development engineering (London) Limited
CONSULTING CIVIL ENGINEERS

February 21, 2024 Project: DEL19-072

Dancor Creamery Inc.
101-16 Melanie Drive
Brampton, Ontario
L6T 4K9

Attn: Mr. Sean Ford

Re: Servicing Brief to Support Application for Consent
1425 Creamery Road
London, Ontario

INTRODUCTION

Development Engineering (London) Limited (herein referred to as DevEng), has been retained by Dancor Creamery Inc. (herein referred to as the Owner) to provide Professional Engineering services in support of the Application for Consent to sever the property registered as 1425 Creamery Road in London, Ontario. This Servicing Brief has been prepared to address the engineering-related conditions outlined in the *Notice of Provisional Consent Decision* which was issued by the City of London (herein referred to as the City) on August 30, 2019.

BACKGROUND INFORMATION & EXISTING CONDITIONS

The 13.84 ha subject site is located in northeast London, just south of the London International Airport. The property is bound by a Canadian Pacific Railway (CPR) corridor to the north, Creamery Road to the east, Dundas Street the south, and a Light Industrial parcel to the west.

The property currently contains a 4,656 m^2 (50,117 sq.ft) manufacturing facility and associated parking lot, but the majority of the land cover is agricultural farmland. The application for consent seeks to divide the subject site into two parcels. The 2.91 ha *Lands to be Retained* will consist of the manufacturing facility; parking lot; and some greenspace, while the 10.93 ha *Lands to be Severed* will consist of the remaining greenspace and farmland.

The topography is mild (±1.1% gradient) and slopes from southeast to northwest, releasing runoff to a ditch within the CPR corridor abutting the north property line of the lands to be severed, and to the adjacent property to the west which ultimately discharges to a stormwater drainage system including swales, culverts, and sewers at the rear of lots fronting Kostis Ave. Runoff is conveyed to the Loveless Drain where it continues south through the Parkinson Drain, the Crumlin Drain, and ultimately to the south branch of the Thames River. The subject site falls within the Crumlin Drain Subwatershed and is subject to the recommendations in the Pottersburg Creek and Crumlin Drain Subwatershed Study (herein referred to as the Crumlin SWS).

There are no municipal storm sewers, sanitary sewers, or watermain adjacent to the subject site. The nearest municipal infrastructure consists of the following:

- A 250mm diameter sanitary sewer and multiple 300mm to 450mm diameter watermains at the intersection of Crumlin Sideroad and Dundas Street (approximately 1.4 km southwest of the subject site)
- A storm sewer of unknown diameter at the intersection of Driver Lane and Dundas Street (approximately 1.6 km southwest of the subject site)
- A ditch inlet catchbasin on the adjacent lands to the west located behind the properties fronting Kostis Avenue (approximately 340m west of the subject site and across private lands)

The City provided a high-level cost estimate of $5.0M to extend the municipal services to subject site; however, it is not currently scheduled as a project under the Growth Management Implementation Strategy (GMIS) and, therefore, the costs would have to be borne by the Owner.

A detailed geotechnical analysis has not been completed for the subject site at this time; however, using the Ontario Borehole Records, it has been determined that the soils in the area consist of surficial layers of sand and gravel (±1.0m deep) underlain by silt, clay, and sand.

The existing manufacturing facility discharges its sewage to a private septic system located at the southwest corner of the building. The building also has an internal private water well to supply potable water to meet its demands. The appended Private Servicing Layout Plan shows the location of the existing septic system and well, with setbacks denoted to verify conformance with Article 8.2.1.6 of the Ontario Building Code.

Since there are no storm sewers available to the existing building, it is assumed that the building roof water leaders discharge to grade where roof runoff can follow the natural topography of the subject site and discharge at the northwest corner into the CPR corridor.

SERVICING THE SEVERED PARCEL

Since municipal infrastructure is not currently available to the subject site and it is not feasible for the Owner to front-end costs for a large-scale infrastructure extension project, the Owner is proposing private on-site servicing instead.

A 1.0 ha portion of the 10.93 ha lands to be severed is denoted on the Private Servicing Layout Plan as the Development Area. This will be the focus of the servicing analysis for the severed parcel.

Based on current zoning (LI1 – Light Industrial) and the City of London Official Plan Landuse Schedule Designation (Light Industrial), it is assumed that the severed parcel will contain an industrial warehouse establishment with a 15,000 ft^2 (1,394 m^2) building footprint. This has also been shown on the Private Servicing Layout Plan.

SEPTIC SYSTEM

A Class 4 Sewage System in accordance with Part 8 of the Ontario Building Code is being proposed to provide a sanitary sewer outlet for the proposed building within the development area since municipal servicing is not available.

As shown on the conceptual site plan prepared by the Owner, the proposed building would include four loading bays. Assuming sewage is produced at a rate of 150 L/day per loading bay (Ref. Table 8.2.1.3.B., OBC), a design sewage production rate of 600 L/day can be computed.

Given that the surficial soils are likely to consist of sand and gravel, a percolation time between 1 and 20 min/cm can be assumed. When the daily sewage production rate and percolation time are applied to Table 8.7.3.1 of the OBC, the resulting minimum length of distribution pipe equals 8m. However, length of the distribution pipe would be governed by Article 8.7.3.1.1.a) of the OBC which states that the length shall not be less than 30 m when construction as a shallow buried trench.

In accordance with Article 8.2.2.3 of the OBC, a septic tank for non-residential use shall have a capacity of at least 3,600 L or three times the daily sewage production rate, which would be 2,700 L in this case. Therefore, for the purposes of this schematic design, the proposed septic system shall be comprised of a 2-compartment septic tank with a storage capacity of 3,600 L discharging to a leaching bed with five 6.0m long laterals.

The septic tank and leaching bed are shown schematically on the Private Servicing Layout Plan; however, both the size and location of each will have to be confirmed during detailed design. A geotechnical engineer and/or hydrogeologist should be retained during detailed design to confirm the in-situ hydraulic conductivity of the underlying soils. This will allow the designer of the leaching bed to estimate a release rate (Percolation Time) of the pretreated wastewater.

PRIVATE GROUNDWATER WELL

Since a municipal watermain is not currently available for the severed parcel to utilize, a new private groundwater water well will be required. The existing manufacturing facility within the lands to be retained is serviced from a groundwater well which indicates that the hydrogeological setting is conducive to their use. Moreover, according to Upper Thames River Conservation Authority (UTRCA) mapping, the subject site is not located within a Significant Groundwater Recharge Area, a Highly Vulnerable Aquifer, Intake Protection Zone, or Wellhead Protection Area. Based on this information, it is our opinion that the lands to be retained could feasibly be serviced to meet the potable water demands by utilizing a groundwater well.

For the purposes of this schematic design, it is assumed that the average day domestic potable water demand is equal to the sanitary sewage production rate based on a principle of *water-in-equals-water-out*. Assuming the average day demand is 0.007 L/s (600 L/day), the peak hour flow rate would be approximately 0.054 L/s (assuming peaking factor of 7.8 in accordance with Section 7.3.2.2 of the City of London Design Specifications and Requirements Manual). On a preliminary basis, the well pump and shaft diameter should be sized to accommodate a peak flow rate of 0.054 L/s. During detailed design, a comprehensive analysis of the tenant's water supply demands would be required, and a comprehensive groundwater well design would have to be undertaken by a qualified professional.

To meet the fire protection requirements, a dry hydrant and storage tank system are proposed. Calculations have been appended to this Design Brief; however, a summary of assumptions and results is provided below:

- Assumed Group F, Division 1, non-combustible construction with fire resistance; K=23
- Building Volume = 9,340 m^3
- Separation Distances are greater than 10m on all building sized; therefore, Spatial Coefficient = 1.0
- Total Required Volume, Q = 214,815 L

This volume could be accommodated using two pre-cast holding tanks, each with a capacity of 114 m^3 (i.e., total storage capacity of 228,000 L).

A geotechnical engineer and/or hydrogeologist should be retained during detailed design to confirm water pressure (artesian well), depth of the water table elevation, and contamination levels (if any).

STORMWATER MANAGEMENT SYSTEM

Quantity Control

The Crumlin SWS recommends providing peak flow attenuation (Ref. Section E2.2.1.i) and establishes a target of matching or reducing post-development peak flows to those of the existing conditions during all design storm events from the 2 to 100-year return period (Ref. Crumlin Drain Tributary and Catchment Area Factsheet: Area 9). The SWS also recommends infiltration facilities be utilized where permeable soils exist.

To accommodate these stormwater management requirements, an infiltration dry basin is recommended. This would allow for above-ground attenuation of stormwater using a restricted outflow while simultaneously encouraging infiltration of runoff back into the groundwater system.

The subject development area is approximately 1.0 ha which is currently grassland with an imperviousness of 0% (C=0.20). The proposed development would increase the imperviousness of this area to 96% (C=0.87). Rational Method computations have been prepared to estimate the required attenuation volume to reduce the post-development 100-year peak discharge rate to that of the existing/pre-development condition.

Based on these calculations, the greatest difference between the runoff volume and the release volume would occur at T=44 minutes and would require 352 m^3 of attenuation volume. While these calculations do not account for infiltration, it is anticipated that the additional peak flow and volume reduction provided by infiltration measures, if feasible, would reduce the required attenuation volume.

Since the subject site has no underground storm sewer to which the dry-basin's controlled outflow pipe could be connected, it must discharge to grade. Given the relatively flat topography adjacent to the development area (±1.0%) the dry-basin must remain shallow. A 1.0m deep infiltration dry-basin has been shown schematically on the Private Servicing Layout Plan with an average area of 360 m^2 yielding a storage volume of 360 m^3. A detailed stormwater management analysis and report should be prepared during detailed design.

Additionally, during detailed design, a qualified geotechnical engineer should confirm the proposed infiltration dry-basin has adequate separation from the proposed septic leaching bed such that the groundwater mounding plume from the stormwater infiltration does not adversely affect the percolation time of the leaching bed tiles.

Quality Control

The source of sediment introduced onto the development area is expected to be primarily road sands which would be collected and deposited on-site by transport trucks. As such, a negligible content of fine sediments such as silts and clays are expected to be suspended in the runoff from the proposed development.

Perimeter swales along the west and north limits of the development area are proposed to capture and convey the runoff from the development area to the dry-basin. A pea gravel diaphragm strip between the conveyance swales and the edge of the asphalt should be included to pre-treat runoff. The fine pea gravel acts as a filter to remove larger particles and trash. It also has a higher roughness coefficient than the asphalt which causes a reduction in sheet flow velocity allowing finer particles to settle out.

This pre-treatment measure, combined with the polishing effect of the grassed swales, should increase the longevity and function of the infiltration component of the dry-basin, as well as provide quality control for discharge from the dry-basin. Moreover, the property's discharge point is approximately 350m downgradient from the dry-basins outlet pipe. Any small amount of sediment that is not removed by the pre-treatment measures or the dry-basin would be removed by the vegetated land over which the dry-basin discharge will be released. From the perspective of the subject site outlet, these measures should result in no appreciable difference in water quality.

Erosion Control

The Crumlin SWS recommends development utilize extended detention and infiltration measures (where permeable soils exist) to help control erosion. Extended detention for erosion control is typically limited to quality control storm events (e.g. the 25mm design storm) and, since stormwater quantity control is provided for all storm events up to and including the 100-year return period, it is expected that no further permanent erosion control measures will be required. Temporary erosion and sediment controls will still be implemented during construction.

Water Balance

Given the tight subsoils (clays & silts) underlying the surficial sand and gravel layer, pre-development infiltration is expected to be very minor. The Crumlin SWS notes that approximately 10% of the rainfall within the subwatershed infiltrates into the groundwater system while the remaining 90% translates to runoff or evapotranspiration. Moreover, the proposed 1.0 ha development area represents 0.1% of the 609 ha Crumlin Drain subwatershed tributary area (Ref. Crumlin Drain Tributary and Catchment Area Factsheet: Area 9, Crumlin SWS).

An infiltration dry-basin is to be proposed in support of stormwater quantity and quality control objectives which also functions as an unquantified water balance measure. Undertaking a hydrogeological analysis and water balance calculations to inform the implementation of targeted mitigation measures is considered excessive and not warranted as the impact to the hydrologic cycle within the subwatershed is negligible.

Fluvial Morphology

The Crumlin SWS notes that the Crumlin Drain is in a degraded condition due to a high degree of entrenchment and is susceptible to bank erosion and bed scour. However, the subject development is not discharging directly to the watercourse and the stormwater flow reduction requirements are provided in accordance with the objectives of the Crumlin SWS. As such, it is our opinion that a fluvial geomorphology study is not warranted for the subject development.

SITE PREPARATION AND EROSION & SEDIMENT CONTROLS

Detailed erosion and sediment control (ESC) measures are to be included with the detailed site engineering design as part of a future submission package (i.e. Site Plan Approval). Temporary measures will be proposed to mitigate the offsite migration of sediments by incorporation of various BMP's and control measures. Such controls include silt fencing, straw bale barriers for inlet grate protection (CBs, and CBMHs), construction entrance mud mats, robust silt barriers in concentrated flow routes, tree preservation fencing and erosion control blanket treatment of significant fill/cut slopes. The control measures to be implemented on site should include:

- Installation of silt control fencing around the site perimeter at down-gradient locations;
- Preventing silt or sediment laden runoff from entering inlets (catchbasins/catchbasin manholes) by wrapping their inlet grates with filter fabric and incorporating straw bale filters (flow checks);
- Sodding the invert of swales as soon as possible after being constructed to mitigate erosion and downcutting; in general, minimizing the duration of soil exposure in erosion prone areas by temporary vegetation coverage (i.e. hydro-seeding) is recommended;

- Maintaining sediment and erosion control structures in good repair (including periodic cleaning as required) until such time as the Engineer or the municipality approves their removal;
- Incorporation of temporary measures at site construction entrances to minimize tracking of mud and debris onto road allowances; and,
- Scheduling of critical conveyance works during forecasts of little to no precipitation.

We trust this letter adequately outlines the proposed site servicing and stormwater management design strategy in support of the Application for Consent. Please do not hesitate to contact the undersigned if there are any questions.

DEVELOPMENT ENGINEERING (LONDON) LIMITED

LICENSED PROFESSIONAL ENGINEER
J. R. SMITH
100144789
FEB 21/24
PROVINCE OF ONTARIO

Josh Smith, P.Eng.
Partner

FIRE FLOW REQUIREMENTS - UNSPRINKLERED BUILDING
DEL19-072: Creamery Rd. Application for Consent

Date: February 2024
By: Jon Bakker, P.Eng. Checked: Josh Smith, P.Eng

Water supply for fire fighting of an unsprinklered building is determined using OBC A-3.2.5.7.

Occupancy Classification = Group F, Division 1
Type of Construction = noncombustible construction with fire separations and fire-resistance ratings provided in accordance with Subsection 3.2.2., including loadbearing walls, columns and arches.

Minimum supply of water = $Q = K \cdot V \cdot S_{TOT}$

K = water supply coefficient = see Table 1 = 23
V = total volume of building in m^3 = 9,340

Building Footprint = 1394.0 m^2
Average Height = 6.7 m

S_{TOT} = spatial coefficient of all sides = $1.0 + [S_{side\ 1} + S_{side\ 2} + S_{side\ 3} + \ldots$ etc.]

	Separation Distance	Spatial Coeff.
N Side	10.0	0.00
E Side	10.0	0.00
S Side	10.0	0.00
W Side	10.0	0.00

Stot = 1.0 + [0 + 0 + 0 + 0] = 1.00

Q = 214,815 L, look this value up in Table 2 to determine the Required Minimum Water Supply Flow rate (L/min)

DEL19-072: 1425 Creamery Rd.

Rational Method Computations for Estimating Stormwater Attentuation Volume

By: Jon Bakker, P.Eng.
Date: February 20, 2024

Existing Conditions Peak Runoff Calculator (CofL IDF Parameters)

Storm Event	A	B	C	Tc (min)	i (mm/hr)
100	2619.363	10.500	0.884	32.4*	94.43
Ave. Runoff 'C'	**Area (hectares)**	**Peak Flow (l/s)**			
0.20	1.00	52.0			

**Time of Concentration Estimated using the Airport Method*

Post-Development Conditions Peak Runoff Calculator (CofL IDF Parameters)

Storm Event	A	B	C	Tc (min)	i (mm/hr)
100	2619.363	10.500	0.884	7.0*	208.62
Ave. Runoff 'C'	**Area (hectares)**	**Peak Flow (l/s)**			
0.87	1.00	504.0			

**Time of Concentration Estimated using the Airport Method*

Inflow vs. Outflow Storage Calculation

Release Rate (L/s)	Maximum Storage Req'd (m3)
52.0*	352

**Existing Conditions Peak Runoff*

Storm Duration	Runoff Rate (m3/s)	Runoff Vol. (m3)	Release Vol. (m3)	Req'd Storage (m3)
35	0.217	455.70	109.20	347
36	0.213	460.08	112.32	348
37	0.209	463.98	115.44	349
38	0.205	467.40	118.56	349
39	0.201	470.34	121.68	349
40	0.198	475.20	124.80	351
41	0.194	477.24	127.92	350
42	0.191	481.32	131.04	351
43	0.188	485.04	134.16	351
44	0.185	488.40	137.28	**352**
45	0.182	491.40	140.40	351
46	0.179	494.04	143.52	351
47	0.176	496.32	146.64	350
48	0.174	501.12	149.76	**352**
49	0.171	502.74	152.88	350
50	0.169	507.00	156.00	351
51	0.166	507.96	159.12	349
52	0.164	511.68	162.24	350
53	0.161	511.98	165.36	347
54	0.159	515.16	168.48	347
55	0.157	518.10	171.60	347
56	0.155	520.80	174.72	347
57	0.153	523.26	177.84	346
58	0.151	525.48	180.96	345
59	0.149	527.46	184.08	344
60	0.147	529.20	187.20	342

SEPTIC SYSTEM SETBACK REQUIREMENTS
ONTARIO BUILDING CODE, ARTICLE 8.2.1.6:

8.2.1.6. Clearances for a Class 4 or 5 Sewage System

(1) Except as provided in Sentences 8.2.1.4.(1) and (2), a *treatment unit* shall not be located closer than the minimum horizontal distances as set out in Table 8.2.1.6.A.

Table 8.2.1.6.A.
Minimum Clearances for Treatment Units
Forming Part of Sentence 8.2.1.6.(1)

Column 1	2
Object	Minimum Clearance, m
Structure	1.5
Well	15
Lake	15
Pond	15
Reservoir	15
River	15
Spring	15
Stream	15
Property Line	3

▼ **(2)** Except as provided in Sentences 8.2.1.4.(1) and (2), a *distribution pipe* shall not be located closer than the minimum horizontal distances set out in Table 8.2.1.6.B. and these distances shall be increased when required by Sentence 8.7.4.2.(11).

Table 8.2.1.6.B.
Minimum Clearances for Distribution Piping
Forming Part of Sentence 8.2.1.6.(2)

Column 1	2
Object	Minimum Clearance, m
Structure	5
Well with a watertight casing to a depth of 6 m	15
Any other well	30
Lake	15
Pond	15
Reservoir	15
River	15
Spring not used as a source of *potable* water	15
Stream	15
Property Line	3

EXISTING SERVICES	DRAWING #, SOURCE	DATE	AS CONSTRUCTED SERVICES	COMPLETION	DETAILS	No.	REVISIONS	DATE	CONSULTANT
					DESIGN BY PJB	1	ISSUED FOR CONSENT APPLICATION	APR. 17/20	DEVENG
					DRAWN BY JC/PJB	2	REVISED SITE PLAN	FEB 21/24	DEVENG
					CHECKED BY JS				
					F.BK. ***				
					FILE: DEL19-072 - BASE - PRIVATE SERVICING.DWG				

CONSULTANT OR DIVISION

London Office
41 Adelaide St. N., Unit 71
(519) 672-8310

Paris Office
31 Mechanic St., Unit 301
(519) 442-1441

development engineering (London) Limited
CONSULTING CIVIL ENGINEERS

ENGINEER'S STAMP

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SCALE
SCALE - 1:1000
10 0 20m

TITLE
DANCOR CREAMERY INC.
1425 CREAMERY ROAD

PRIVATE SERVICING LAYOUT PLAN

PROJECT No. DEL19-072
SHEET No. 1
PLAN FILE No.

jsmith Feb.20/24-5:30pm DEL19-072 - BASE - Private Servicing.dwg