



7611 Pine Valley Dr.
Unit 28
Woodbridge, Ontario
L4L 0A2
Tel: 905.264.7414

EXTERNAL CONTRACTOR

APPENDIX B

Environment Canada IDF Curves

Environment and Climate Change Canada
Environnement et Changement climatique Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes
de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2022/10/31

=====

BELLEVILLE ON 6150689
(composite)
Latitude: 44 9'N Longitude: 77 23'W Elevation/Altitude: 76 m
Years/Années : 1960 - 2016 # Years/Années : 44

=====

Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1960	6.3	9.1	12.4	23.4	25.4	35.1	53.8	55.1	55.9
1961	6.1	7.9	8.9	12.2	18.0	18.8	23.9	34.0	36.3
1963	12.4	19.0	23.1	28.4	30.7	31.0	31.0	31.7	44.4
1964	4.3	5.6	7.4	12.2	12.4	20.6	45.2	45.7	45.7
1966	6.3	8.9	10.4	11.9	13.2	16.0	32.8	37.8	38.1
1967	7.4	9.9	10.4	10.4	11.9	13.2	26.4	42.4	58.7
1968	7.9	11.9	13.7	18.5	21.8	27.2	43.9	57.1	57.1
1969	5.8	9.7	13.0	17.5	24.4	31.0	37.8	43.2	62.2
1971	7.4	11.4	13.0	23.9	25.1	25.4	25.7	25.7	32.5
1972	9.4	10.7	11.7	12.4	14.7	20.6	28.2	33.5	50.5
1973	7.4	10.7	11.9	18.0	21.3	21.8	37.3	45.5	48.0
1974	10.9	15.2	17.8	25.4	25.4	25.4	34.3	42.7	42.7
1977	4.8	8.1	9.9	14.7	25.1	30.2	60.5	66.0	66.0
1980	13.2	16.9	19.0	20.5	20.5	34.6	46.9	47.6	59.6
1981	-99.9	-99.9	13.3	25.5	29.4	34.6	46.2	49.2	57.4
1982	4.6	8.5	10.1	14.2	18.3	24.7	39.8	45.0	45.0
1983	6.5	8.9	10.5	18.4	22.2	30.7	39.6	39.6	50.3
1984	5.1	8.1	10.1	11.3	19.7	23.7	33.4	51.4	55.1
1985	10.5	16.2	20.0	27.0	27.4	42.3	42.3	44.5	44.5
1986	9.1	14.4	16.4	23.2	25.2	35.0	59.2	68.8	78.9

1987	4.3	6.6	9.3	14.2	24.7	37.1	39.2	39.2	39.2
1988	3.7	6.2	7.4	8.6	9.2	10.6	20.8	22.2	28.2
1989	14.5	16.7	17.9	18.4	24.2	24.2	27.7	27.7	37.7
1990	6.9	8.3	10.0	12.3	13.6	20.8	29.7	34.8	38.7
1991	8.5	13.8	18.5	18.8	32.0	32.4	32.4	32.4	32.7
1992	6.3	7.6	8.4	13.2	18.8	21.9	38.2	48.3	50.1
1993	8.2	15.8	23.6	28.3	28.3	28.3	-99.9	-99.9	69.6
1994	8.8	10.2	14.5	18.3	23.6	25.5	38.2	49.2	52.8
1995	8.0	12.9	14.9	19.3	27.5	31.5	48.5	58.5	67.3
1996	6.9	10.4	13.4	19.2	25.1	41.3	41.5	53.8	53.8
1997	10.3	16.8	20.9	25.5	42.8	50.0	56.0	56.0	56.0
1998	9.5	12.1	15.1	22.1	25.0	32.6	38.6	38.6	50.2
1999	9.6	13.1	17.9	23.2	29.4	36.9	42.8	72.7	72.7
2000	10.4	13.4	14.7	16.8	29.0	39.8	52.0	52.4	53.0
2001	7.4	10.1	11.0	11.8	16.7	17.4	21.2	31.6	39.8
2002	7.1	9.4	14.0	21.0	22.4	26.0	39.4	44.2	49.8
2003	7.6	13.5	20.1	26.2	27.0	27.0	31.1	-99.9	56.2
2004	14.4	22.1	28.8	33.3	33.3	49.0	89.9	114.4	124.5
2006	9.0	14.7	18.8	19.5	19.5	19.5	37.3	42.7	59.8
2009	6.3	10.7	14.1	20.2	21.7	30.6	36.9	52.3	68.0
2011	12.7	20.3	25.8	31.1	39.7	44.3	49.1	49.1	51.2
2012	6.2	11.3	16.1	26.0	30.0	30.8	44.4	64.6	65.4
2013	7.4	11.6	15.7	19.3	21.5	23.2	29.0	32.8	33.9
2014	7.7	12.0	14.7	16.6	18.1	26.7	34.0	42.4	63.2
2015	5.3	8.5	11.4	17.0	29.0	48.2	56.4	60.6	73.7
2016	7.0	9.8	14.7	22.9	24.4	24.6	32.2	41.6	41.6

# Yrs.	45	45	46	46	46	46	45	44	46
Années									
Mean	8.0	11.8	14.7	19.4	23.7	29.2	39.9	47.0	53.4
Moyenne									
Std. Dev.	2.6	3.8	4.9	5.9	6.8	9.2	12.5	15.5	16.1
Écart-type									
Skew.	0.80	0.78	0.86	0.26	0.30	0.43	1.54	1.96	1.88
Dissymétrie									
Kurtosis	3.48	3.43	3.72	2.61	3.95	3.11	7.71	10.31	10.23

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount

Avertissement : la quantité maximale annuelle excède la quantité
pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
2004	6 h	89.9	78.9
2004	12 h	114.4	95.6
2004	24 h	124.5	104.0

Table 2a : Return Period Rainfall Amounts (mm)

Quantité de pluie (mm) par période de retour

Duration/Durée	2	5	10	25	50	100	#Years Années
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	
5 min	7.6	9.9	11.4	13.3	14.8	16.2	45
10 min	11.1	14.5	16.7	19.5	21.5	23.6	45
15 min	13.9	18.2	21.0	24.6	27.3	29.9	46
30 min	18.4	23.6	27.1	31.4	34.7	37.9	46
1 h	22.6	28.5	32.5	37.5	41.3	45.0	46
2 h	27.7	35.8	41.1	47.9	52.9	57.9	46
6 h	37.8	48.8	56.1	65.3	72.2	78.9	45
12 h	44.5	58.1	67.2	78.6	87.1	95.6	44
24 h	50.8	65.0	74.5	86.4	95.2	104.0	46

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits

Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

Duration/Durée	2	5	10	25	50	100	#Years Années
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	
5 min	90.7	118.5	136.9	160.1	177.4	194.5	45
	+/- 8.4	+/- 14.2	+/- 19.2	+/- 25.9	+/- 31.0	+/- 36.1	45
10 min	66.8	86.8	100.0	116.7	129.1	141.4	45
	+/- 6.1	+/- 10.2	+/- 13.8	+/- 18.6	+/- 22.2	+/- 25.9	45
15 min	55.5	72.7	84.0	98.4	109.1	119.6	46
	+/- 5.2	+/- 8.7	+/- 11.7	+/- 15.8	+/- 18.9	+/- 22.0	46
30 min	36.9	47.3	54.2	62.9	69.3	75.8	46
	+/- 3.1	+/- 5.3	+/- 7.1	+/- 9.6	+/- 11.5	+/- 13.4	46
1 h	22.6	28.5	32.5	37.5	41.3	45.0	46
	+/- 1.8	+/- 3.0	+/- 4.1	+/- 5.5	+/- 6.6	+/- 7.7	46
2 h	13.8	17.9	20.6	24.0	26.5	29.0	46
	+/- 1.2	+/- 2.0	+/- 2.8	+/- 3.7	+/- 4.5	+/- 5.2	46
6 h	6.3	8.1	9.4	10.9	12.0	13.2	45
	+/- 0.6	+/- 0.9	+/- 1.3	+/- 1.7	+/- 2.0	+/- 2.4	45
12 h	3.7	4.8	5.6	6.6	7.3	8.0	44
	+/- 0.3	+/- 0.6	+/- 0.8	+/- 1.1	+/- 1.3	+/- 1.5	44
24 h	2.1	2.7	3.1	3.6	4.0	4.3	46
	+/- 0.2	+/- 0.3	+/- 0.4	+/- 0.5	+/- 0.7	+/- 0.8	46

Table 3 : Interpolation Equation / Équation d'interpolation: $R = A \cdot T^B$

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)

RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)

T = Rainfall duration (h) / Durée de la pluie (h)

Statistics/Statistiques	2	5	10	25	50	100
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans
Mean of RR/Moyenne de RR	33.1	43.0	49.6	57.9	64.0	70.1
Std. Dev. /Écart-type (RR)	31.6	41.3	47.7	55.8	61.9	67.8
Std. Error/Erreur-type	7.6	9.6	10.9	12.6	13.9	15.1
Coefficient (A)	20.5	26.5	30.5	35.5	39.3	43.0
Exponent/Exposant (B)	-0.676	-0.677	-0.678	-0.679	-0.679	-0.679
Mean % Error/% erreur moyenne	8.5	8.1	8.0	7.8	7.7	7.7

APPENDIX C

Storm Sewer Design Sheets

STORM SEWER DESIGN SHEET

Peak Runoff Estimate by Rational Method

$Q = \frac{1}{360} C i A$

Where:

Q =

Peak Flow in cms

C =

Runoff Coefficient

i =

Rainfall Intensity in mm/hr

A =

Area in hectares

Intensity for Belleville

$i = A \cdot T_c^B$

Where:

i =

Rainfall Intensity in mm/hr

T_c =

Time of Concentration in hours

10-Year Parameters

A =

30.4

B =

-0.678

Pipe Capacity by Manning's Equation

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

Where:

A =

area of pipe in m²

R =

Hydraulic radius = A / P

P =

Wetted perimeter

S =

Slope (m/m)

n =

Manning's friction coef.

Check

$q \leq Q$

$V \leq 6 \text{ m/s}$

LOCATION			PEAK FLOW CALCULATION									PROPOSED SEWER									
LOCATION	FROM	TO	CATCHMENT AREAS				R.C. x A	CUM. R.C x A	TIME OF CONCENTRATION	INTENSITY	PEAK FLOW	Pipe Size	Length	Type of Pipe	Grade (use m/m)	Capacity, n = 0.013	Full Flow Velocity	Time of Flow	Actual Velocity at Q _d	Check Capacity	q/Q
			RUNOFF COEFFICIENT																		
			0.25	0.4	0.6	0.9	ha	ha	min	mm/hr	m ³ /s	(mm)	(m)	(%)	(m ³ /s)	(m/s)	min	(m/s)			
	MH2a	MH2					0.00	0.00	20.0	64.0	0.000	750	31.0	RCP	0.50%	0.787	1.78	0.29	0.00	OK	
102, 103	MH2	MH4			5.65		3.39	3.39	20.0	64.0	0.603	750	71.7	RCP	1.80%	1.494	3.38	0.35	3.19	OK	0.40
106	MH4	MH5				0.78	0.70	4.09	20.4	63.3	0.719	750	30.8	RCP	0.50%	0.787	1.78	0.29	2.02	OK	0.91
	MH5	MH5a					0.00	4.09	20.6	62.7	0.712	900	13.4	RCP	0.50%	1.280	2.01	0.11	2.06	OK	0.56
	MH5a	MH6					0.00	4.09	20.8	62.4	0.710	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	0.55
100, 101, 105	CB3	CB4	5.71	7.67		1.57	5.91	5.91	30.0	48.6	0.798	750	63.0	RCP	1.00%	1.113	2.52	0.42	2.74	OK	0.72
108	CB4	MH6				1.21	1.09	7.00	30.4	48.2	0.937	750	11.0	RCP	1.00%	1.113	2.52	0.07	2.83	OK	0.84
107	MH6	MH7				0.53	0.48	11.57	30.5	48.1	1.546	1066	53.8	RCP	1.00%	2.843	3.19	0.28	3.24	OK	0.54
110	MH7	MH8				0.40	0.36	11.93	30.8	47.8	1.584	1066	57.1	RCP	1.40%	3.364	3.77	0.25	3.70	OK	0.47
109	CB9	MH8		2.04			0.82	0.82	20.0	64.0	0.145	375	84.8	RCP	3.00%	0.304	2.75	0.51	2.71	OK	0.48
111	MH8	MH9				1.62	1.46	14.20	31.0	47.5	1.875	1066	38.9	RCP	0.50%	2.010	2.25	0.29	2.56	OK	0.93
	MH9	MH9a					0.00	14.20	31.3	47.2	1.864	1050	18.8	RCP	1.00%	2.731	3.15	0.10	3.38	OK	0.68
112	DDI1	MH9a				1.26	1.13	1.13	20.0	64.0	0.202	900	55.0	RCP	0.50%	1.280	2.01	0.46	1.47	OK	0.16
	MH9a	MH10					0.00	15.33	31.4	47.1	2.008	1066	75.0	RCP	0.60%	2.202	2.47	0.51	2.79	OK	0.91
	MH10	MH11					0.00	15.33	31.9	46.6	1.987	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.56	OK	0.99
113, 114	MH11	MH12				0.74	0.67	16.00	32.2	46.4	2.062	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.55	Surch	1.03
	MH12	Gate Chamber					0.00	16.00	32.4	46.1	2.050	1066	5.0	RCP	0.50%	2.010	2.25	0.04	2.55	Surch	1.02
104, 115	900 CSP	Gate Chamber			10.95		6.57	6.57	20.0	64.0	1.168	900	191.0	CSP	0.50%	0.693	1.09	2.92	#REF!	Surch	1.69
															(n = 0.024)						
	Gate Chamber	Holding Pond					0.00	22.57	32.5	46.1	2.890	1066	100.0	RCP	0.50%	2.010	2.25	0.74	#REF!	Surch	1.44
MTO Chart 2.30 shows a pipe will convey 7% beyond Manning full flow																					



CORNERSTONE BUILDERS LTD.

Cornerstone Builders Ltd.

195 Bellevue Drive

Belleville, ON, K8N 4Z5

Ph. 613-968-3501

Fx. 613-968-4986

Designed:

Bryon Keene, P.Eng.

Checked:

Peter Yee, P.Eng.

Date:

Sunday, July 1, 2018

Project:

Lehigh Cement

Prince Edward County

[illegible]

STORM SEWER DESIGN SHEET - 2026 PROPOSED UPGRADES

Peak Runoff Estimate by Rational Method

Q= 1/360 C i A

Where:

- Q = Peak Flow in cms
- C = Runoff Coefficient
- i = Rainfall Intensity in mm/hr
- A = Area in hectares

Intensity for Belleville

i= A*Tc^B

Where:

- i = Rainfall Intensity in mm/hr
- T_c = Time of Concentration in hours

10-Year Parameters

- A = 30.5
- B = -0.678

Manning's Coef

- CSP 0.024
- RCP/PVC 0.013

Where:

- A = area of pipe in m²
- R = Hydraulic radius = A / P
- P = Wetted perimeter
- S = Slope (m/m)
- n = Manning's friction coef.

Check

- q ≤ Q
- V ≤ 6 m/s

LOCATION			PEAK FLOW CALCULATION										PROPOSED SEWER									
LOCATION	FROM	TO	CATCHMENT AREAS					R.C. x A ha	CUM. R.C x A ha	TIME OF CONCENTRAT ION min	INTENSITY mm/hr	PEAK FLOW m³/s	Pipe Size (mm)	Length (m)	Type of Pipe	Grade (use m/m) (%)	Capacity, n = 0.013 (m³/s)	Full Flow Velocity (m/s)	Time of Flow min	Actual Velocity at Q _d (m/s)	Check Capacity	q/Q
			RUNOFF COEFFICIENT																			
			0.25	0.4	0.6	0.63	0.9															
	MH2a	MH2						0.00	0.00	20.0	64.2	0.000	750	31.0	RCP	0.50%	0.787	1.78	0.29	0.00	OK	
102, 103	MH2	MH4			5.65			3.39	3.39	20.0	64.2	0.605	750	71.7	RCP	1.80%	1.494	3.38	0.35	3.19	OK	0.40
106	MH4	MH5					0.78	0.70	4.09	20.4	63.5	0.722	750	30.8	RCP	0.50%	0.787	1.78	0.29	2.02	OK	0.92
	MH5	MH5a						0.00	4.09	20.6	62.9	0.715	900	13.4	RCP	0.50%	1.280	2.01	0.11	2.06	OK	0.56
	MH5a	MH6						0.00	4.09	20.8	62.6	0.712	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	0.56
100, 101, 105	CB3	CB4	5.71	7.67			1.57	5.91	5.91	30.0	48.8	0.801	750	63.0	RCP	1.00%	1.113	2.52	0.42	2.74	OK	0.72
108	CB4	MH6					1.21	1.09	7.00	30.4	48.3	0.940	750	11.0	RCP	1.00%	1.113	2.52	0.07	2.83	OK	0.84
107	MH6	MH7					0.53	0.48	11.57	30.5	48.3	1.551	1066	53.8	RCP	1.00%	2.843	3.19	0.28	3.24	OK	0.55
110	MH7	MH8					0.40	0.36	11.93	30.8	48.0	1.589	1066	57.1	RCP	1.40%	3.364	3.77	0.25	3.71	OK	0.47
109	CB9	MH8		2.04				0.82	0.82	20.0	64.2	0.146	375	84.8	RCP	3.00%	0.304	2.75	0.51	2.72	OK	0.48
111	MH8	MH9					1.62	1.46	14.20	31.0	47.7	1.882	1066	38.9	RCP	0.50%	2.010	2.25	0.29	2.56	OK	0.94
	MH9	MH9a						0.00	14.20	31.3	47.4	1.870	1050	18.8	RCP	1.00%	2.731	3.15	0.10	3.39	OK	0.68
112	DDI1	MH9a					1.26	1.13	1.13	20.0	64.2	0.202	900	55.0	RCP	0.50%	1.280	2.01	0.46	1.47	OK	0.16
	MH9a	MH10						0.00	15.33	31.4	47.3	2.015	1066	75.0	RCP	0.60%	2.202	2.47	0.51	2.80	OK	0.91
	MH10	MH11						0.00	15.33	31.9	46.8	1.993	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.56	OK	0.99
113, 114	MH11	MH12					0.74	0.67	16.00	32.2	46.5	2.068	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.55	Surch	1.03
	MH12	Gate Chamber						0.00	16.00	32.4	46.3	2.057	1066	5.0	RCP	0.50%	2.010	2.25	0.04	2.55	Surch	1.02
104, 115	900 CSP	Gate Chamber				10.95		6.90	6.90	18.8	67.0	1.284	1066	191.0	CSP	0.50%	1.089	1.22	2.61	#REF!	Surch	1.18
																	(n = 0.024)					
	Gate Chamber	Holding Pond						0.00	22.90	32.5	46.2	2.942	1200	100.0	RCP	0.50%	2.757	2.44	0.68	2.80	Surch	1.07
												MTO Chart 2.30 shows a pipe will convey 7% beyond Manning full flow										



Jewell Engineering Inc
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Designed: Duncan Burgess, BASc
Checked: Bryon Keene, P.Eng.
Date: Wednesday, March 18, 2026

Project: Heidelberg Materials
Picton Cement Plant Improvements

APPENDIX D

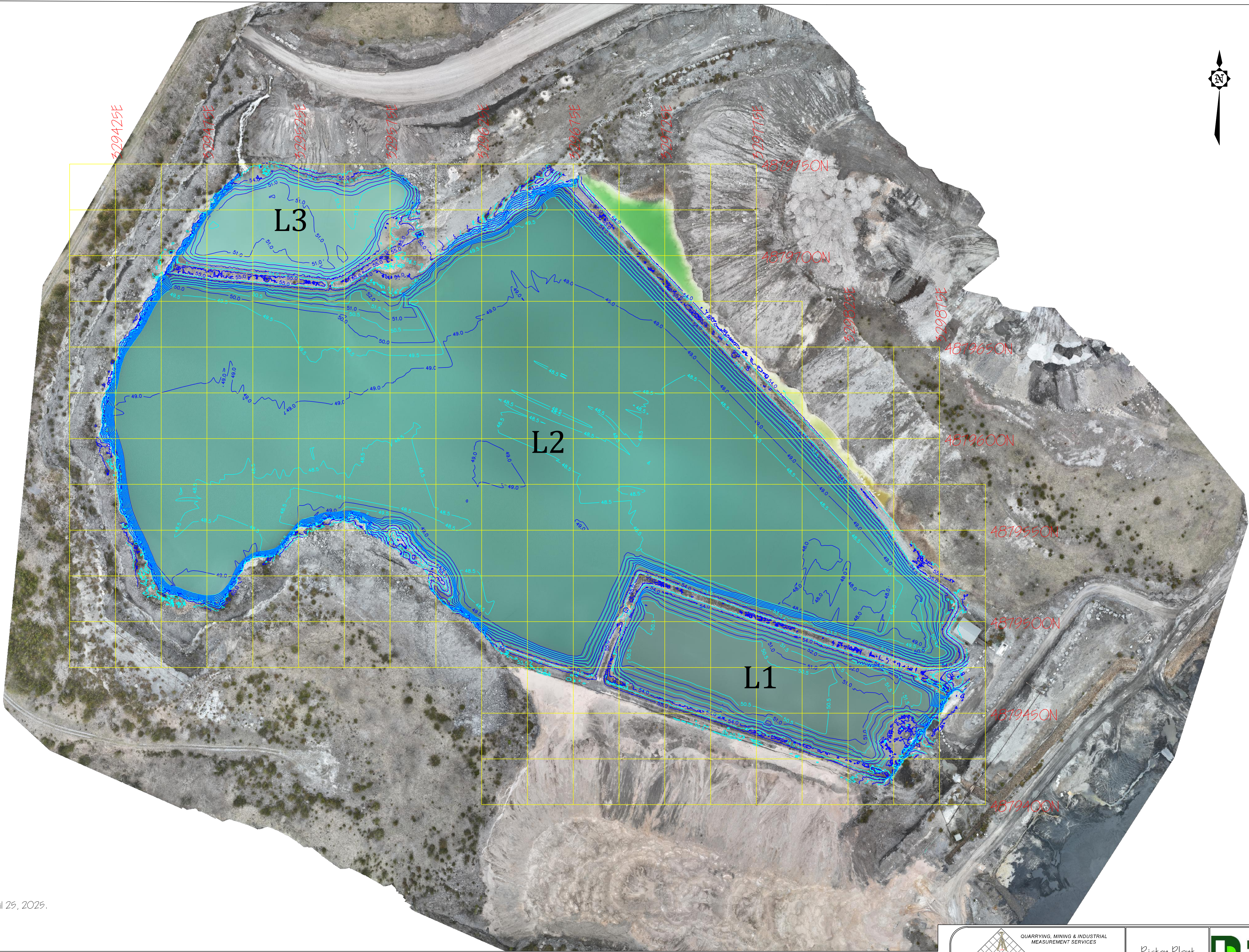
2025 Surveys



SEWER INVERT DATA TABLE									
MAN/CD	DIRECTION	DIAMETER	INVERT	TOP OF JUV. OR GRADE ELEV.	INVERT ELEVATION	NOTES			
CB#1	SW	300	1.27	98.69	97.42				
CB#2	NW	1500	0.98	98.92	97.94	CAPPED			
	SW	1500	0.98	98.92	97.94	CAPPED			
CB#3	NE	200	0.99	99.29	98.30				
	SW	200	0.99	99.29	98.30				
DCB#4	NE	150	1.07	98.28	97.21				
	SW	500	1.13	98.28	97.15				
CB#5	<0°	N/A	N/A	100.11	N/A	FILLED WITH ICE			
CB#6	<0°	N/A	N/A	100.15	N/A				
CB#7	NW	300	0.65	99.80	99.11				
	NE	300	0.74	99.10	98.25	RECESSED			
CB#8	SE	N/A	N/A	99.73	N/A	FILLED WITH DEBRIS			
	SW	N/A	N/A	99.73	N/A	DB=0.78			
CB#9	N	100	0.84	98.66	97.55	660'w DB=0.58			
	NE	200	0.96	97.78	97.18				
	SW	200	0.84	97.94	97.34				
CB#10	NE	300	1.16	99.38	98.22				
CB#11	N/A	N/A	N/A	99.34	N/A	UNABLE TO SEE			
CB#12	W	200	0.72	100.15	99.43	FILLED WITH WATER			
CB#13	SE	300a	1.09	105.49	104.40				
CB#14	NE	300	N/A	99.30	N/A	FILLED WITH DEBRIS			
STM WH#1	NW	200	N/A	99.17	N/A	UNABLE TO OPEN			
	SE	250	N/A	N/A	N/A				
STM WH#2	NW	100	0.60	99.84	99.24	BOC=1.86			
	SE	75	0	98.54	98.54				
STM WH#3	NE	300	1.84	98.79	96.95				
	SW	400	2.25	98.79	96.95				
STM WH#4	NW	1000	N/A	97.38	N/A	RECESSED			
	SE	1000	N/A	97.38	N/A				
STM WH#5	NW	1000	3.45	96.58	93.14				
	SE	1000	3.45	96.58	93.14	OUTFALL SEWER			
STM WH#6	NE	600a	2.76	94.28	91.30				
	SW	600a	2.82	94.28	91.30				
STM WH#7	NE	150	0.88	99.78	98.92				
	E	300	N/A	N/A	N/A	RECESSED			
STM WH#8	NE	250	N/A	99.20	N/A				
	E	250	N/A	N/A	N/A				
STM WH#9	NW	N/A	N/A	99.70	N/A	BROKEN PIPE, RECESSED			
	SE	N/A	N/A	99.70	N/A	RECESSED			
SAN WH#1	<0°	400	N/A	99.45	N/A				
	SE	600	1.81	97.76	97.94				
SAN WH#2	<0°	400	N/A	100.52	N/A				
	S	200	3.03	97.78	97.40				
SEPTIC TANK	SE	700	2.07	96.79	94.72				

Available Volume from 54.945m elevation to current Depths

Pond #	Volume (m³)	DATE
L1	33,040	Apr. 2025
L2	395,580	Apr. 2025
L3	18,100	Apr. 2025



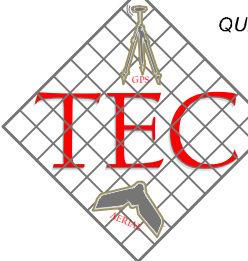
LEGEND

- Contours (April 25, 2025)

NOTES:

- 1.) All elevations are in metres.
- 2.) Coordinates are UTM NAD83.
- 3.) Aerial image and sonar data collected April 25, 2025.

REV #	DESCRIPTION	DATE
ROA	Initial Issue	Apr. 2025



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DRAWN BY:	BA	DATE:	May 1 / 25
CHECKED BY:	JP	DATE:	May 2 / 25
SURVEYED BY:	JP & BA	DATE:	Apr. 25 / 25

Pictou Plant



Ponds

SCALE:	25 m X 25 m	PROJ. NO.:	H-Ponds-01	ROA:	
SCALE:	1 : 1000				

SHEET 1 OF 1

APPENDIX E

1996 Water Management Plan Excerpts – Water and Environmental Services Ltd.

1.3 Previous Reports by Others

The Canadian Portland Cement Association contracted Beak Engineering Ltd. in 1991 to complete an industry-wide study to evaluate demonstrated technologies applicable for the management of suspended solids discharges from cement plants in Ontario. This study was initiated in anticipation of MISA regulations for the Industrial Minerals Sector. Beak completed a single composite report for all of the sites which included information on MISA data, an evaluation of effluent settleability (column settling tests), water balances, and an evaluation of various treatment technologies.

The results of effluent settling tests for the ESSROC site indicated that significant settling occurred after 24 hours of detention. A simulated sample of cement plant effluent having an initial TSS concentration of 24,000 mg/L had a 24 hour TSS concentration of 62 mg/L. A simulated sample of quarry water effluent having an initial TSS concentration of 27,000 mg/L had a TSS concentration of <2 mg/L after 24 hours of detention. Beak determined that a mean settling time of 24 hours is required to achieve a TSS Long Term Average (LTA) of 12 mg/L and a 95th percentile concentration of 15 mg/L.

Effluent limits of TSS for the Industrial Minerals Sector had not yet been determined when Beak undertook their study; therefore, they presented four treatment technologies to accommodate various effluent limits and process handling scenarios. The facilities common to the four technologies included construction of a settling pond in the south quarry for treatment of the combined stormwater, process cooling water and quarry effluent; installation of a new dewatering system in the south quarry; construction of a diversion chamber on the existing effluent sewer and a gravity sewer to direct flow to the settling pond; and drainage improvement works in and around the plant site.

Technologies 1, 2 and 3 proposed the discharge of the settling pond effluent to the lake. However, Technology 3, incorporated a sand filter on the discharge line from the quarry to further improve effluent quality. Technology 4 incorporated a flow diversion chamber on the discharge line with a pump and forcemain to direct flow to the existing pumphouse for recirculation. A percentage of the effluent would be discharged to the lake and, therefore, a smaller sand filter was proposed. Also, it was assumed that make-up water would continue to be supplied from the lake and that no cooling tower would be required.

A comparison of the four technologies is presented in Table 1-1.

5.2.5 TSS 28 and TSS 30

Water samples collected from these locations during a rain event on May 11, 1995 underwent a general surface water suite analysis (refer to Table 5-6). This additional testing was performed to characterize the chemical impacts on surface water caused by the application of Domtar Road Binder (DRB) dust suppressant. Sample TSS 28 contained very high levels of TSS (4550 mg/L) and high levels of TDS. The sample at TSS 30 had very high levels of alkalinity (2600 mg/L), sulphate (997 mg/L), and potassium (2220 mg/L). The concentration of these parameters at TSS 28 were significantly lower than at TSS 30, but above drinking water objectives. Conductivity values were high at both TSS 30 (14500 $\mu\text{S}/\text{cm}$) and TSS 28 (3800 $\mu\text{S}/\text{cm}$). The pH values at both locations were high, indicating the presence of strong bases which result in low hardness concentrations (< 100 mg/L of CaCO_3). Precipitation of carbonates or ion exchange result in lower dissolved calcium and magnesium levels. Based on the chemical data, it appears that the use of DRB for dust control has a negative impact on surface water quality.

5.2.6 Storm Event Sampling

Surface water samples were collected from approximately 20 locations around the plant site on four different occasions during storm events. These samples were analyzed for TSS content in order to define the areas of the site that are producing significant TSS during storm events. The results are presented in Table 5-7, and the sampling locations are shown on Drawing 2.

The results indicate that the highest concentrations of TSS are generated from the coal/coke storage area, where a TSS concentration of 29,900 mg/L was measured during a storm event in April, 1996. On August 4, 1995, high TSS concentrations were also generated in the vicinity of the maintenance and machine shed and changehouse (1870 - 8530 mg/L). The clinker storage area is generating elevated TSS during storm events as indicated by sample TSS 101 (36 - 795 mg/L). The coal silo area generates significant concentrations of TSS as revealed by samples TSS 100 (143 - 1050 mg/L) and TSS 28 (130 - 4550 mg/L). Seepage from the clinker pile near the dock into Picton Bay (TSS 102) also contains elevated TSS (100 - 603 mg/L).

5.3 Recommendations for Water Quality Improvement

To ensure MISA regulations are adhered to, treatment of the quarry and plant process water is required prior to discharge into the bay. Treatment is also required to enable re-use of plant process

water.

Rather than attempting to treat each process stream individually, the most cost effective and technically feasible approach involves diverting all of the process/stormwater streams to a common treatment area. The most obvious scenario involves diverting water from the outfall and north quarry into the south quarry, and using either a portion or all of the south quarry as a settling pond.

The quality of water resulting from the mixing of the various process and storm streams is impossible to predict. However, given the elevated levels of dissolved calcium carbonate in water from the north quarry and storm sewer, it is quite possible that the carbonate will need to be precipitated and allowed to settle in the pond so that precipitation does not occur in pipes during recirculation. Carbonate precipitation is very temperature sensitive, with precipitation usually increasing with higher temperatures. Enhanced precipitation can be achieved by spraying the water or volatilizing it in some way so that CO_2 is liberated. This can be accomplished by mechanical spraying or aeration of the pond water. The effluent from the settling pond should be analyzed for dissolved carbonates after the system is operational in order to determine if additional volatilization is required.

The presence of elevated counts of bacteria in the storm sewer outfall may cause biofouling problems in piping during recirculation. Bacteriological monitoring of the settling pond effluent should be performed once the system is operational. It is likely that the sewage treatment plant is the source of bacteria in the storm sewer, therefore, efforts should be made to optimize the treatment efficiency of the sewage treatment plant.

The concentrations of TSS in the water entering the south quarry settling pond will be elevated during storm events, however, the results of column settling tests completed by Beak during an earlier study indicate that the TSS levels will be reduced to well below the MISA effluent limits after 24 hours of settling.

APPENDIX F

2018 SWM Report – Cornerstone Builders Ltd.

Lehigh Heidelberg

1370 County Road 49, PEC

Stormwater Management Report

C7913

July 1, 2018



FAX: 613-968-4986

general@cornerstonebuilders.ca

PH: 613-968-3501

***Lehigh Heidelberg Cement Company
Stormwater Management Report***

Revision Notes

The July 2018 version is the initial submission.

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Appendix B	Hydrologic Calculations
Appendix C	OTTHYMO Model Output
Appendix D	STORM WATER MANAGEMENT DRAWINGS SP01 - SP07

1 Background

Lehigh Heidelberg operates a Cement manufacturing facility on County Road 49, north of Picton (see Figure 1-1). The plant is situated between County Road 49 and Picton Bay. Quarries are located on the west and east sides of County Road 49.

A portion of the east quarry was repurposed into a water management pond under the Municipal Industrial Strategy for Abatement (MISA) program. Process waters are collected through the plant and drain into the water management pond. There, water is clarified through settling and is eventually pumped back through the site to the plant for reuse.

Stormwater drainage is mixed with the process waters sharing the same sewer network which collects water by gravity and drains it back into the quarry. The surface of the water management facility is below the elevation of Picton Bay and thus, infiltration and stormwater inputs generate a surplus of water. Clarified surplus waters are pumped from the water management facility to Picton Bay through a permit to take water.



Figure 1-1: Lehigh Heidelberg Site Location

Lehigh Heidelberg Cement Company
Stormwater Management Report

Lehigh has made significant alterations in recent years to improve water quality leaving the site and also to reduce fugitive dust emissions.

Cornerstone Builders Limited was retained by Lehigh Heidelberg Cement to assist with planned facility improvements that will replace two large granular surface areas with a concrete surface. This will help to control dust and improve the quality of stormwater runoff. The granular surfaces are difficult to maintain and cannot be swept. The concrete surfaces provide a hardened base to permit the sweeping vehicles to keep the work areas clean.

2 Purpose of the Report

CBL prepared the stormwater management report to assist with design of drainage for the proposed concrete surfaces. The report authors have investigated the storm sewer network, overall drainage patterns and have made recommendation herein for grading of the new concrete areas and for improvements to the sewer network.

Three concrete pads are proposed. The first is under Kiln #3 – area = 1,538m². The second is under Kiln #4 – area = 2,800m². The last location is along Road #3 – area = 2,361m². The concrete paving will replace very poorly drained areas with high imperviousness.

3 Background Documents

Cornerstone Builders Ltd reviewed the following documents in the preparation of this report:

- Water Management Plan, Bluemetric Environmental, Final Report, May 2016 (embedded report is circa 1996)
- Site Plan, JL Richards, 1995
- Existing Services, JL Richards, 1995
- South Quarry Layout, JL Richards, 1995
- Storm Drainage Areas, JL Richards, 1995
- Stormwater Management Site Improvements, JL Richards, 1996
- Stormwater Management Plan Drainage Areas, JL Richards, 1996

4 Storm Sewer Design

Proposed site improvements will increase the percentage of hard surfaces and may increase the peak runoff. CBL has prepared a storm sewer design and completed a review of the water management

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Stormwater Management Report

facility to understand the impact of the proposed site modifications. The discussion of the water management facility is provided in Section 5.

The site is gently sloped and drains from County Road 49 easterly to Picton Bay. Native soils have been significantly disturbed over large portions of the site and where remaining soils are shallow over the underlying limestone bedrock.

The 1996 WESA report indicates the existing 1050mm storm sewer system was sized to convey the 10-yr runoff event plus process waters. No supporting documentation was contained in the study with the exception of catchment drawings and proposed sewer data. CBL found useful information in the proposed sewer drawings, but opted to prepare more detailed contour mapping and sewer catchment drawings to better understand the site drainage. Despite the intensive campaign, several sewer sections could not be exposed or investigated due to site conditions.

Storm sewers were designed to flow under gravity flow conditions following the Ministry of the Environment's Design Guidelines for Sewage Works, 2008. Peak flows are calculated using the Rational Method and flow capacities are determined using Manning's formula.

The Rational Method is described in many documents (including the MTO Drainage Manual, 1997) as a hydrologic method for determining the peak flows from catchment areas. It is commonly used for the design of storm sewers. The Rational Method equation is shown below. It relies on an estimation of runoff coefficient, peak flow intensity and drainage area.

Equation 1: Rational Method Equation

$$Q = \frac{1}{360} CiA$$

Where:

Q = Peak Flow in cms

C = Runoff Coefficient

i = Rainfall Intensity in mm/hr

A = Area in hectares

Each of the inputs are discussed below.

4.1.1 Rainfall Intensity

In the Rational Method, the intensity of rainfall is set to the time of concentration of the contributing area. Time of concentration is discussed later.

Precipitation intensities (i) are estimated using intensity-duration-frequency curves that are maintained by Environment Canada. CBL used the City of Belleville (6150689) IDF curves in the development of the Rational Method design sheets. Belleville IDF curves are published by Environment Canada and are reproduced in Appendix A.

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Stormwater Management Report

The Intensity Duration Frequency curves for. The 10-yr coefficients for A and B are shown in column 4 of Table 4-1. These solve for Intensity using Equation 2.

Equation 2: Solving for Intensity using IDF Values

$$i = A \times T_c^B$$

Where:

I = Intensity in mm/hr

T_c = Time of Concentration in hours

A and B = coefficients per Table 4-1.

Table 4-1: IDF Coefficients for Belleville

Coefficient	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
A	20.3	26.4	30.4	35.5	39.3	43
B	-0.677	-0.677	-0.678	-0.678	-0.678	-0.678

The time-intensity pairs are reproduced below in Table 4-2.

Table 4-2: Intensity Duration Frequency Curves for Belleville

Duration	Intensity (mm/hr) for Given Frequency					
	2 Yr	5 Yr	10 Yr	25 Yr	50 Yr	100 Yr
5 Min	89.5	132	160.1	195.7	222.1	248.3
10 Min	63.6	85.9	100.7	119.4	133.2	147
15 Min	49.6	66.7	78.1	92.4	103.1	113.7
30 Min	31.2	41.3	48	56.4	62.7	68.9
1 H	19.1	27.6	33.3	40.4	45.7	50.9
2 H	11.8	16.9	20.4	24.7	27.9	31
6 H	5.7	7.4	8.6	10.1	11.2	12.3
12 H	3.3	4.2	4.8	5.5	6	6.6
24 H	1.9	2.4	2.7	3.1	3.3	3.6

Time of Concentration

The Rational Method requires a determination of the time it takes for the drainage area to contribute to the peak flows. This is called the time of concentration. The initial time of concentration of the upstream inlet was determined using the Airport Formula following the methodology described in MTO Drainage Manual. The time of concentration is adjusted at downstream pipes by adding the time of flow in the sewer. As the time of concentration increases the intensity is reduced.

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Time of concentration is determined following the Airport Method that is presented in the Ministry of Transportation Drainage Manual (Ontario Ministry of Transportation, 1997). The Airport Method is commonly used for lands with runoff coefficient less than 0.4. But, it can also represent areas with higher runoff coefficients that are dominated by overland flows.

The Airport Method Formula is given by Equation 5.

Equation 3: Airport Method of Determining Time of Concentration

$$T_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{0.33}}$$

Where:

T_c = Time of concentration (min)

C = runoff coefficient (dimensionless)

L = Length of the flow path (m)

S_w = Slope of the watershed (%)

4.1.2 Catchment Areas

Catchment areas are shown in drawing SP01, SP02 and SP04. Areas are resolved by interpreting topographic data and considering drainage features. Topographic information derived from LiDAR was purchased from Quinte Conservation. LiDAR data was resolved to 1m and the 1m contours are shown on the CBL drawings. CBL also undertook several detailed site surveys in the areas where concrete surfaces are proposed.

Catchments are provided in Table 4-3:

Table 4-3: Catchment Areas

Catchment	Area (hectares)
100	5.71
101	7.67
102	2.31
103	3.31
104	9.75
105	1.57
106	0.78
107	0.53
108	1.21
109	2.04
110	0.40
111	1.62
112	1.26

113	0.31
114	0.43
115	1.20

4.1.3 Runoff Coefficient

The runoff coefficient is a dimensionless factor between 0 and 1 and represents the proportion of precipitation that leads to direct runoff. The time of concentration varies with runoff coefficient. The runoff coefficient used above is calculated using a weighted average and the values provided in MTO Chart 1.07 (Ontario Ministry of Transportation, 1997). Equation 3 is used to calculate the weighted runoff coefficient.

Equation 4: Weighted Runoff Coefficient

$$C_w = \frac{C_1 \times A_1 + C_2 \times A_2 + \dots C_n \times A_n}{A_1 + A_2 + \dots A_n}$$

Where:

C_i = Runoff Coefficient for A_i

A_i = Land Area (ha) of Cover Type

A detailed calculation was completed for catchment 101 that represents a governing catchment that most influences the time of concentration. The runoff coefficient was estimated to be 0.40. The calculations are provided in Appendix B

4.1.4 Results

The storm sewer network was analyzed using the 10-yr frequency. A time of concentration of 34.5 minutes was estimated using the Airport Method. CBL used 30 minutes for conservatism. Both the existing network and sewer improvements were analyzed. The storm sewer design sheets are contained in Appendix B.

The existing storm sewer network was found to be adequately sized for the main trunk. However, three pipes are undersized. These are CB9 to MH8, CB3 to CB4, and CB4 to MH6. Each is discussed below.

Proposed Storm Sewer Improvements

1. CB9 to MH8 is a 300mm CSP that has a q/Q of 1.6. A value greater than 1 is surcharged. The CSP has a higher friction factor (0.024 vs 0.013 for concrete). It is recommended this sewer be upgraded to a 375mm PVC DR35 pipe. **The improvement is recommended to proceed in conjunction with the 2018 concrete improvements.**
2. CB3 to CB4 is a 600mm RCP that has a q/Q of 1.3. This should be improved to a 750mm PVC DR35 pipe. This sewer can be replaced in a later phase of work.

3. CB4 to MH6 is a 600mm RCP with a q/Q of 1.52. This should be improved to a 750mm PVC DR35 pipe. This sewer can be replaced in a later phase of work.

The 900mm CSP that received flows from the north is also undersized, but this is based on assumed slopes. Should the pipe be undersized, it would likely still perform satisfactorily as it would impose storage in the ditch system. No improvements are recommended on this pipe.

The pipe leaving the gate chamber is not well understood. It has been analyzed assuming it is a 1050 (1066mm) pipe. It is possible the pipe is a 1200mm pipe which would perform satisfactorily.

Drainage Improvements

Additional inlet capacity is needed in catchment 112 to reduce water ponding. The storm sewer is 900mm, which has excellent capacity. The single CB south of the Hammer Mill is proposed to be improved to a double ditch inlet. This will provide much more inlet capacity and will also resist clogging.

Three catch basins are recommended to be added in the vicinity of the proposed slab expansion Road #3 and Slab Expansion Under Kiln #4.

5 Water Management Facility

The water management facility will not be impacted by the proposed concrete improvements. The surface area of the site contributing to the facility is unchanged. The surface is hardened slightly with the concrete.

Drainage from these areas contributes mostly to surface runoff. The portion that infiltrates would generally flow toward Picton Bay following the groundwater gradient. However, the sewer system gravel bedding would intercept the groundwater flow and cut off some of this flow. Groundwater from the site discharges into the water management facility.

The water residence time in the water management facility was estimated by WESA to be 97 days. A settling period of 1 day is sufficient for the water quality targets to be met. In 1996, WESA noted the volume of water in the reservoir was not sufficient to provide the process water and water had to be pumped from Picton Bay. Additional water that could be generated from the proposed hardening would not be unwelcomed in the facility and would not negatively affect residence time.

The concrete paving will not negatively affect the water management facility with either added volume or with impact on residence time for water quality (TSS removal).

6 Conclusions

Lehigh Cement is proposing to harden three locations within their site. Three concrete paving areas are proposed:

- The first is under Kiln #3 – area = 1,538m².
- The second is under Kiln #4 – area = 2,800m².
- The third location is along Road #3 – area = 2,361m².

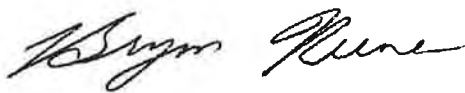
The concrete paving will improve site maintenance to reduce fugitive dust and improve stormwater quality. The impact on drainage was analyzed using detailed LiDAR topography and supported by site surveys with total station equipment. Storm sewer design sheets were prepared using the newly developed site statistics to analyze the sewer network. In general, CBL found the trunk sewer was sufficiently sized to handle the 10-yr flows. The exceptions were two lateral sewers. One is recommended for replacement in 2018 and the other sewer (CB3 to CB4 and CB4 to MH6) can be improved at a later date.

Additional catch basins are recommended as part of the concrete paving work to improve the drainage management and inlet capacity. Drainage improvements are recommended to be undertaken as part of the 2018 concrete paving.

The following improvements are recommended in 2018:

- 1) Replace the 300mm CSP from CB9 to MH8 with a 375mm PVC DR35 sewer.
- 2) Replace the CB in catchment 112 with a double ditch inlet.
- 3) Add three new CBs along the road slab.

Report Submitted by



Bryon Keene, P.Eng.

Peter Yee, P.Eng.



References

Ontario Ministry of the Environment. (2003). *Stormwater Management Planning and Design Manual*.
Queen's Printer for Ontario.

Ontario Ministry of Transportation. (1997). *Drainage Management Manual*.

APPENDIX A
IDF CURVES – STATION 6150689
BELLEVILLE

Lehigh Heidelberg Cement Company
Stormwater Management Report

Environment Canada/Environnement Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes
de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2012/02/09

=====

BELLEVILLE ON 6150689
(composite)
Latitude: 44 9'N Longitude: 77 23'W Elevation/Altitude: 76 m
Years/Années : 1960 - 2006 # Years/Années : 37

=====

Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1960	6.3	9.1	12.4	23.4	25.4	35.1	53.8	55.1	55.9
1961	6.1	7.9	8.9	12.2	18.0	18.8	23.9	34.0	36.3
1963	12.4	19.0	23.1	28.4	30.7	31.0	31.0	31.7	44.4
1964	4.3	5.6	7.4	12.2	12.4	20.6	45.2	45.7	45.7
1966	6.3	8.9	10.4	11.9	13.2	16.0	32.8	37.8	38.1
1967	7.4	9.9	10.4	10.4	11.9	13.2	26.4	42.4	58.7
1968	7.9	11.9	13.7	18.5	21.8	27.2	43.9	57.1	57.1
1969	5.8	9.7	13.0	17.5	24.4	31.0	37.8	43.2	62.2
1971	7.4	11.4	13.0	23.9	25.1	25.4	25.7	25.7	32.5
1972	9.4	10.7	11.7	12.4	14.7	20.6	28.2	33.5	50.5
1973	7.4	10.7	11.9	18.0	21.3	21.8	37.3	45.5	48.0
1974	10.9	15.2	17.8	25.4	25.4	25.4	34.3	42.7	42.7
1977	4.8	8.1	9.9	14.7	25.1	30.2	60.5	66.0	66.0
1980	13.2	16.9	19.0	20.5	20.5	34.6	46.9	47.6	59.6
1981	-99.9	-99.9	13.3	25.5	29.4	34.6	46.2	49.2	57.4
1982	4.6	8.5	10.1	14.2	18.3	24.7	39.8	45.0	45.0
1983	6.5	8.9	10.5	18.4	22.2	30.7	39.6	39.6	50.3
1984	5.1	8.1	10.1	11.3	19.7	23.7	33.4	51.4	55.1
1985	10.5	16.2	20.0	27.0	27.4	42.3	42.3	44.5	44.5
1986	9.1	14.4	16.4	23.2	25.2	35.0	59.2	68.8	78.9
1987	4.3	6.6	9.3	14.2	24.7	37.1	39.2	39.2	39.2
1988	3.7	6.2	7.4	8.6	9.2	10.6	20.8	22.2	28.2
1989	14.5	16.7	17.9	18.4	24.2	24.2	27.7	27.7	37.7
1990	6.9	8.3	10.0	12.3	13.6	20.8	29.7	34.8	38.7

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Stormwater Management Report

1991	8.5	13.8	18.5	18.8	32.0	32.4	32.4	32.4	32.7
1992	6.3	7.6	8.4	13.2	18.8	21.9	38.2	48.3	50.1
1993	8.2	15.8	23.6	28.3	28.3	28.3	-99.9	-99.9	69.6
1994	8.8	10.2	14.5	18.3	23.6	25.5	38.2	49.2	52.8
1995	8.0	12.9	14.9	19.3	27.5	31.5	48.5	58.5	67.3
1996	6.9	10.4	13.4	19.2	25.1	41.3	41.5	53.8	53.8
1997	10.3	16.8	20.9	25.5	42.8	50.0	56.0	56.0	56.0
1998	9.5	12.1	15.1	22.1	25.0	32.6	38.6	38.6	50.2
1999	9.6	13.1	17.9	23.2	29.4	36.9	42.8	72.7	72.7
2000	10.4	13.4	14.7	16.8	29.0	39.8	52.0	52.4	53.0
2001	7.4	10.1	11.0	11.8	16.7	17.4	21.2	31.6	39.8
2002	7.1	9.4	14.0	21.0	22.4	26.0	39.4	44.2	49.8
2003	7.6	13.5	20.1	26.2	27.0	27.0	31.1	-99.9	56.2
2004	14.4	22.1	28.8	33.3	33.3	49.0	89.9	114.4	124.5
2006	9.0	14.7	18.8	19.5	19.5	19.5	37.3	42.7	59.8
# Yrs. Années	38	38	39	39	39	39	38	37	39
Mean Moyenne	8.1	11.7	14.4	18.9	23.2	28.6	39.8	46.6	52.8
Std. Dev. Écart-type	2.7	3.8	4.9	6.0	6.7	9.0	13.0	16.3	16.4
Skew. Dissymétrie	0.69	0.66	0.83	0.29	0.20	0.37	1.60	2.08	2.19
Kurtosis	3.44	3.23	3.63	2.58	4.05	3.21	7.90	10.48	11.56

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount

Avertissement : la quantité maximale annuelle excède la quantité
pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
2004	6 h	89.9	80.5
2004	12 h	114.4	97.6
2004	24 h	124.5	104.4

Table 2a : Return Period Rainfall Amounts (mm)

Quantité de pluie (mm) par période de retour

Duration/Durée	2	5	10	25	50	100	#Years Années
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	
5 min	7.6	10.0	11.6	13.5	15.0	16.5	38
10 min	11.1	14.4	16.7	19.5	21.6	23.6	38
15 min	13.6	18.0	20.8	24.5	27.2	29.9	39
30 min	18.0	23.2	26.7	31.1	34.4	37.6	39
1 h	22.1	28.0	31.9	36.8	40.5	44.1	39
2 h	27.1	35.1	40.4	47.0	52.0	56.9	39
6 h	37.7	49.1	56.7	66.3	73.4	80.5	38

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Stormwater Management Report

12 h	44.0	58.3	67.8	79.9	88.8	97.6	37
24 h	50.1	64.7	74.3	86.5	95.5	104.4	39

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits
 Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

Duration/Durée	2	5	10	25	50	100	#Years Années
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	
5 min	91.6	120.0	138.8	162.6	180.2	197.7	38
	+/- 9.4	+/- 15.8	+/- 21.3	+/- 28.8	+/- 34.4	+/- 40.1	38
10 min	66.5	86.6	100.0	116.9	129.4	141.8	38
	+/- 6.7	+/- 11.2	+/- 15.1	+/- 20.4	+/- 24.4	+/- 28.5	38
15 min	54.4	71.8	83.4	97.9	108.7	119.5	39
	+/- 5.7	+/- 9.6	+/- 12.9	+/- 17.4	+/- 20.8	+/- 24.3	39
30 min	35.9	46.5	53.4	62.2	68.8	75.3	39
	+/- 3.4	+/- 5.8	+/- 7.8	+/- 10.5	+/- 12.6	+/- 14.7	39
1 h	22.1	28.0	31.9	36.8	40.5	44.1	39
	+/- 1.9	+/- 3.2	+/- 4.4	+/- 5.9	+/- 7.1	+/- 8.2	39
2 h	13.5	17.5	20.2	23.5	26.0	28.5	39
	+/- 1.3	+/- 2.2	+/- 3.0	+/- 4.0	+/- 4.8	+/- 5.6	39
6 h	6.3	8.2	9.5	11.1	12.2	13.4	38
	+/- 0.6	+/- 1.1	+/- 1.4	+/- 1.9	+/- 2.3	+/- 2.7	38
12 h	3.7	4.9	5.7	6.7	7.4	8.1	37
	+/- 0.4	+/- 0.7	+/- 0.9	+/- 1.2	+/- 1.5	+/- 1.7	37
24 h	2.1	2.7	3.1	3.6	4.0	4.4	39
	+/- 0.2	+/- 0.3	+/- 0.4	+/- 0.6	+/- 0.7	+/- 0.8	39

Table 3 : Interpolation Equation / Équation d'interpolation: $R = A \cdot T^B$

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)

RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)

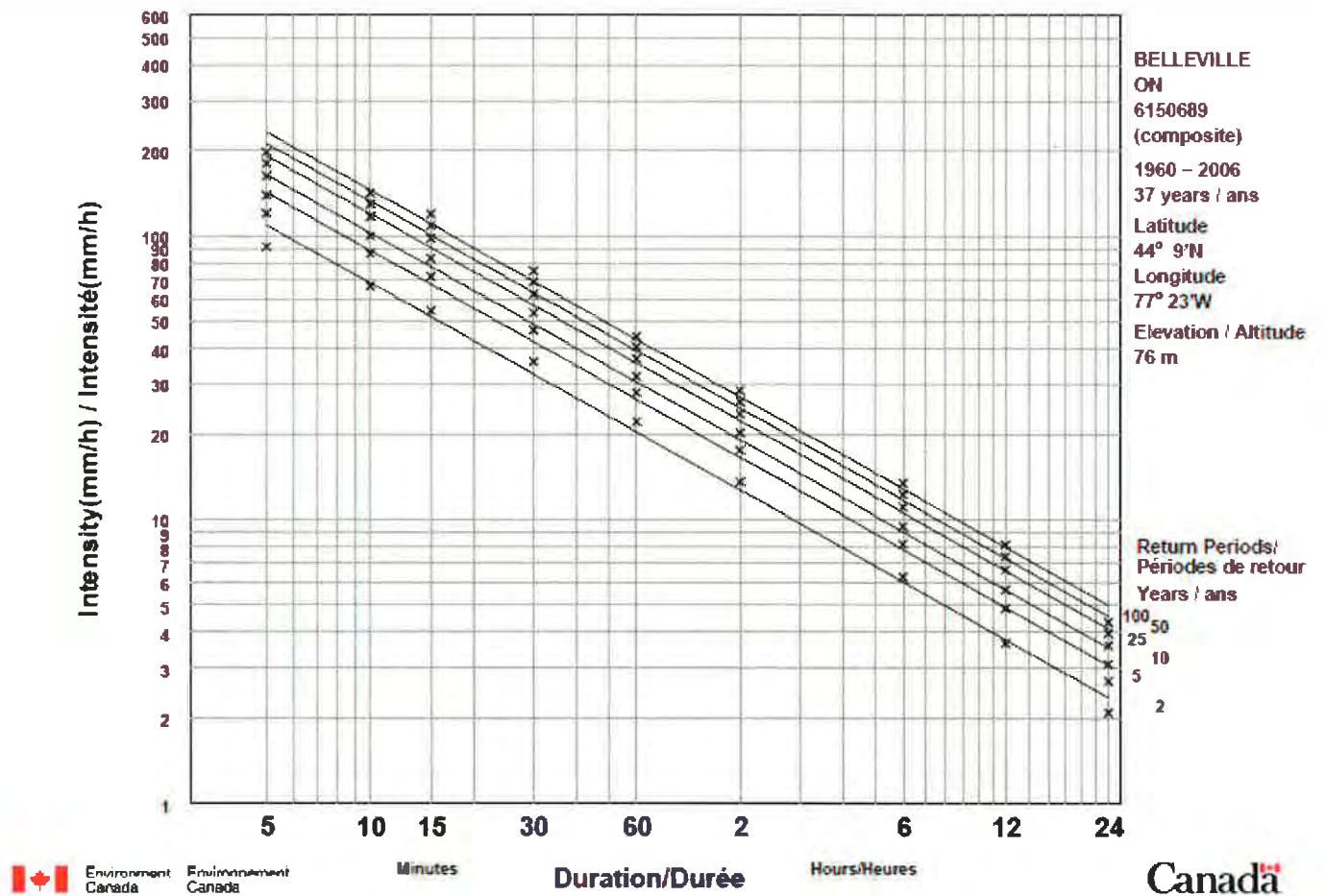
T = Rainfall duration (h) / Durée de la pluie (h)

Statistics/Statistiques	2	5	10	25	50	100
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans
Mean of RR/Moyenne de RR	32.9	42.9	49.5	57.9	64.1	70.3
Std. Dev. /Écart-type (RR)	31.7	41.6	48.1	56.4	62.5	68.6
Std. Error/Erreur-type	6.8	8.7	9.9	11.4	12.6	13.7
Coefficient (A)	20.3	26.4	30.4	35.5	39.3	43.0
Exponent/Exposant (B)	-0.677	-0.677	-0.678	-0.678	-0.678	-0.678
Mean % Error/% erreur moyenne	7.8	7.4	7.2	7.2	7.2	7.2

Short Duration Rainfall Intensity–Duration–Frequency Data

2012/02/09

Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée



**APPENDIX B
HYDROLOGIC CALCULATIONS**

Lehigh Heidelberg Cement Company
Stormwater Management Report

Runoff Coefficients
Weighted Area Method
Lehigh Cement

Bryon Keene, P.Eng.
 July 1, 2018

Cover Type	Land Area (ha)	
	Pre	Post
Grassed	7.67	5.57
Gravel	0.00	0.70
Asphalt/Conc	0.00	0.90
Building	0.00	0.50
Total Area	7.67	7.67

$$C_w = \frac{C_1 \times A_1 + C_2 \times A_2 + \dots C_n \times A_n}{A_1 + A_2 + \dots A_n}$$

Where:

C_i = Runoff Coefficient for A_i

A_i = Land Area (ha) of Cover Type

Imperviousness 23%

Return Period	Grassed		Gravel		Asphalt		Building		Weighted	
	Range/Adj *	Used	Range/Adj *	Used	Range/Adj *	Used	Range/Adj *	Used	Pre	Post
2	0.10 – 0.30	0.25	0.40 – 0.60	0.6	0.80 – 0.95	0.9	0.70 – 0.95	0.9	0.25	0.40
5	0.10 – 0.30	0.25	0.40 – 0.60	0.6	0.80 – 0.95	0.9	0.70 – 0.95	0.9	0.25	0.40
10	0.10 – 0.30	0.25	0.40 – 0.60	0.6	0.80 – 0.95	0.9	0.70 – 0.95	0.9	0.25	0.40
25	10%	0.275	10%	0.66	10%	0.98	10%	0.98	0.28	0.44
50	20%	0.3	20%	0.72	20%	0.98	20%	0.98	0.30	0.46
100	25%	0.3125	25%	0.75	25%	0.98	25%	0.98	0.31	0.47

* from MTO Chart 1.07

Lehigh Heidelberg Cement Company
Stormwater Management Report

Time to Peak

Time of Concentration - Area 101

(Based on Curve Number for 10Yr Return Period)

Full Development

Calculate Time of Concentration by ***Airport Method***

$$T_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{0.33}}$$

Where:

T_c = Time of Concentration in min

L = Watershed length in metres =

320 m

C = Runoff Coefficient =

0.4

S_w = Watershed Slope in %

1.7 %

A = Watershed area in hectares

7.67 ha

T_c = **34.5 min**

Slope Calculation

85/10 Method

Longest Flow Length = 320 m

Elevation at 85% = 104.0 m

Elevation at 10% = 100.0 m

Length at 85/10 = 240.0 m

Slope = 0.017 m/m

Lehigh Heidelberg Cement Company Stormwater Management Report

Existing Network

STORM SEWER DESIGN SHEET

Peak Runoff Estimate by Rational Method

$$Q = 1/360 C I A$$

Where:

Q = Peak Flow in cms
C = Runoff Coefficient
I = Rainfall Intensity in mm/hr
A = Area in hectares

Intensity for Belleville

$$i = A^*T_c^*B$$

Where:

I = Rainfall Intensity in mm/hr
T_c = Time of Concentration in hours

10-Year Parameters

A = 30.4
B = -0.678

Manning's Coef

CSP 0.024
RCP/PVC 0.013

Pipe Capacity by Manning's Equation

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

Where:

A = area of pipe in m²
R = Hydraulic radius = A / P
P = Wetted perimeter
S = Slope (m/m)
n = Manning's friction coef.

Check

$$q \leq Q$$

$$V \leq 6 \text{ m/s}$$

LOCATION			PEAK FLOW CALCULATION					PROPOSED SEWER												
LOCATION	FROM	TO	CATCHMENT AREAS				CUM. R.C. x A	TIME OF CONCENTRATION	INTENSITY	PEAK FLOW	Pipe Size	Length	Type of Pipe	Grade (use m/m)	Capacity, Full Flow n = 0.013	Time of Flow	Actual Velocity at Q _d	Check Capacity	q/Q	
			RUNOFF COEFFICIENT																	
			0.25	0.4	0.6	0.9														
	MH2a	MH2					0.00	20.0	64.0	0.000	750	31.0	RCP	0.50%	0.787	1.78	0.29	0.00		
102, 103 106	MH2	MH4			5.65		3.39	20.0	64.0	0.603	750	71.7	RCP	1.80%	1.494	3.38	3.19	OK	0.40	
	MH4	MH5					0.70	20.4	63.3	0.719	750	30.8	RCP	0.50%	0.787	1.78	0.29	0.02	OK	
	MH5	MH5a					0.00	20.6	62.7	0.712	900	13.4	RCP	0.50%	1.280	2.01	0.11	2.06	OK	
	MH5a	MH6					0.00	20.8	62.4	0.710	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	
100, 101, 105 108	CB3	CB4	5.71	7.67			5.91	30.0	48.6	0.798	600	63.0	RCP	1.00%	0.614	2.17	0.48	#REF!	1.30	
	CB4	MH6					1.09	30.5	48.1	0.935	600	11.0	RCP	1.00%	0.614	2.17	0.08	#REF!	1.52	
107 110	MH6	MH7					0.53	30.6	48.0	1.543	1066	53.8	RCP	1.00%	2.843	3.19	0.28	3.24	OK	
	MH7	MH8					0.40	30.8	47.7	1.581	1066	57.1	RCP	1.40%	3.364	3.77	0.25	3.70	OK	
109	CB9	MH8		2.04			0.82	20.0	64.0	0.145	300	84.8	CSP	3.00%	0.091	1.28	1.10	#REF!	1.60	
111	MH8	MH9					1.46	31.1	47.5	1.872	1066	38.9	RCP	0.50%	2.010	2.25	0.29	2.56	OK	
	MH9	MH9a					0.00	31.4	47.2	1.861	1050	18.8	RCP	1.00%	2.731	3.15	0.10	3.38	OK	
112	DD1	MH9a					1.26	20.0	64.0	0.202	900	55.0	RCP	0.50%	1.280	2.01	0.46	1.47	OK	
113, 114	MH9a	MH10					0.00	31.5	47.1	2.005	1066	75.0	RCP	0.60%	2.202	2.47	0.51	2.79	OK	
	MH10	MH11					0.00	32.0	46.6	1.983	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.56	OK	
	MH11	MH12					0.67	32.3	46.3	2.058	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.55	Surch	
	MH12	Gate Chamber					0.00	32.5	46.1	2.047	1066	5.0	RCP	0.50%	2.010	2.25	0.04	2.55	Surch	
104, 115	900 CSP	Gate Chamber			10.95		6.57	20.0	64.0	1.168	900	191.0	CSP	0.50%	0.693	1.09	2.92	#REF!	1.69	
	Gate Chamber	Holding Pond					0.00	32.6	46.0	2.885	1066	100.0	RCP	0.50%	2.010	2.25	0.74	#REF!	1.44	
										MTO Chart 2.30 shows a pipe will convey 7% beyond Manning full flow										
										Designed:	Project:									
										Checked:	Lehigh Cement									
										Date:	Prince Edward County									



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Lehigh Heidelberg Cement Company Stormwater Management Report

Proposed Improvements

STORM SEWER DESIGN SHEET

Peak Runoff Estimate by Rational Method
 $Q = 1/360 \ C / A$

Where:
 Q = Peak Flow in cms
 C = Runoff Coefficient
 I = Rainfall Intensity in mm/hr
 A = Area in hectares

Intensity for Belleville
 $I = A^*T_c^{0.08}$

Where:
 I = Rainfall Intensity in mm/hr
 T_c = Time of Concentration in hours

10-Year Parameters
 A = 30.4
 B = -0.678

Pipe Capacity by Manning's Equation
 $Q = 1/n \ A R^{2/3}$

Where:
 A = area of pipe in m²
 R = Hydraulic radius = A / P
 P = Wetted perimeter
 S = Slope (m/m)
 n = Manning's friction coef.

Manning's Coef
 CSP 0.024
 RCP/PVC 0.013

Check
 $q \leq Q$
 $V \leq 6 \text{ m/s}$

LOCATION			PEAKFLOW CALCULATION						PROPOSED SEWER											
LOCATION	FROM	TO	CATCHMENT AREAS				CUM. R.C. x A	TIME OF CONCENTRATION	INTENSITY	PEAK FLOW	Pipe Size	Length	Type of Pipe	Grade (use m/m)	Capacity/ Full Flow Velocity n =	Time of Flow	Actual Velocity at Q _d	Check Capacity	q/Q	
			RUNOFF COEFFICIENT																	
			0.25	0.4	0.6	0.9	ha	min	mm/hr	m ³ /s	(mm)	(m)		(%)	(m ³ /s)	min	(m/s)			
							0.00	20.0	64.0	0.000	750	31.0	RCP	0.50%	0.787	1.78	0.29	0.00	OK	
102, 103 106		MH2a					3.39	20.0	64.0	0.608	750	71.7	RCP	1.80%	1.494	3.38	0.35	3.19	OK	0.40
		MH2			5.65		0.70	20.4	63.3	0.719	750	30.8	RCP	0.50%	0.787	1.78	0.29	2.02	OK	0.91
		MH4				0.78	4.09	20.6	62.7	0.712	900	13.4	RCP	0.50%	1.280	2.01	0.11	2.06	OK	0.56
		MH5					0.00	20.8	62.4	0.710	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	0.55
		MH5a					0.00	20.8	62.4	0.710	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	0.55
		MH6					0.00	20.8	62.4	0.710	900	42.0	RCP	0.50%	1.280	2.01	0.35	2.06	OK	0.55
100, 101, 105 108		CB3	5.71	7.67			5.91	30.0	48.6	0.788	750	63.0	RCP	1.00%	1.113	2.52	0.42	2.74	OK	0.72
		CB4					1.09	30.4	48.2	0.937	750	11.0	RCP	1.00%	1.113	2.52	0.07	2.83	OK	0.84
		MH6					1.21	30.4	48.2	0.937	750	11.0	RCP	1.00%	1.113	2.52	0.07	2.83	OK	0.84
107 110		MH7					0.53	30.5	48.1	1.546	1066	53.8	RCP	1.00%	2.843	3.19	0.28	3.24	OK	0.54
		MH8					0.40	30.8	47.8	1.584	1066	57.1	RCP	1.40%	3.364	3.77	0.25	3.70	OK	0.47
109 111		MH8					0.82	20.0	64.0	0.145	375	84.8	RCP	3.00%	0.304	2.75	0.51	2.71	OK	0.48
		MH9					1.46	31.0	47.5	1.875	1066	38.9	RCP	0.50%	2.010	2.25	0.29	2.56	OK	0.93
		MH9a					0.00	31.3	47.2	1.864	1050	18.8	RCP	1.00%	2.731	3.15	0.10	3.38	OK	0.68
112		DD11				1.26	1.13	20.0	64.0	0.202	900	55.0	RCP	0.50%	1.280	2.01	0.46	1.47	OK	0.16
		MH9a					0.00	31.4	47.1	2.008	1066	75.0	RCP	0.60%	2.202	2.47	0.51	2.79	OK	0.91
113, 114		MH10					0.00	31.9	46.6	1.987	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.56	OK	0.99
		MH11					0.00	32.2	46.4	2.062	1066	35.0	RCP	0.50%	2.010	2.25	0.26	2.55	Surch	1.03
		MH12				0.74	0.67	32.4	46.1	2.050	1066	5.0	RCP	0.50%	2.010	2.25	0.04	2.55	Surch	1.02
		Gate Chamber					0.00	32.4	46.1	2.050	1066	5.0	RCP	0.50%	2.010	2.25	0.04	2.55	Surch	1.02
104, 115		900 CSP			10.95		6.57	20.0	64.0	1.168	900	191.0	CSP	0.50%	0.693	1.09	2.92	#REF!	Surch	1.69
		Gate Chamber					0.00	32.5	46.1	2.890	1066	100.0	RCP	0.50%	2.010	2.25	0.74	#REF!	Surch	1.44
		Holding Pond					0.00	32.5	46.1	2.890	1066	100.0	RCP	0.50%	2.010	2.25	0.74	#REF!	Surch	1.44
										(MFD Chart 2.30 shows a pipe will convey 7% beyond Manning full flow)										
										Designed: Bryan Keene, P.Eng.										
										Checked: Peter Yee, P.Eng.										
										Date: July 1, 2018										
										Project: Lehigh Cement										
										Prince Edward County										



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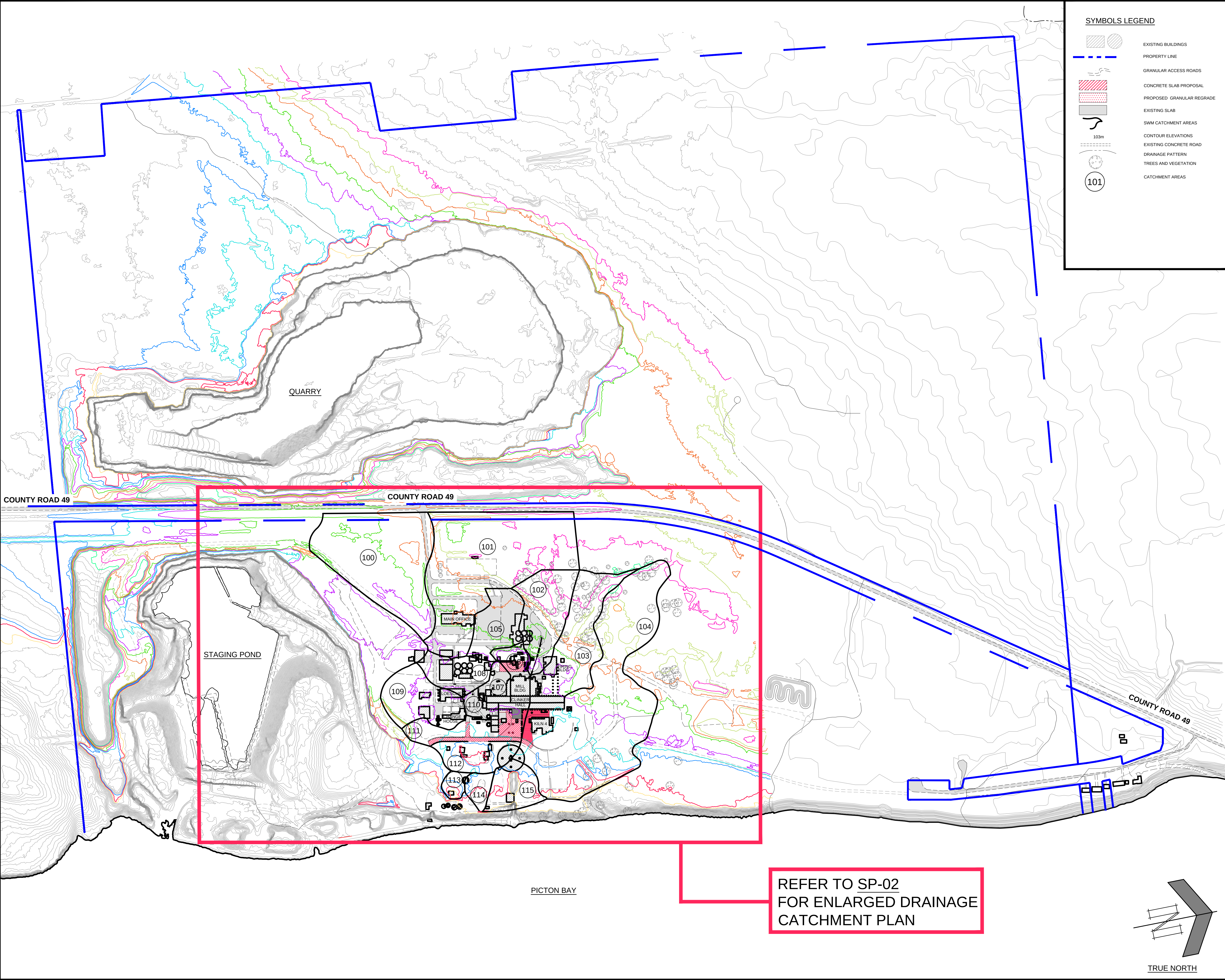
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***Lehigh Heidelberg Cement Company
Stormwater Management Report***

***Lehigh Heidelberg Cement Company
Stormwater Management Report***

APPENDIX D
STORM WATER MANAGEMENT DRAWINGS
SP01-SP07

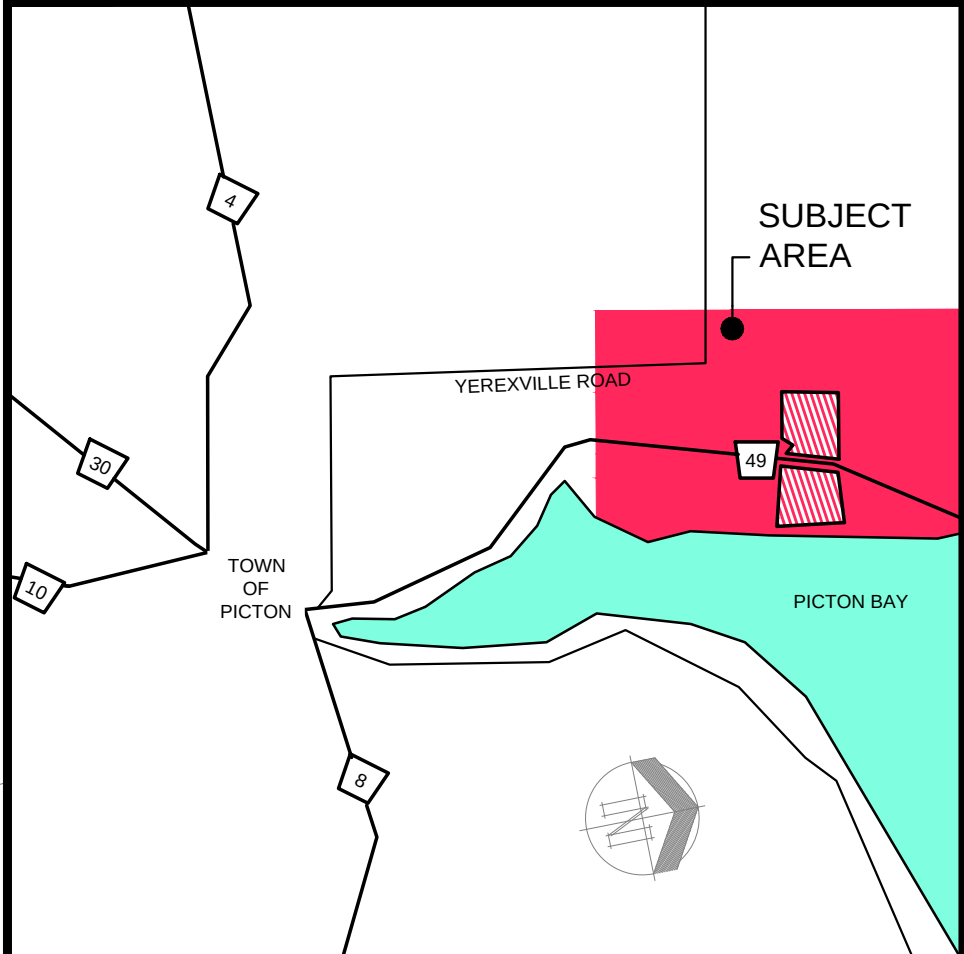


SYMBOLS LEGEND

- EXISTING BUILDINGS
- PROPERTY LINE
- GRANULAR ACCESS ROADS
- CONCRETE SLAB PROPOSAL
- PROPOSED GRANULAR REGRADE
- EXISTING SLAB
- SWM CATCHMENT AREAS
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

CONTOUR LEGEND

- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m
- 56m SWM POND LEVEL



KEY PLAN
SCALE N.T.S.

E	REVISED TO SUIT SWM COMMENTS	JPS	05JULY2018
D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018
No.	REVISIONS	MADE BY	DATE

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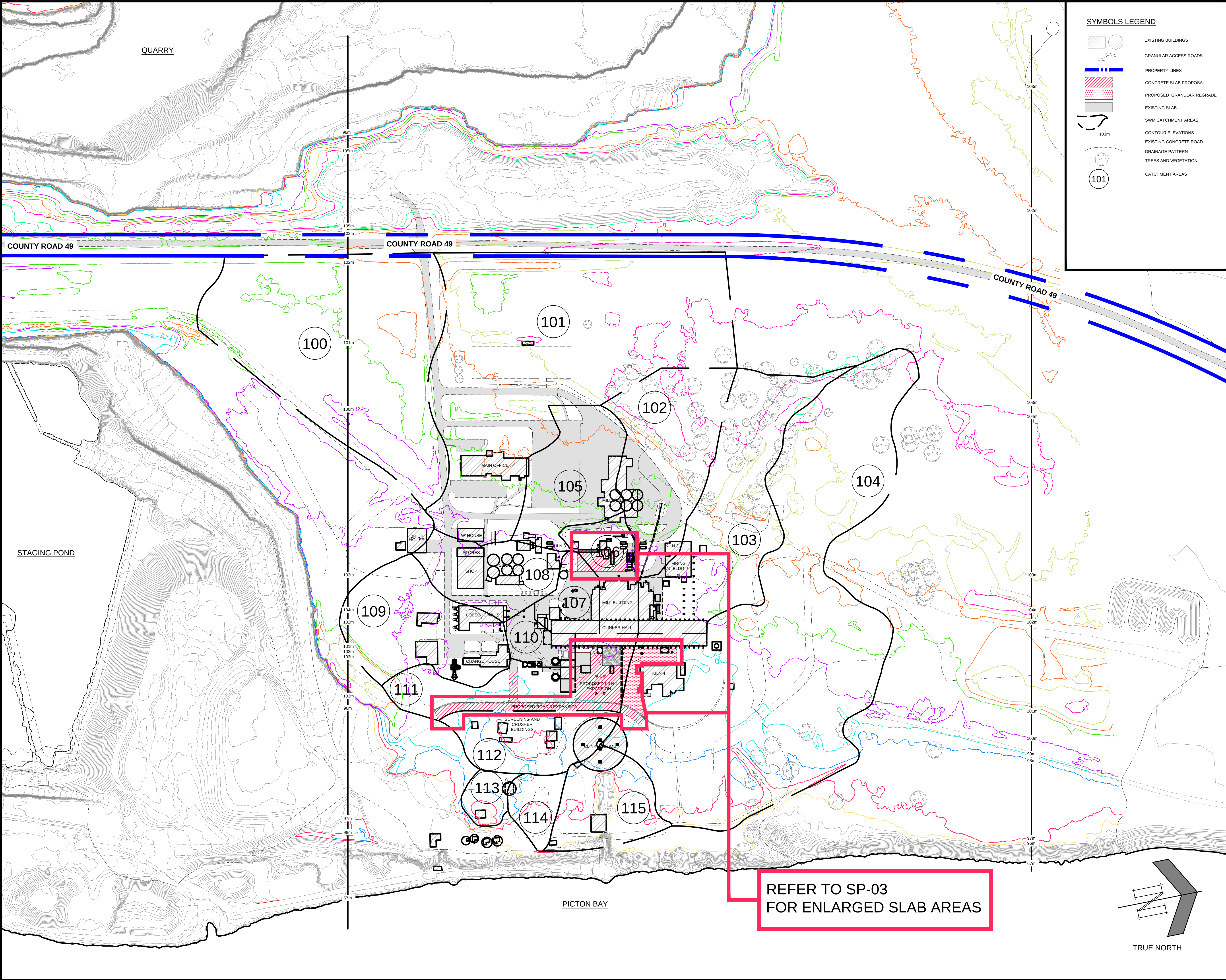
SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY
1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

DRAINAGE CATCHMENT AREAS

CHECKED BY:	DATE:	CONTRACT NO.:
CGS	12MAY18	
DESIGNED BY:	DRAWING NO.:	
CGS		
DRAWN BY:		
JPS		
SCALE:	REVISION NO.:	
1:4000		

SP01

PLOT DATE:

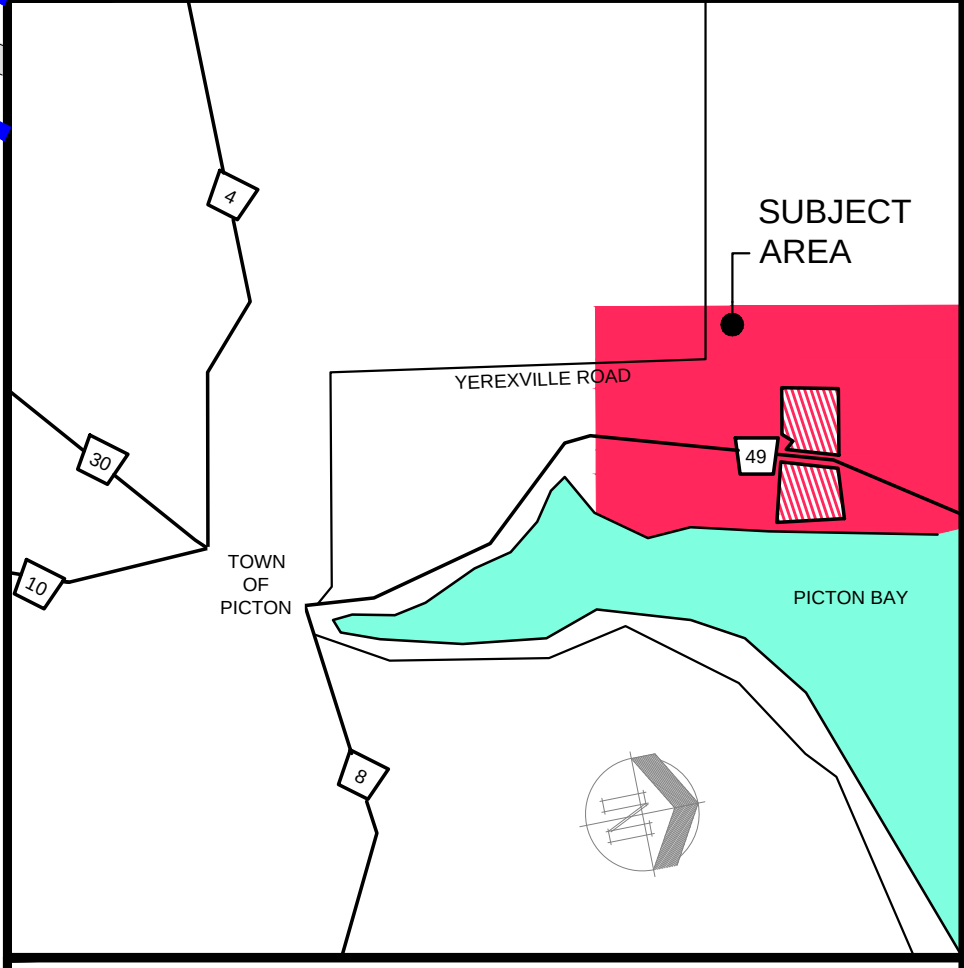


SYMBOLS LEGEND

- EXISTING BUILDINGS
- GRANULAR ACCESS ROADS
- PROPERTY LINES
- CONCRETE SLAB PROPOSAL
- PROPOSED GRANULAR REGRADE
- EXISTING SLAB
- SWM CATCHMENT AREAS
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

CONTOUR LEGEND

- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m
- 56m SWM POND LEVEL



KEY PLAN
SCALE N.T.S.

No.	REVISIONS	MADE BY	DATE
D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018

CORNERSTONE BUILDERS LTD.

FAX: 613-968-4986 general@cornerstonebuilders.ca PH: 613-968-3501

SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY

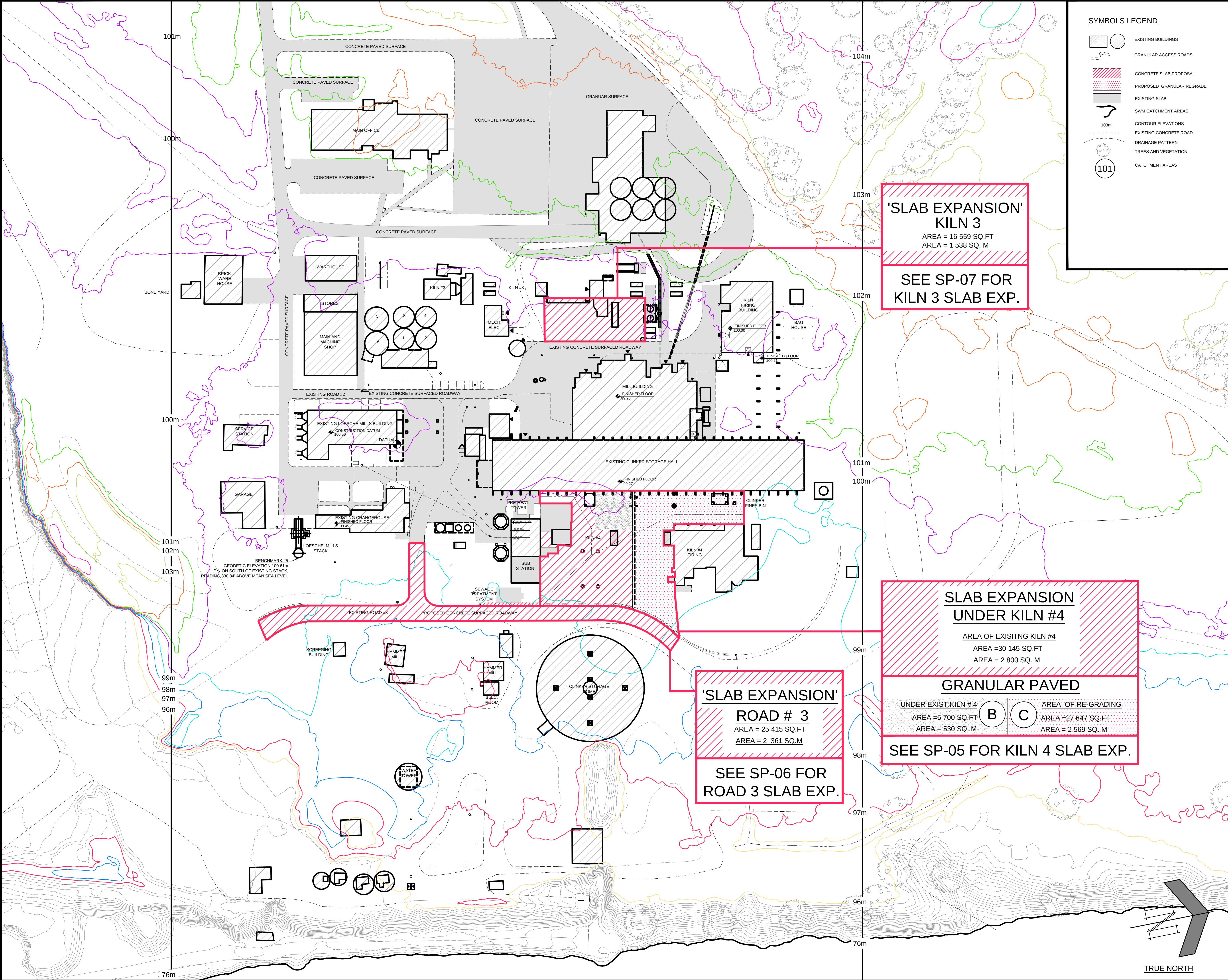
1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

ENLARGED DRAINAGE CATCHMENT AREAS

CHECKED BY:	DATE:	CONTRACT NO:
CGS	12MAY18	
DESIGNED BY:		DRAWING NO:
CGS		
DRAWN BY:		
JPS		
SCALE:		REVISION NO:
1:2000		

SP02

PLOT DATE:



SYMBOLS LEGEND

- EXISTING BUILDINGS
- GRANULAR ACCESS ROADS
- CONCRETE SLAB PROPOSAL
- PROPOSED GRANULAR REGRADE
- EXISTING SLAB
- SWM CATCHMENT AREAS
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

CONTOUR LEGEND

- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m
- 56m SWM POND LEVEL

'SLAB EXPANSION' KILN 3
AREA = 16 559 SQ.FT
AREA = 1 538 SQ. M

SEE SP-07 FOR KILN 3 SLAB EXP.

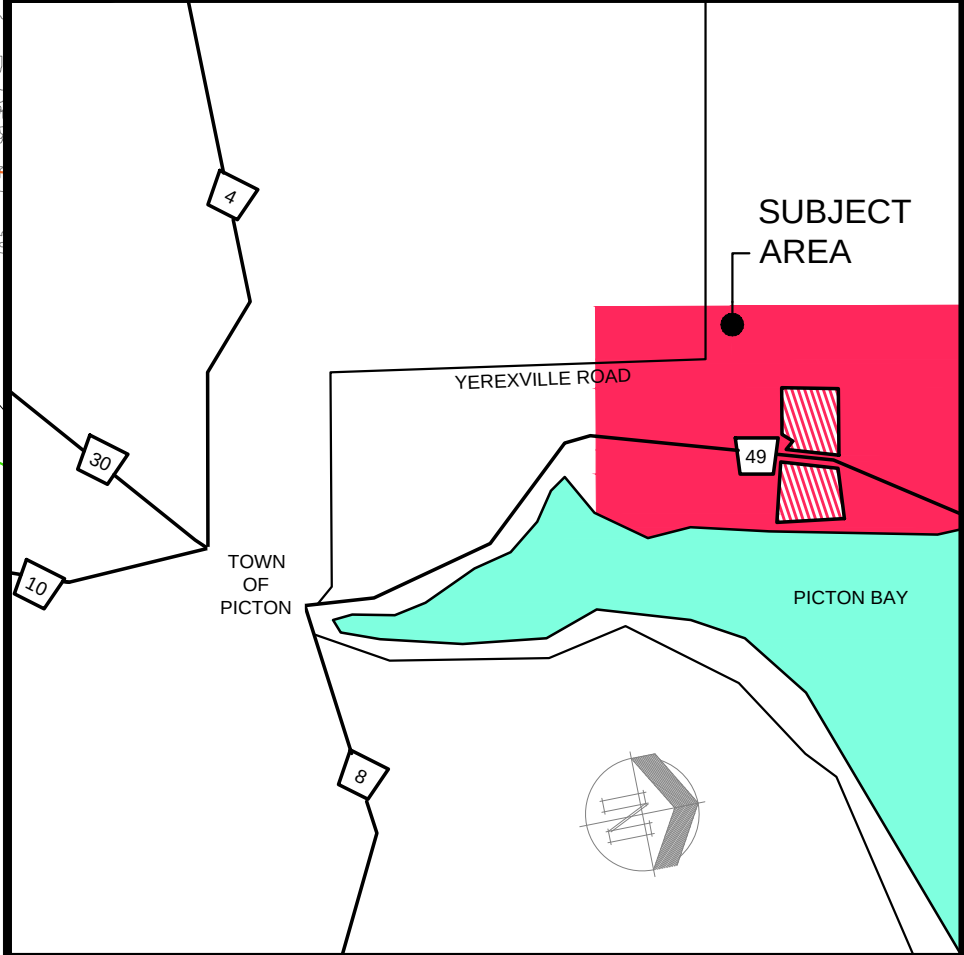
SLAB EXPANSION UNDER KILN #4
AREA OF EXISITNG KILN #4
AREA =30 145 SQ.FT
AREA = 2 800 SQ. M

GRANULAR PAVED
UNDER EXIST.KILN # 4
AREA =5 700 SQ.FT
AREA = 530 SQ. M
B C
AREA OF RE-GRADING
AREA =27 647 SQ.FT
AREA = 2 569 SQ. M

SEE SP-05 FOR KILN 4 SLAB EXP.

'SLAB EXPANSION' ROAD # 3
AREA = 25 415 SQ.FT
AREA = 2 361 SQ.M

SEE SP-06 FOR ROAD 3 SLAB EXP.



KEY PLAN
SCALE N.T.S.

E	REVISED TO SUIT SWM COMMENTS	JPS	06JULY2018
D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018
No.	REVISIONS	MADE BY	DATE

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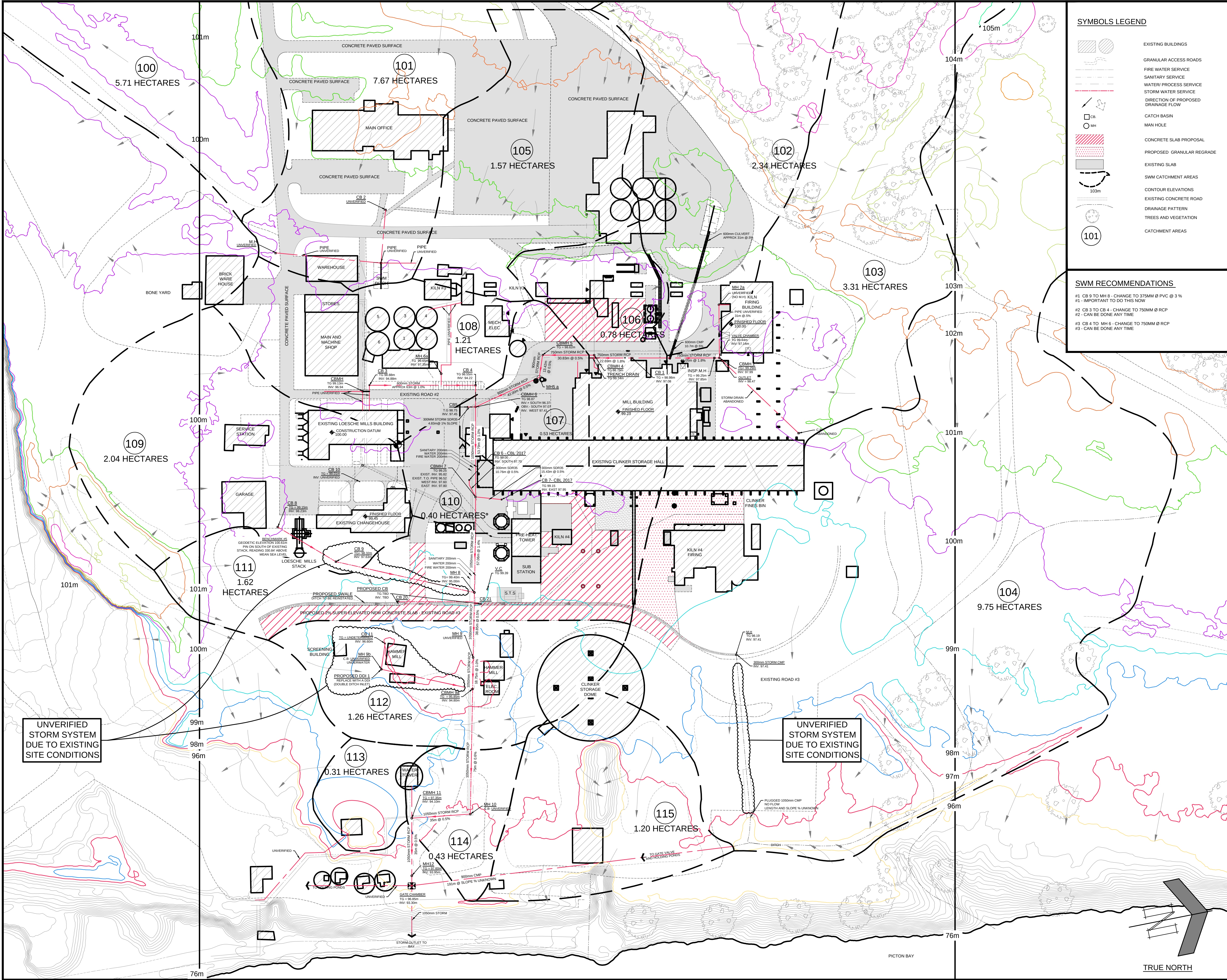
SITE IMPROVEMENTS FOR:
LEIGH HEIDELBERG CEMENT COMPANY
1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

OVERALL SLAB EXPANSION SCOPE

CHECKED BY:	DATE:	CONTRACT NO:
CGS	12MAY18	
DESIGNED BY:		DRAWING NO:
CGS		
DRAWN BY:		
JPS		
SCALE:		REVISION NO:
1:2000		

SP03

PLOT DATE:



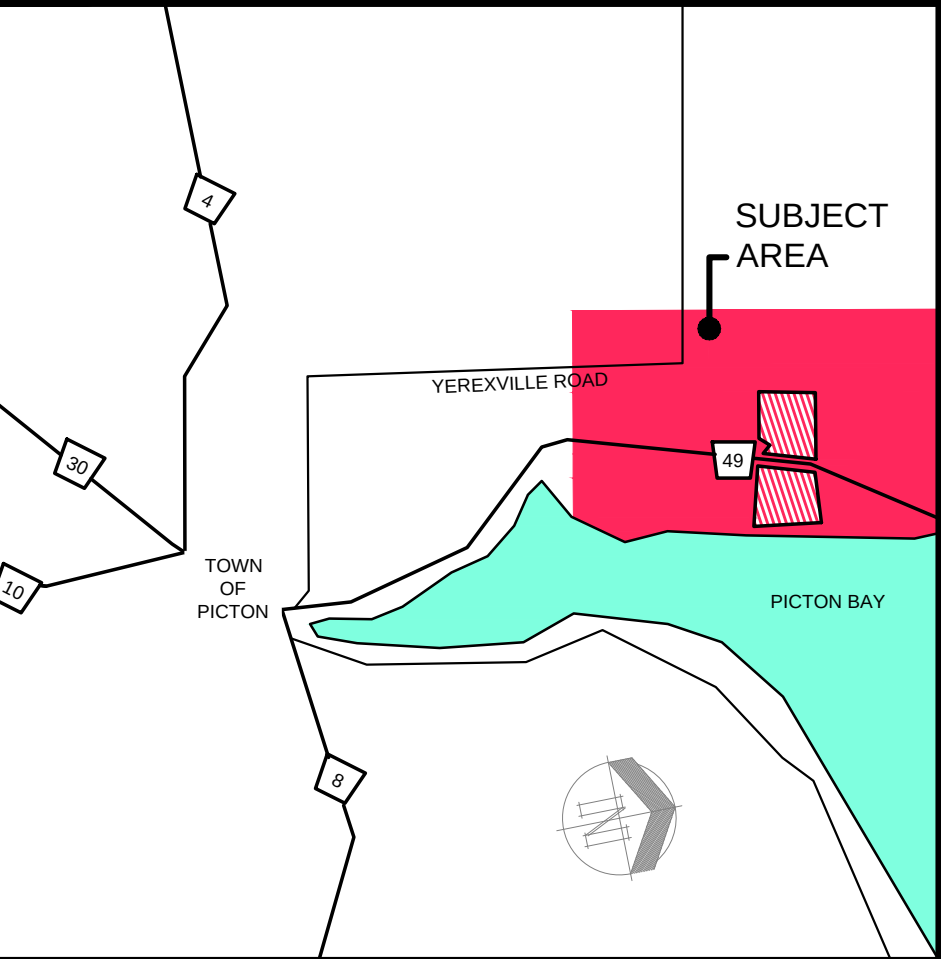
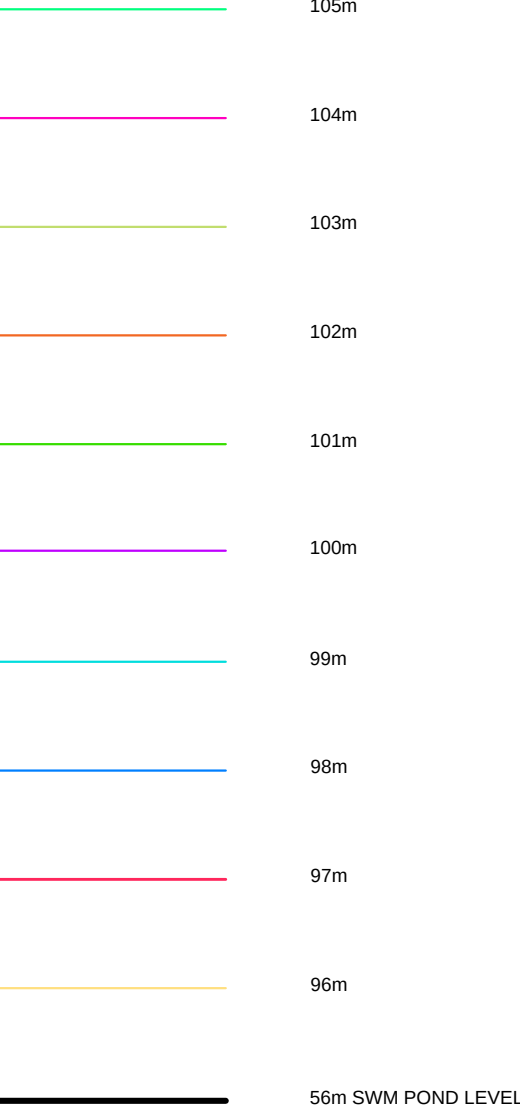
SYMBOLS LEGEND



SWM RECOMMENDATIONS

- #1 CB 9 TO MH 8 - CHANGE TO 375MM Ø PVC @ 3 %
- #1 - IMPORTANT TO DO THIS NOW
- #2 CB 3 TO CB 4 - CHANGE TO 750MM Ø RCP
- #2 - CAN BE DONE ANY TIME
- #3 CB 4 TO MH 5 - CHANGE TO 750MM Ø RCP
- #3 - CAN BE DONE ANY TIME

CONTOUR LEGEND



KEY PLAN SCALE N.T.S.			
E	REVISED TO SUIT SWM COMMENTS	JPS	05JULY2018
D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018
No.	REVISIONS	MADE BY	DATE

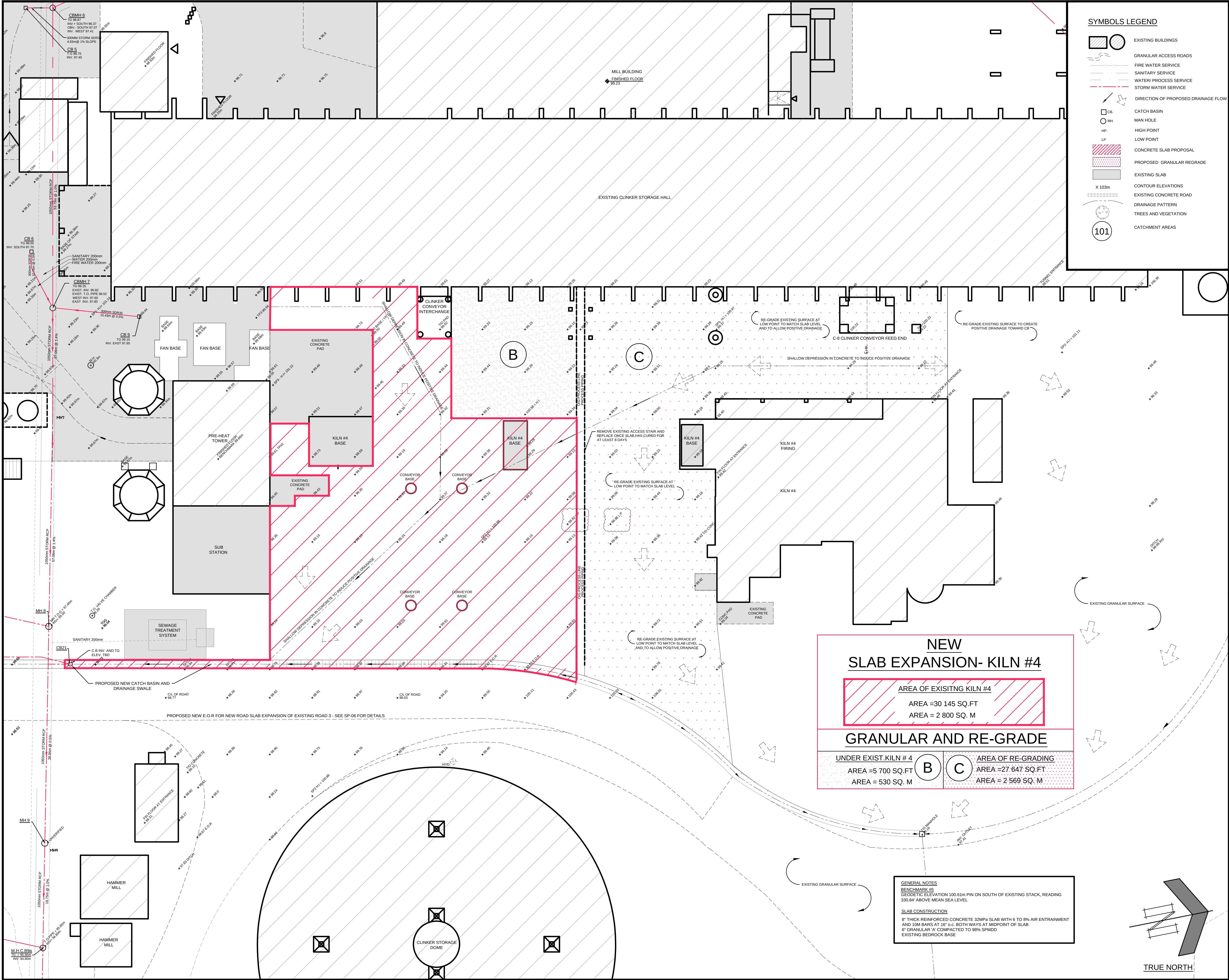
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FAX: 613-968-4986 general@cornerstonebuilders.ca PH: 613-968-3501

SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY
1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

ENLARGED STORM DRAINAGE AND CATCHMENT PLAN

CHECKED BY:	DATE:	CONTRACT NO:
DESIGNED BY:	12MAP18	DRAWING NO:
DRAWN BY:	JPS	SP04
SCALE:	1:1000	REVISION NO:





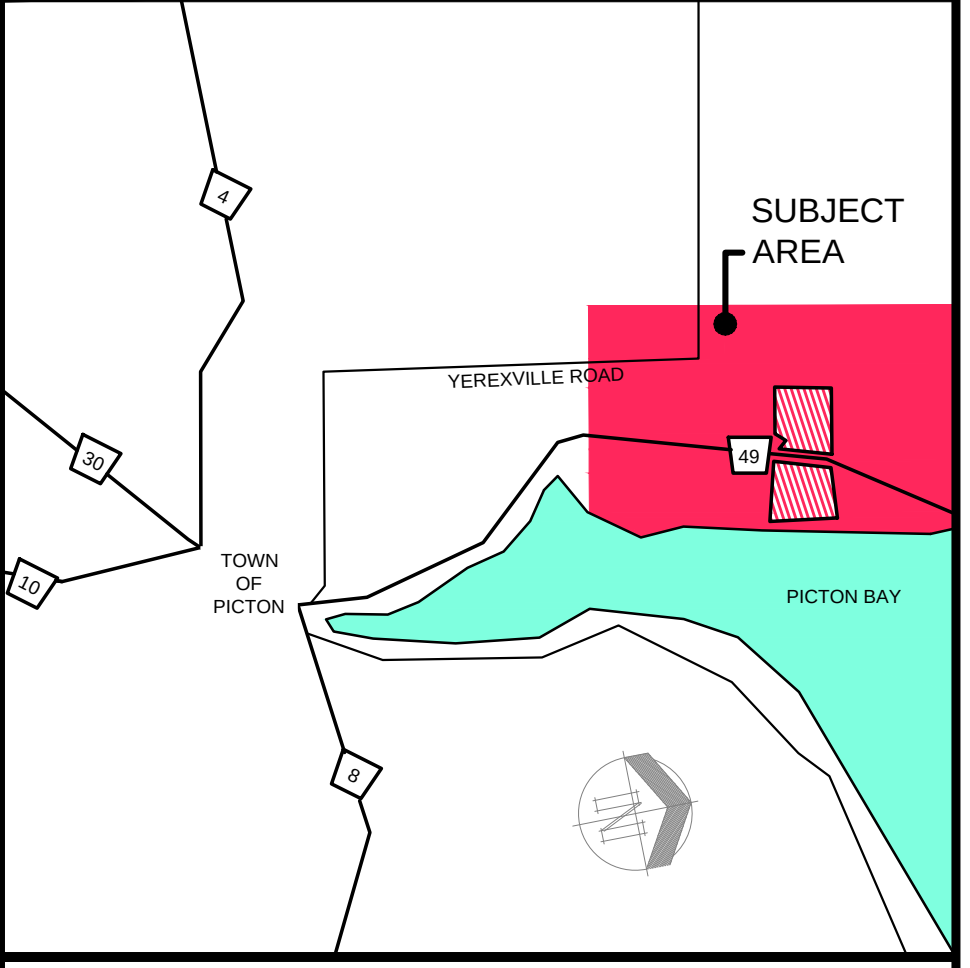
SYMBOLS LEGEND

- EXISTING BUILDINGS
- GRANULAR ACCESS ROADS
- FIRE WATER SERVICE
- SANITARY SERVICE
- WATER/ PROCESS SERVICE
- STORM WATER SERVICE
- DIRECTION OF PROPOSED DRAINAGE FLOW
- CATCH BASIN
- MAN HOLE
- HIGH POINT
- LOW POINT
- CONCRETE SLAB PROPOSAL
- PROPOSED GRANULAR REGRADE
- EXISTING SLAB
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

101

CONTOUR LEGEND

- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m
- 56m SWM POND LEVEL



KEY PLAN
SCALE N.T.S.

D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018
No.	REVISIONS	MADE BY	DATE

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SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY

1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

SLAB EXPANSION UNDER KILN #4

CHECKED BY:	DATE:	CONTRACT NO.:
CGS	12MAY18	
DESIGNED BY:		DRAWING NO.:
CGS		
DRAWN BY:	JPS	SP05
SCALE:	1/300	REVISION NO.:
		D

PLOT DATE:

NEW SLAB EXPANSION- KILN #4

AREA OF EXISTING KILN #4
AREA = 30 145 SQ.FT
AREA = 2 800 SQ. M

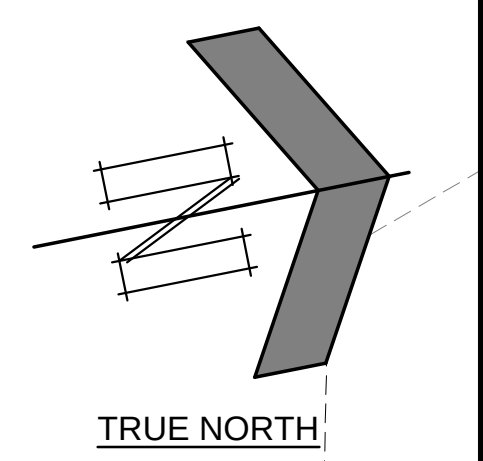
GRANULAR AND RE-GRADE

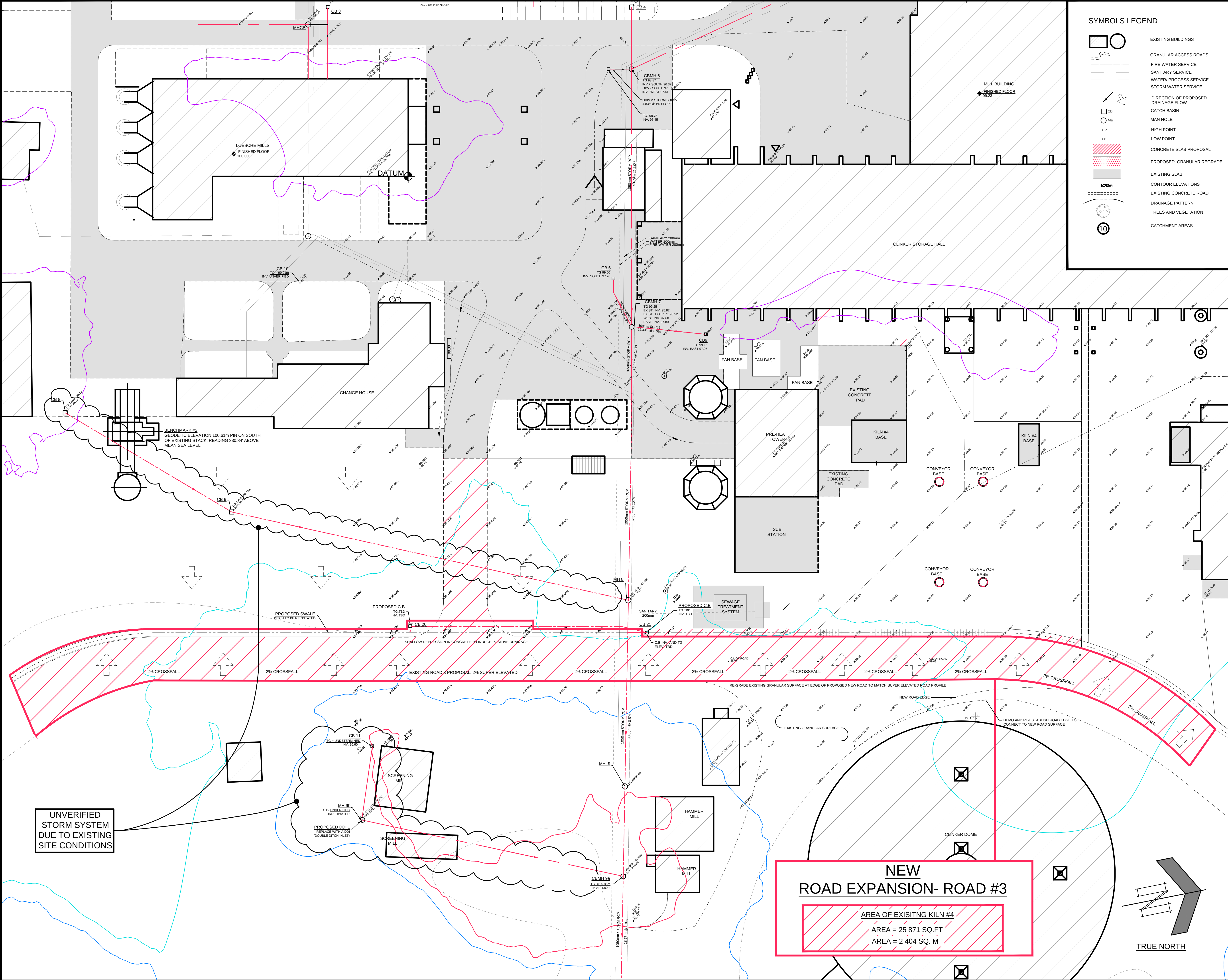
UNDER EXIST. KILN # 4
AREA = 5 700 SQ.FT
AREA = 530 SQ. M

AREA OF RE-GRADING
AREA = 27 647 SQ.FT
AREA = 2 569 SQ. M

GENERAL NOTES
BENCHMARK #5
GEODETIC ELEVATION 100.61m PIN ON SOUTH OF EXISTING STACK, READING 330.84' ABOVE MEAN SEA LEVEL

SLAB CONSTRUCTION
8" THICK REINFORCED CONCRETE 32MPa SLAB WITH 6 TO 8% AIR ENTRAINMENT AND 10M BARS AT 16" o.c. BOTH WAYS AT MIDPOINT OF SLAB
6" GRANULAR A' COMPACTED TO 98% SPMD
EXISTING BEDROCK BASE



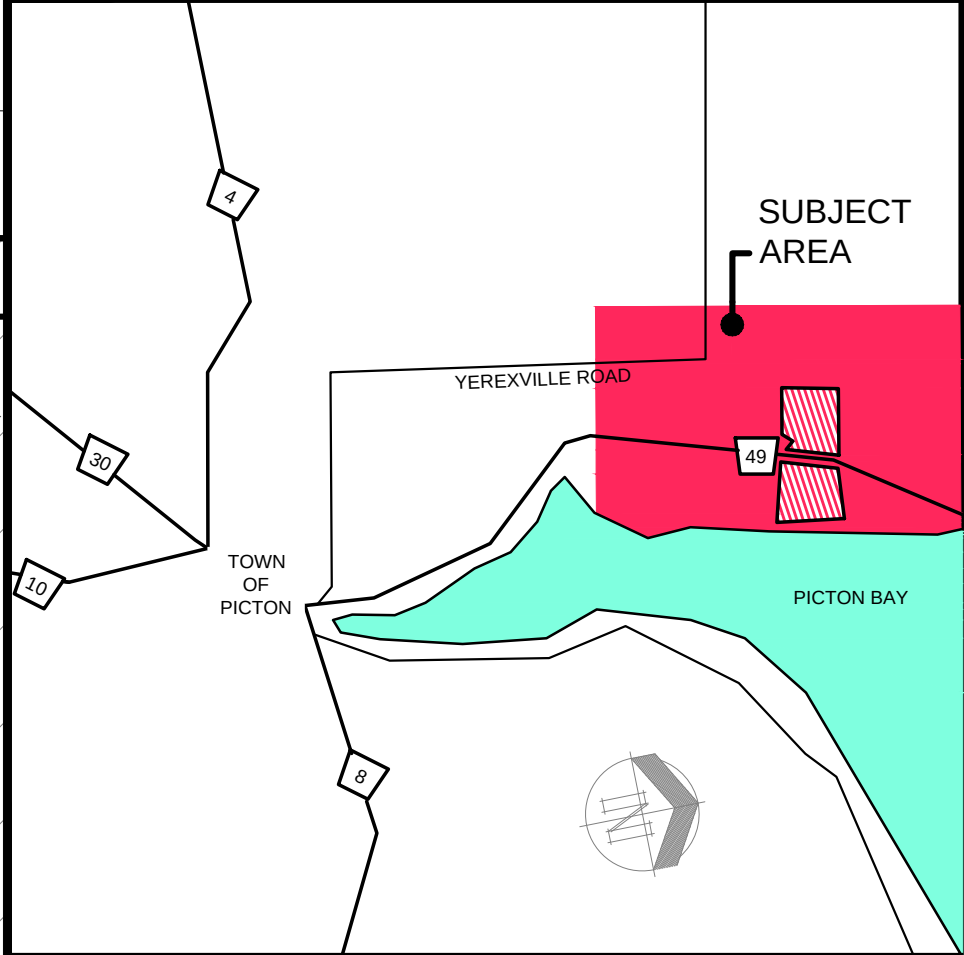


SYMBOLS LEGEND

- EXISTING BUILDINGS
- GRANULAR ACCESS ROADS
- FIRE WATER SERVICE
- SANITARY SERVICE
- WATER/ PROCESS SERVICE
- STORM WATER SERVICE
- DIRECTION OF PROPOSED DRAINAGE FLOW
- CATCH BASIN
- MAN HOLE
- HIGH POINT
- LOW POINT
- CONCRETE SLAB PROPOSAL
- PROPOSED GRANULAR REGRADE
- EXISTING SLAB
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

CONTOUR LEGEND

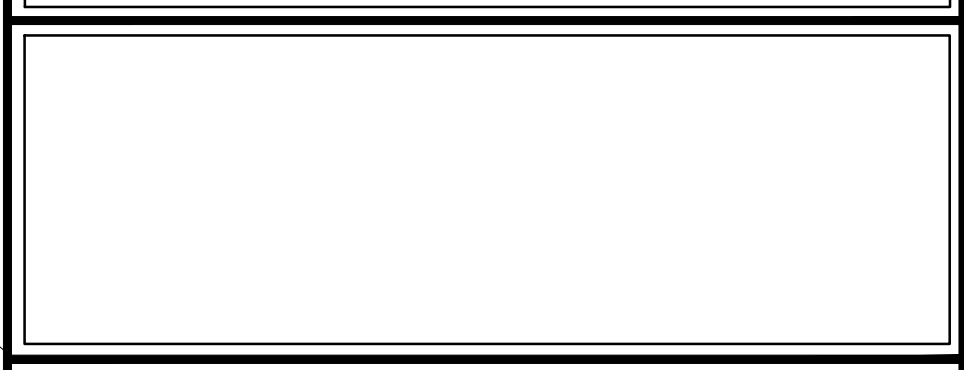
- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m
- 56m SWM POND LEVEL



KEY PLAN
SCALE N.T.S.

No.	REVISIONS	MADE BY	DATE
D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	11JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018

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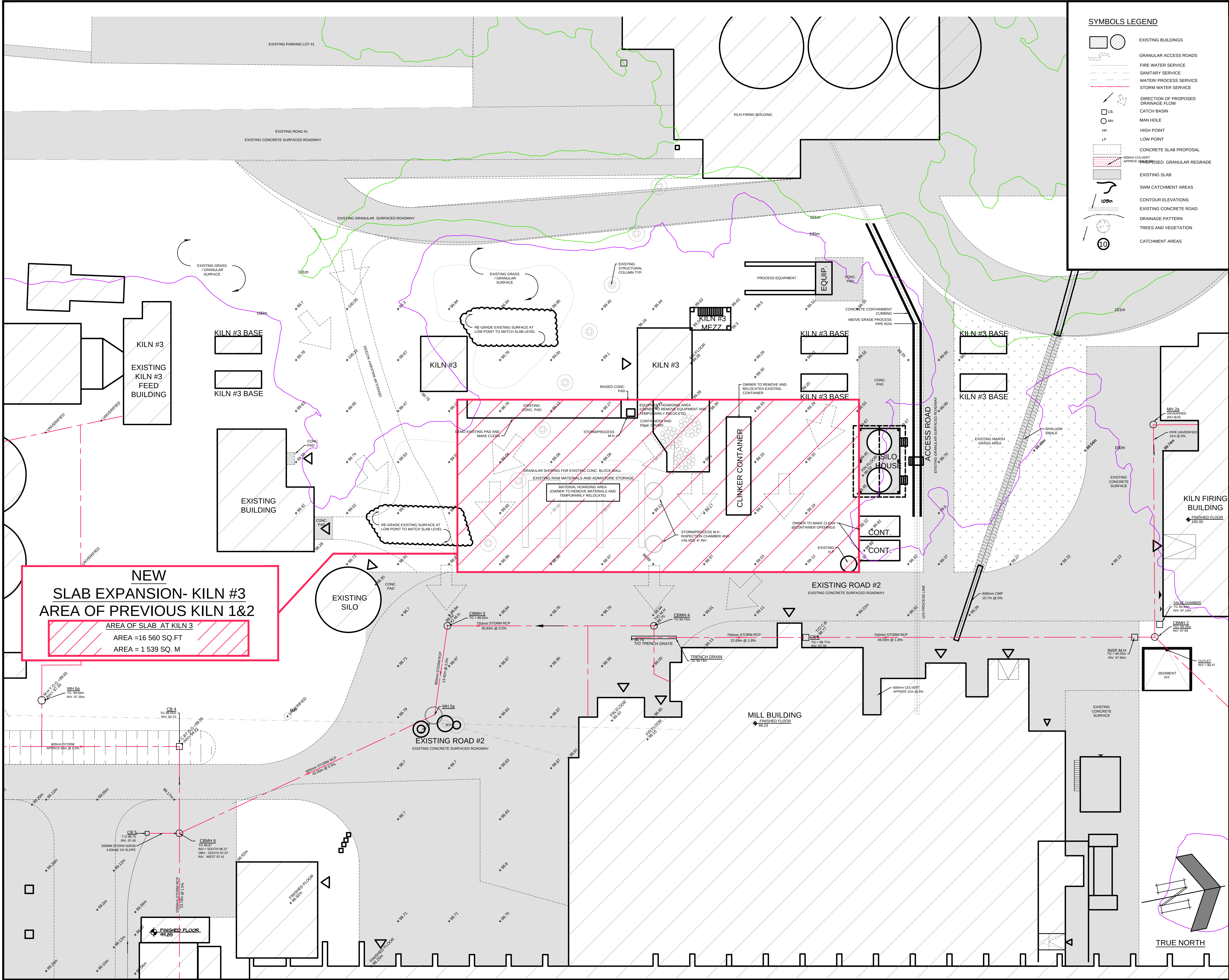
SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY
1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

ROAD #3 SLAB EXPANSION

	CHECKED BY:	DATE:	CONTRACT NO:
	CGS	12MAR18	
	DESIGNED BY:	CGS	DRAWING NO:
	DRAWN BY:	JPS	SP06
	SCALE:	1:350	
			REVISION NO:

NEW ROAD EXPANSION- ROAD #3

AREA OF EXISTING KILN #4
AREA = 25 871 SQ. FT
AREA = 2 404 SQ. M



SYMBOLS LEGEND

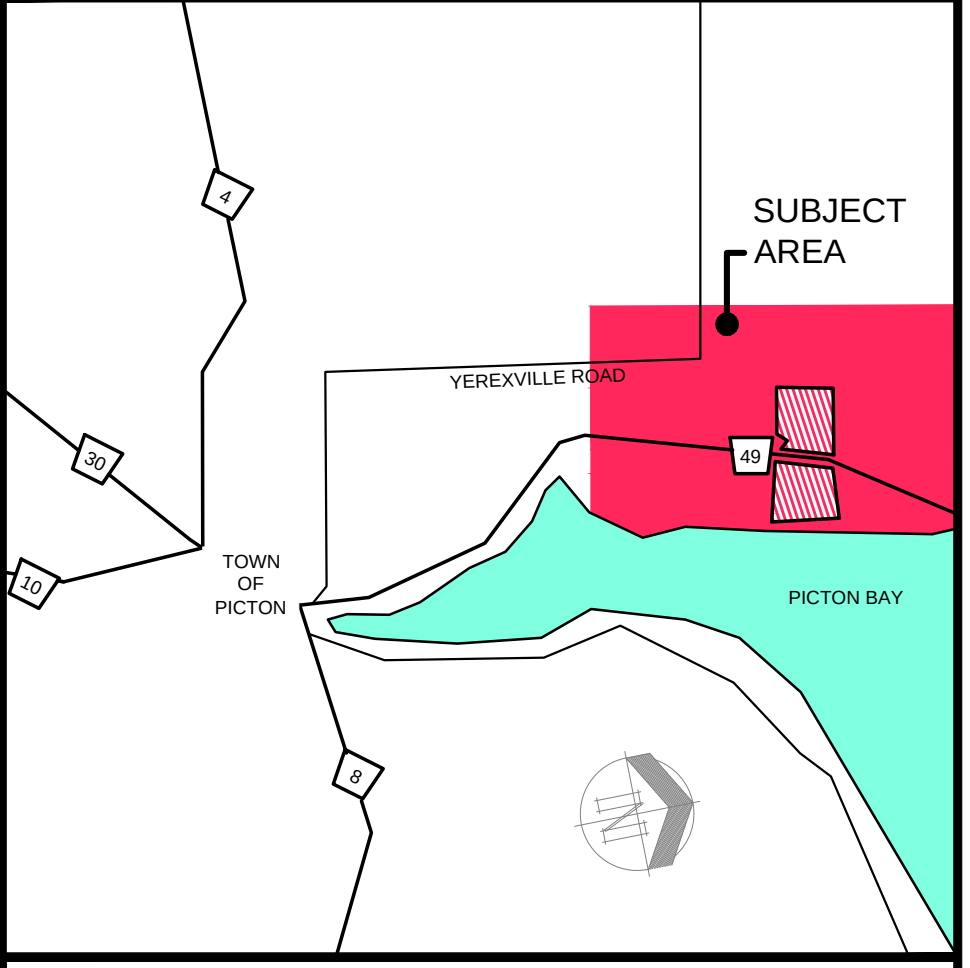
- EXISTING BUILDINGS
- GRANULAR ACCESS ROADS
- FIRE WATER SERVICE
- SANITARY SERVICE
- WATER/ PROCESS SERVICE
- STORM WATER SERVICE
- DIRECTION OF PROPOSED DRAINAGE FLOW
- CATCH BASIN
- MAN HOLE
- H.P. HIGH POINT
- L.P. LOW POINT
- CONCRETE SLAB PROPOSAL
- 600mm CULVERT APPROX 10.0m @ 1.0%
- EXISTING SLAB
- SWM CATCHMENT AREAS
- CONTOUR ELEVATIONS
- EXISTING CONCRETE ROAD
- DRAINAGE PATTERN
- TREES AND VEGETATION
- CATCHMENT AREAS

CONTOUR LEGEND

- 56m SWM POND LEVEL
- 105m
- 104m
- 103m
- 102m
- 101m
- 100m
- 99m
- 98m
- 97m
- 96m

NEW
SLAB EXPANSION- KILN #3
AREA OF PREVIOUS KILN 1&2

AREA OF SLAB AT KILN 3
AREA = 16 560 SQ.FT
AREA = 1 539 SQ. M



KEY PLAN
SCALE N.T.S.

D	REVISED TO SUIT SWM COMMENTS	JPS	26JUNE2018
C	ISSUED FOR SWM REVIEW	JPS	13JUNE2018
B	REVISED TO SUIT SWM COMMENTS	JPS	07JUNE2018
A	ISSUED FOR SWM REVIEW	JPS	14MARCH2018
No.	REVISIONS	MADE BY	DATE

CORNERSTONE BUILDERS LTD.

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SITE IMPROVEMENTS FOR:
LEHIGH HEIDELBERG CEMENT COMPANY

1370 COUNTY ROAD 49
PRINCE EDWARD COUNTY
K0K 2T0

SLAB EXPANSION UNDER KILN #3

CHECKED BY:	DATE:	CONTRACT NO:
DESIGNED BY:	12MAR18	DRAWING NO:
DRAWN BY:	JPS	SP07
SCALE:	1:250	REVISION NO:

PLOT DATE: