APPENDIX C
Supporting Documentation

Site Geotechnical Review

DILLON CONSULTING

MEMO

TO: Magda Bielawski

FROM: Bram Bontje
Louis Tasfi

DATE: November 10, 2010

SUBJECT: Acton Wastewater Treatment Plant Class EA – Site Geotechnical Review

OUR FILE: 06-6413

Dillon was provided by Halton Region with the 2010 *Geotechnical Investigation: Acton WWTP Expansion* prepared by Peto MacCallum Ltd. Dillon reviewed this report for information regarding the following constructability issues related to proposed Acton WWTP expansions:

- Construction of additional aeration tanks on the location of the existing settling pond as indicated in the Preferred Solution site plan (Option 1)
- Construction of additional tanks and process buildings on unused land towards south end of the property, as indicated in the Preferred Solution (Option 2)

It was noted that the Peto MacCallum report was prepared primarily to address constructability issues related to the new headworks building. The report addresses foundation requirements for a one-story 9x14 m structure to be constructed adjacent to the existing screening building. No specific consideration is given to tank or building construction elsewhere on the site.

General Comments

Based on the information included in the Peto MacCallum report it is not possible to comment on feasibility of constructing tanks on the site of the existing pond, as outlined in Preferred Solution (Option 1). The following outstanding issues were not addressed in the current geotechnical investigation which may impact the feasibility of future construction:

- The Peto investigation did not extend to the pond area. Therefore, it is not possible to comment on the constructability of tanks within this area;
- No specific recommendations for the design and construction of tank structures are provided;
- The Peto report investigation identified a high groundwater level; however, there were no firm recommendations on constructability issues or design requirements to resist buoyant uplift for future structures;
- The potential need for a large excavated volume to remove saturated soil was not addressed;
- Potential difficulties related to the disposal of sludge currently present in the lagoon were not addressed;
- Potential requirements for large quantities of non-native soil and granular backfill materials during construction were not addressed.

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The discussion in the Peto MacCallum report for the future expansion is preliminary at best and does not provide specific recommendations for the design of tank structures. Based on available data provided in the report the following general comments can be made:

- Site soil quality is generally poor;
- Ground water table elevation is high. This will have a significant impact of constructability, by increasing dewatering requirements and the design of permanent structures to resist buoyant uplift;

Additional concerns related to the existing pond:

- The pond may eventually have to be decommissioned.
- Dewatering the pond may have a major impact on overall cost; water quality may be such that it requires treatment prior to discharge to a receiver
- Condition of the pond's liner is unknown. Consequently, water in the pond may be interconnected to the groundwater table. This may have an impact on pond dewatering and construction.

It is recommended that an additional geotechnical investigation be conducted to investigate soil conditions at the location of proposed upgrades. This investigation would be required before any definite conclusions related to constructability could be made. At this stage we can only carry a contingency estimate related to constructing a tank on the pond area. The actual cost can only be confirmed once the above technical issues are further investigated.

Design Summary

1. Hydraulic load according to the Proposed Design Criteria

Table 1: Population Projections for ultimate area serviced by the Acton Wastewater Treatment Plant

Estimated Residential Population Growth (2009 – Mature State)	4880 persons
Estimated Institutional/Commercial/Industrial (ICI) growth (2009 – Mature State)	50 ha

Table 2: Estimated per-capita sewage production for future population [l/cap/d]

Per Capita Sewage Flow [l/(person*day[1])]	365

[1]Per capita sewage flow provided by Halton Region

Table 3: Projected Nominal Design Flow rates [m^3/d] for Acton Wastewater Treatment Plant

Flow Rates for Design (m^3/d)	
Average Day Flow Rate	7000
Maximum Day Flow Rate (@ peaking factor of 2.95[1])	20650
Peak Hour Flow Rate (@ peaking factor of 3.67[2])	25687

[1]Peaking factor following earlier design specifications (Earth Tech, 2006)
[2]Peaking factor for estimated Peak Hour Flow (AECOM, 2008)

2. Contaminant Load and Effluent Objective

Table 4: Design Wastewater Contaminant Loads for Acton WWTP Biological System

	Sewage Load		
Parameters	**Design Loading (g/cap/d)**	**Average Daily Mass Load from sewer loads (kg/d)**	**Conc'n at Average Flow & Load (mg/L)**
Q (l/cap/d)	365		
BOD_5	85	1245.5	177
Suspended Solids	95	1460.7	208
TKN	13.3	252.5	36.6
NH_3-N	7.8	160.6	22.9
Total Phosphorus	3.28	44.1	6.3

Table 5: Treated Effluent Objectives and Compliance Limits for Acton WWTP[1]

Parameter	Effluent Objective	Effluent Limit
BOD_5	2 mg/L	5 mg/L
TSS	2 mg/L	5 mg/L
Total Phosphorus*		
Short Term (5,600 m^3/d)	0.1 mg/L (204 kg/yr)	0.2 mg/L (409 kg/yr)
Long Term (7,000 m^3/d)	0.1 mg/L (255 kg/yr)	0.2 mg/L (511 kg/yr)
(Ammonia + Ammonium) Nitrogen**		
Non-freezing period (May 1 – Nov. 31):	0.5 mg/L as N	2.0 mg/L as N
Freezing period (Dec. 1 – April 30):	1.0 mg/L as N	4.0 mg/L as N
Escherichia Coli (monthly geometric mean density)	100 organisms/100mL	150 organisms/100mL

*It is understood that the total phosphorus loading objective to the receiver will be maintained at its current loading of 156kg/yr.

** The corresponding un-ionized ammonia values (based on effluent pH and temperature) are as follows:

- ammonia objective always meets the PWQO for unionized ammonia of 0.016 mg/L (or 0.02 mg/L as NH_3)
- ammonia limit always meets the acute target value for un-ionized ammonia of 0.08 mg/L as N (or the current single sample compliance limit of 0.1 mg/L as unionized NH_3).

[1]Design objectives and compliance limits obtained from Black Creek Assimilative Capacity Study (Dillon, 2011)

3. Design Parameters for Process Units

Note: Inlet works process units not included within scope of present upgrade.

3.1 Extended Aeration Process

3.2 Primary Clarification

Design flow:	
Maximum day flow:	20650 m^3/d
Average day flow:	7000 m^3/d
Number of tanks:	5 (2 existing, 3 new)
(Primary clarifier will co-thicken waste activated sludge)	
Existing tanks:	
Surface area:	60.4 m^2
New tanks:	
Length:	24 m
Width:	5 m
Depth:	4 m
Total area (including existing tanks):	480.8 m^2
Total volume (including existing tanks):	2004.8 m^3
Total hydraulic retention time (maximum day flow):	1.87 h
Total hydraulic retention time (average day flow):	6.87 h
Total surface hydraulic load at max day flow:	42.9 $m^3/m^2 \cdot d$
(MOE guideline 2008, WAS co-thickening option):	60 $m^3/m^2 \cdot d$
Inlet BOD:	1245.5 kg/d
Outlet BOD to biological process:	996 kg/d
BOD removal efficiency:	20%
Inlet TSS:	1460.7 kg/d
Outlet TSS to biological process:	730 kg/d
TSS removal efficiency:	50%

3.2.1 Biological Process

Design flow:	
Maximum day flow:	20650 m^3/d
Average day flow:	7000 m^3/d
Contaminant loading at ultimate average day flow	
BOD (20% reduction in primary clarifier):	996 kg/d

TSS (50% reduction in primary clarifier):	730 kg/d
NH3-N:	160.6 kg/d
TKN:	262.5 kg/d
Design temperature:	
Minimum:	8 ºC
Number of tanks:	7 (4 existing, 3 new)
New tank dimensions:	
Water depth:	5.5 m
Side wall depth:	6.0 m
MLSS:	$4 kg/m^3$
Biomass (MLVSS) required for nitrification: (8 ºC @ 0.03 g/(g·d))	8750 kg/d
Sludge inventory (MLSS):	17052 kg
Sludge inventory (MLVSS @ 75%):	12790 kg
Safety factor for nitrification:	1.5
F to M ratio (based on MLVSS):	0.08 kg/kg·d
(MOE guideline 2008):	0.05 to 0.15 kg/kg*d
Waste sludge:	
Biological:	796 kg/d
Chemical:	284.4 kg/d
Total:	1080.4 kg/d
Sludge age:	21.6 d
New tank dimensions per train L x W x H:	34m x 8.5m x 5.5m
Process zone volumes:	
Pre-anoxic zone (existing tanks):	$988 m^3$
Aeration zone:	$4263 m^3$
Swing anoxic-aerobic zone:	$290 m^3$
Re-aeration zone:	$290 m^3$
Total volume:	$5831 m^3$
Treatment zone hydraulic retention times:	
Pre-anoxic zone:	
Average day flow:	3.39 h
Maximum hour flow:	0.92 h
F to M ratio:	0.33 kg/kg·d
(MOE guideline 2008):	0.5 to 1.0 d^{-1}

Aeration zone (including swing and re-aerate zones):	
Average day flow:	16.6 h
Maximum hour flow:	4.52 h
(MOE guidelines 2008):	15h at average day flow
Swing anoxic-aerobic zone:	
Average day flow:	1 h
Maximum hour flow:	16.3 min
Re-aeration zone:	
Average day flow:	1h
Maximum hour flow:	16.3 min
Total volume:	5834 m^3
New tank volume	4843 m^3
Existing tank volume (pre-anoxic zone):	988 m^3

3.2.2 *Aeration System*

Average design BOD load:	996 kg/d
Peak design BOD load (peaking factor of 2.0):	1992 kg/d
Design TKN load:	262.5 kg/d
Peak TKN load (peaking factor of 2.0):	534 kg/d
AOR:	
For BOD (@1.0 kg/kg):	
For nitrification (@4.6 kg/kg):	
Total AOR (@ peak loading):	3959 kg/d
Average WL above diffusers:	5m
Alpha:	0.6
Beta:	0.95
Maximum design temperature for aeration:	20°C
O_2 concentration (@ peak loading):	1 mg/L
Site elevation:	335m
AOR/SOR ratio	39.2%

SOR:	8907 kg/d
SOTE:	25%
Maximum air required:	132696 m^3/d
Number of blowers for biological process:	2 duty, 1 standby
Diffuser type:	Fine bubble

3.2.3 Aeration Blower

Blower type:	Positive displacement
Blower configuration:	2 duty, 1 common standby
Maximum combined blower capacity:	141408 m^3/d
Min airflow (1 unit operating):	43200 m^3/d
Motor rating (each):	75 kW
Electrical:	575/3/60
Starter:	VFD
Turndown:	1.64:1

3.2.4 Mixers

Pre-anoxic zone:	
Mixed volume:	988 m^3
Power demand (@ 30 W/m^3):	29.6 kW
Swing anoxic-aerobic zone:	
Mixed volume:	290 m^3
Power demand (@ 30 W/m^3):	8.7 kW

3.2.5 Secondary Clarifiers

Design considerations:	
Maximum hour flow:	25687 m^3/d
Maximum day flow:	20650 m^3/d
Maximum MLSS in aeration tank:	4.0 kg/m^3
Underflow solids concentration:	8.5 kg/m^3
Maximum return rate:	100%
Maximum volumetric return rate (max day flow):	25687 m^3/d
Clarifier surface area:	
Existing tanks:	308 m^2
New tanks:	629 m^2
Total area:	937 m^2
Number of clarifiers:	5 (2 existing, 3 new)
New clarifier depth:	3.6 m
Total clarifier volume:	3373 m^3

Effluent TSS:	10 mg/L
Hydraulic retention time:	
Average day flow:	11.6h
Maximum day flow:	3.9h
Maximum hour flow:	3.2h
New clarifier dimensions:	
Tank length per clarifier:	29 m
Tank width per clarifier:	7.24 m
Length to width ratio:	4.0
(MOE guideline 2008):	4
Width to Depth ratio:	2.01
(MOE guideline 2008):	1 to 2.25
Surface hydraulic load at maximum hour flow (total):	27.41 m^3/m^2*d
(MOE guideline 2008):	≤40 m^3/m^2*d
Surface solids load at maximum day flow (total):	167.5 kg/m^2*d
(MOE guideline 2008):	≤170 kg/m^2*d

3.2.6 New Return Activated Sludge Pumps

Number of new tanks:	3
Number of RAS pumps per new tank:	1
Total number of new pumps:	3
Flow to new secondary clarifiers:	
Percentage of flow total flow:	37%
Flow per new clarifier at maximum day flow:	4621 m^3/d
Flow per new clarifier at average day flow:	1566 m^3/d
Maximum return rate:	100% return at maximum day flow
Maximum return rate per pump:	4621 m^3/d
Minimum return rate:	90% return at 50% of 2009 average day flow
Minimum return rate per pump:	426.5 m^3/d
Pump type:	Vertical dry pit centrifugal
Pump configuration:	3 operating (one per new clarifier, 1 standby)
Average return rate per pump:	1566 m^3/d
Total capacity:	13863 m^3/d
Design maximum flow per pump	4621 m^3/d
TDH:	5 m
Motor rating (each):	5.6 kW
Electrical:	575/3/60
Starter:	VFD

3.2.7 New Waste Activated Sludge Pumps

Maximum wastage rate per pump:	462 m^3/d
Minimum waste rate per pump:	47.4 m^3/d
Average return rate per pump:	156.6 m^3/d

3.3 Tertiary Treatment:

3.3.1 Sand Filtration

Design flow:	
Maximum hour flow:	25,687 m^3/d
Inlet TSS concentration:	10 mg/L
Inlet TP concentration (solids and soluble):	1 mg/L
(chemical addition to aeration tank will reduce soluble TP – see section 3.4.2)	
Effluent TSS concentration:	5 mg/L
Effluent TP concentration:	0.1 mg/L
Type:	Granular sand filter
Unit dimensions (L x W x D):	7.22 m x 19.41 m x 7.17 m
Number of cells:	6
Number of modules per cell:	4
Number of distinct feed/filtrate units:	6
Total filtration area:	111.5 m^2
Hydraulic surface loading:	2.66 L/m^2/sec (with 6 cells online)
	3.2 L/m^2/sec (with 1 cell offline)
(MOE guideline 2008):	3.3 L/m^2/sec (with once cell offline)
Solids surface loading :	26.6 mg/(m^2·s)
(MOE guideline 2008):	83 mg/(m^2·s)
Filter reject flow:	38.16 m^3/d to 76.32 m^3/d

3.3.1.1 Tertiary filter compressor

Compressor configuration:	
Maximum blower capacity:	2.35 m^3/min
Type:	Rotary screw
Motor rating:	14.9 kW

3.3.2 Ultraviolet Disinfection

Configuration:	Open-channel, horizontal

Water level control:	Serpentine weir
Maximum flow capacity:	25687 m^3/d
Maximum hour flow:	25687 m^3/d
Maximum day flow:	20650 m^3/d
Design avg. flow:	7000 m^3/d
Minimum UVT:	65%
Design effluent quality:	100 cfu/ 100mL
Number of banks:	2
Number of modules per bank:	5

3.4 Anaerobic Digestion

Total waste sludge to digesters:	1753.7 kg/d
Primary sludge (assuming 50% TSS removal):	730 kg/d
Waste activated sludge:	796.8 kg/d
Chemical sludge:	226.7

Design load:	
Total waste sludge load:	1753.7 kg/d
Volatile solids load:	1221 kg/d
(@ 0.8 kg VSS / kg primary and waste sludge)	

Primary digester	
Number of tanks:	2 (1 new, 1 existing)
Volume per tank:	615 m^3
Total volume:	1230 m^3
Tank type:	Steel tank with fused-glass coating

Secondary digester:	
Number of tanks:	2 (1 new, 1 existing)
Volume per tank:	340 m^3
Total volume:	680 m^3
Tank type:	Steel tank with fused-glass coating

SRT (primary digester):	24.6d
(MOE guideline 2008):	15d

3.4.1 Anaerobic Digester Mixing

Primary digester:	
Mixing type:	Jet mixing
Pump type:	Dry-pit end suction chopper pump
Motor rating:	11.2 kW

Secondary digester:	
Mixing type:	Jet mixing

Pump type:	Dry-pit end suction chopper pump
Motor rating:	7.4 kW

3.4.2 Chemical Dosage for Phosphorous Removal

Total Phosphorous Load at Average day flow:	
Sewage:	44.1 kg/d
Phosphorous in effluent:	0.7 kg/d
Biological phosphorous uptake:	6.54 kg/d
Total phosphorous to be removed:	43.4 kg/d
TP removal efficiency required:	98.4%
Al to P molar ratio:	2.5
Aluminum required:	80.28 kg/d
Alum solution demand @ 48% concentration:	1.380 m^3/d
Mass of alum required at S.G. of 1.2 kg/L:	1.842 kg/d
Alum dosage:	212 mg/L
Chemical sludge produced:	
: Aluminum phosphate:	145.13 kg/d
Aluminum hydroxide:	139.25 kg/d
Total chemical sludge:	284.38 kg/d

3.5 Effluent Pump Station (provisional)

Design flow:	
Peak hour flow:	25687m^3/d
Dimensions (L x W x H):	3.6m x 2.4m x 4.0m
Surcharge chamber (Dia x H):	1.5m x 5m
Number of sumps:	1

3.5.1 Effluent Pumps (provisional)

Configuration:	(3 duty 1 standby)
Pump type:	Submersible horizontal impeller
Maximum flow per pump:	8588m^3/d
TDH:	5m

Peak Flow Management

MEMO

TO: Magda Bielawski

FROM: Bram Bontje
Louis Tasfi

DATE: March 4, 2011

SUBJECT: Acton Wastewater Treatment Plant Class EA – Phase 3, Peak Flow Management

OUR FILE: 06-6413

Technical Memorandum No. 2 was prepared to address Phase 3 of the Acton Wastewater Treatment Plant Class EA. This memo considered a variety of alternative design concepts for major process components. Flow equalization was considered to address the potential need for hydraulic buffering capacity in the event of high flows.

Flow equalization requirements were assessed based on the following information provided to Dillon by Halton Region:

- 2008 and 2009 five-minute interval influent flow data;
- 2005 to 2009 Acton bypass summary;
- Environment Canada Guelph Ontario Intensity-Duration-Frequency (IDF) curve;
- Acton precipitation summaries for Q3 2008, Q4 2008 and Q1 2009;
- Acton Inlet Works Expansion Pre-Design report (R.E. Poisson, 2010);
- Town of Acton Hydraulic Analysis and Capacity Assessment – 2021 and Mature State (AECOM 2008); and
- Capital Needs Assessment (Earth Tech Canada, 2006).

Bypass events reported during 2008 and 2009 were identified. Influent flow data, from days on which bypass events occurred, were analyzed using the following method:

- The peak instantaneous flow was identified by locating the maximum flow recorded on the "five minute data" worksheet included in the 2008 and 2009 influent flow data;
- The maximum hourly average from the day of the bypass event was located and recorded as peak hourly flow; and
- A peaking factor was calculated by dividing peak instantaneous flow by the current design flow (4,545 m^3/d).

The maximum peaking factor recorded from the analysis of 2008 and 2009 bypass events is 2.42. It was confirmed that peaking factors did not exceed 2.42 at any point over the monitoring period.

The original Acton WWTP Certificate of Approval, as referenced by the 2006 Earth Tech Canada Capital Needs assessment report, lists a peak plant capacity of 13,410 m^3/d. This corresponds to a peaking factor of 2.95 above the rated capacity of 4,545 m^3/d. Bypass events have been repeatedly seen at peaking factors of approximately 2.42 (see Table 1) suggesting operational deficiencies with the current facility. Tertiary bypasses make up all observed events, suggesting that tertiary filters may be undersized.

Table 1: Selected Precipitation and Bypass Events during 2008 and 2009

Date	Bypass	Precipitation duration (h)	Average Precipitation Intensity (mm/hour)	Maximum Peaking Factor
Jul 19 2008	No	23.4	2.8	1.93
Jul 22 2008	No	0.4	57.8	1.84
Aug 5 2008	No	8.7	5.2	1.84
Dec 28 2008	Yes	15.4	0.9	2.40
Feb 11-12 2009	Yes	20.2	1.4	2.34
Apr 29 2009	Yes*	NA	NA	1.67
Aug 9 2009	Yes	NA	NA	2.39
Aug 11 2009	Yes	NA	NA	2.42

** Bypass due to equipment maintenance*

Of a total of fifteen bypass events reported between 2005 and 2010, seven were directly influenced by precipitation. Five precipitation or snowmelt-related bypass events were reported during 2008 and 2009, over a period where five-minute interval influent flow data is available.

Available rainfall data was also compared to the Environment Canada IDF curve for Guelph, Ontario to assess plant performance under "design storm" conditions. The following design storms were identified:

- 5-year design storm on July 19, 2008
  - Long duration (23.4h)
  - Peaking factor : 1.93

- 2- year design storm on July 22, 2008
  - Short duration (0.4h)
  - Peaking factor: 1.84

- 2-year design storm on August 5, 2008
  - Long duration (8.7h)
  - Peaking factor: 1.84

None of the three identified design storms produced a bypass event. It is interesting to note that in 2008 there were two 2-year storm events and one 5-year storm event identified, representing a greater frequency than is suggested by IDF curve data. It was originally believed that bypass events would correspond to recorded design storms, but this was not the case. As a result, it was not possible to assess flow equalization requirements for bypass mitigation based on a standard rainfall event. Additional design data available from earlier studies was used to produce a more conservative design basis.

Design Basis

The current headworks upgrade project has assumed a maximum inlet flow of 26,000 m^3/d, corresponding to peak wet weather flow for the mature state. This is detailed in the *Acton Inlet Works Expansion Pre-design Report* (R.E. Poisson, 2010). Peak and average wet weather flows to the facility are estimated from data provided in the *Town of Acton Hydraulic Analysis and Capacity Assessment – 2021 and Mature State* (AECOM 2008). The AECOM report made the following flow assumptions:

- Average wet weather (design) flow of 20,952 m^3/d
  - Peaking factor of 2.99 above the current design flow of 7,000 m^3/d.

- Peak wet weather flow of 25,687 m^3/d
  - Peaking factor of 3.67 above the current design flow of 7,000 m^3/d.

In preparing their peak flow estimates, AECOM assumed wet weather infiltration resulting from a hypothetical 25-year design storm following a Chicago distribution. Dillon's analysis of available flow monitoring and rainfall data for 2008-2009 was not sufficient to identify an appropriate design storm and to confirm AECOM's wet weather flow estimates. Therefore, the data provided by AECOM was used as it provides a conservative estimate for wet weather flows.

The peaking factors which were considered when assessing requirements for flow equalization were as follows:

- 2.95 (20,650 m^3/d) – average day peak hydraulic capacity. This corresponds to the design peaking factor for the current facility. It is also similar to the average day wet weather flow value proposed by AECOM.

- 3.67 (25,687 m^3/d) – maximum peak hydraulic capacity. This value corresponds to the peak wet weather flow which was estimated by AECOM for a 25-year design storm.

Suggested Mitigation Options

Option 1: Flow Equalization Tank:

Flow equalization may be provided to handle plant flows in excess of a peaking factor of 2.95. For the purposes of tank sizing, it is assumed that the equalization tank should be able to accommodate excess flows at a peaking factor of 3.67 for two hours, based on the duration and intensity of the Chicago design storm used by AECOM in calculating peak flow. The total equalization tank volume required under this condition is **420 m^3**. Primary and secondary clarifiers, tertiary filters and UV unit will be sized with a maximum and peak hour flow rate of 20,650 m^3/d. Any flow above this value (i.e. instantaneous peaks) will be shaved off by the equalization tank.

It should be noted that flow equalization is only intended to mitigate peak flows and is not suggested as a method to remove all inlet flow fluctuations to the facility. Such an arrangement would require far greater equalization tank capacity than is proposed in this memo and would likely be impractical.

Option 2: Increased Hydraulic Capacity:

To accommodate all peak flows to the facility without separate flow equalization tankage, the primary clarifier, secondary clarifier, tertiary filter and UV unit capacities would be increased to accommodate flows at a peaking factor of 3.67. These unit operations will be sized for:

- Maximum day flow of 20,650 m^3/d;
- Peak hourly flow of 25,687 m^3/d.

A detailed comparison of flow equalization alternatives is provided in Table 2.

Table 2: Evaluation of Peak Flow Management Alternatives

Evaluation of Peak Flow Management Alternatives	Minimal to no impact/technically preferred (green)	Some impact/technically less preferred (orange)
Criteria/Indicator	**Increase plant capacity**	**Construct flow equalization tank**
Protection of the Cultural and Socio-Economic Environment		
Displacement or disruption of any archaeologically significant findings	Minimal since within property boundary	Minimal, provided space available on property
Displacement or disruption of cultural heritage features	None	None
Potential visual-aesthetic impact associated with new construction (added footprint of new tankage and buildings, new building and tankage height)	Limited visibility of new tankage	Limited visibility of additional tankage
Potential short term disruption (noise, dust, odour, traffic) during construction	Both have similar construction impacts which could be mitigated	Both have similar construction impacts which could be mitigated
Potential long term disruption (noise, dust, odour) during operation	Minimal impacts associated with operation that could be mitigated	Some potential for odour impacts
Protection of the Natural Environment		
Potential of the alternative to minimize adverse impacts to the receiving water quality and aquatic systems	Minimal	Minimal
Potential for impact on terrestrial or aquatic habitat	Minimal	Minimal
Technical Performance		
Ability of the technology to meet the MOE definition of 'proven technology'	Both proven, well established treatment processes	Both proven, well established treatment processes
Relative ease to implement/construct and maintain/operate proposed technology within existing treatment plant	Does not present additional concern	Some concern of solids accumulation within tank
Relative ease at which the plant could be expanded for the alternative, (including new tankage and buildings and to meet more stringent effluent criteria)	Does not present additional concern	Potential expandability concerns due to land requirements
Ability of the treatment process to handle variable loadings and flows	Both provide a reliable form of treatment	Both provide a reliable form of treatment

Cost		
Estimated capital cost (excluding common costs)	$5.7 M	$5.3 M
Estimated or relative annual operating and maintenance costs (excluding common costs to provide an incremental cost)	--	$2000 /yr
Estimated lifecycle cost (over a 20-year period) based on above costs	$5.7 M	$5.4 M
Overall Evaluation	***Preferred***	***NOT Recommended***

The design requirements for the expansion of unit operations does not change as a result of increasing the maximum hourly peaking factor from 2.95 to 3.67. Both primary and secondary clarifiers are required to accommodate peak daily flows which remain constant under both of the alternatives proposed above. Consequently, the size of both types of clarifiers is the same. The size of tertiary filters and UV units are different between the two options as they have to accommodate peak hourly flows.

Constructing the system for an increased peak flow capacity is preferred over the option that presents a system that is sized for a lower peaking factor with a flow equalization tank. A flow equalization tank may result in slightly lower up-front capital cost, however, it has many disadvantages as listed below:

- may cause operational issues related to odour generation and solids accumulation;
- land availability is limited, therefore there could be issues related to finding space for this tank on the existing property;
- additional future facility expansions may be difficult if available land is allocated to an EQ tank during the Phase 1 upgrade; and
- ongoing operation and maintenance costs (pumping is required).

Region Wide Odour Management Strategy

ZORIX ENVIRONMENTAL

Specialists in Odour and VOC Issues

Region-Wide Odour Management Strategy (OMS) for Halton Wastewater Treatment Plants
File # PR-2336A

A Report to:	Regional Municipality of Halton
Attention:	Susan Liver, P.Eng. Project Manager- Public Works 1151 Bronte Road Oakville, Ontario L6M 3L1
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From:	Michael R. E. Rix, B.Sc., QEP Specialist in Odour Impact Assessments And Moe Zolghadr, Ph.D., C.Eng., QEP Specialist in Odour Impact Control In collaboration with: CH2M HILL Canada Limited
Report No.:	07- 0166 R (Final)
Date:	March 19, 2010

2.2 Quantification of Odour

There are many challenges in the technologies of quantifying odour impact because of the complexities inherent to odour perception and atmospheric dispersion of emissions. One approach to quantify odour and establish odour criteria is to use the concentrations of individual odorous chemical species. For wastewater treatment, for example, criteria based on hydrogen sulphide concentration have been used. Chemistry-specific criteria are of limited usefulness, however, since wastewater treatment odours are normally composed of a mixture of compounds, and the threshold concentration of odour detection for a mixture can be very different from the pure compound. There is also wide variation among individual persons as to the concentration of an odorous compound that is detectable.

Achieving a target expressed in terms of one particular contaminant (e.g. hydrogen sulphide), or even several contaminants, can be useful for the engineering design of equipment but would not necessarily protect a facility against odour impacts and complaints.

Often, sensory odour measurements are more pertinent and predictive than chemical testing, since they correlate directly to the likelihood of an offensive odour being detected by people in the community. The most commonly recognized and standardized approach to sensory odour measurement is based on a panel of people with standardized olfactory sensitivity, who smell diluted quantities of air collected from the odorous source. The odour detection threshold (ODT) is the lowest concentration of a certain odour compound that is perceivable by the human sense of smell. It is determined using an odour panel and is the value at which 50% of the panel detect an odour. The ODT is expressed as the value of dilutions to threshold (D/T) or OU or OU/m^3.The procedure for determining ODT using an odour panel is specified in the European Standard EN 13725 [5]. Many jurisdictions, including Ontario, have used this approach for many years; it was also the primary odour measurement method used in the odour assessments for Halton's WWTPs.

Odour panel OU results are typically used to calculate an odour emission rate (OU/s) for the odorous source; this rate can then be processed with a model in conjunction with atmospheric dispersion models to predict off-property impacts in OU.

2.3 Odour Criteria

Although by definition half of the population can detect an odour at a level of 1 OU/m^3, negative impacts and complaints do not tend to occur at this level. Published scientific studies [6, 7, 8] indicate that the threshold where complaints begin to occur as result of odours from WWTPs varies, with values between 4.2 OU/m^3 and 20 OU/m^3 reported.

The possibility of an odour release having a negative impact or being likely to generate complaints depends on a number of factors, commonly summarized as the acronym "FIDOL":

Frequency (how often the odour occurs)
Intensity (related to odour concentration)
Duration (how long an odour lasts)
Offensiveness (related to pleasantness or unpleasantness and odour character)
Location (where the odour is detected, or the context in which the odour occurs)

Ideally, odour criteria for compliance should consider all of these factors.

Frequently, conditions for odour in MOE C of As for industrial facilities have required odour not to exceed 1.0 OU at the property line or at the nearest sensitive receptor at all times; in other cases, the MOE has deemed higher OU values as acceptable. The MOE assesses whether conditions comply with the criterion through dispersion modelling of source data that is derived from odour panel testing; the models are generally used in conjunction with 5 years of regionally specific historical meteorological data. The compliance criterion applies to the predicted "worst-case" value using a 10-minute average. This approach ignores the other FIDOL parameters that are factors in whether an odour impact occurs, and results in unnecessarily onerous odour control requirements for compliance.

Dispersion modelling allows potential impacts to be quantified in terms of all of the FIDOL parameters, except Offensiveness, which can be accounted for by considering the type of odour source or process.

In the United Kingdom (UK), a set of three odour level/ frequency targets has been proposed, depending on whether the odour annoyance levels associated with an industrial sector are considered to be low, medium, or high. [9] Odour targets are then expressed as 98th percentile limits, *i.e.* levels that are only exceeded 2% of the time, based on one-hour averaging times. Proposed UK levels are as follows:

Odour Annoyance Potential	Criterion
High	98^{th} percentile, 1-hour ≤ 1.5 OU
Medium	98^{th} percentile, 1-hour ≤ 3 OU
Low	98^{th} percentile, 1-hour ≤ 6 OU

Odour criteria of this nature are more directly applicable to the objective of preventing offensive odour impacts to neighbours of an odour-generating facility.

2.4 Recommended Metrics for Odour Management Performance

Odour standards need to protect community members from odour impacts in an economically viable way. Ultimately, the purpose of odour management is to prevent offensive impacts. For existing conditions, this is sometimes better measured qualitatively through resident feedback and subjective perception, rather than through numerical prediction.

As an indirect measure of odour management performance, complaint records should be reviewed each year.

Given the complexity of establishing appropriate odour criteria as outlined in the preceding sections, quantitative target criteria are not proposed for Halton's WWTPs.

However, prediction of ambient OU level and frequency of occurrence is recommended as a means to assess the relative degree of potential odour impact associated with each Halton plant and odour source, and to gauge the relative benefit of potential odour control improvements and assist in decision-making.

A plant-specific approach for developing odour criteria should be adopted, recognizing pertinent factors related to each plant. For example, the number and density of nearby receptors, the plant's setting, and the complaint history should be considered.

C.1 Acton WWTP

C.1.1 Baseline Monitoring Results

Table C.1 summarizes the odour and hydrogen sulphide emission rates as determined during the 2005 odour assessment at the Acton WWTP. Most of the odours were from the aeration processes at the facility.

C.1.2 Baseline Modelling Results

Table C.2 below outlines the key source components and emission rates as entered into AERMOD for the impact assessment of Acton's WWTP. The results of baseline modelling (Figure C.1) show no odour impact is expected in the surrounding community. Hydrogen sulphide modelling results show concentrations of less than $0.5\mu g/m^3$ outside the plant's property line, well below the Ontario standard of $30\mu g/m^3$. At the most impacted sensitive receptor the maximum predicted concentration will be 1.2 OU. Table C.4 outlines the 98^{th} and 99^{th} percentile concentrations at the most impacted sensitive receptor. These values represent concentrations that would not be exceeded 98% or 99% of all hours respectively.

C.1.3 Complaint Mapping

The Region of Halton has not received any complaints regarding operations at Acton's treatment plant.

C.1.4 Region of Halton Staff Observations from Odour Workshop

In 2005 when the odour study was conducted at Acton the digesters and drying beds were out of service. They were put back into service in June, 2006. The impact of these sources is not known. Sludge loading is considered to be the most significant source.

C.1.5 Conceptual Design for Infrastructure Changes

No new remedial odour control measures are proposed for Acton at this time. Off-property impact modelling results and historical complaint records suggest that the surrounding receptors are not significantly impacted by odour from the Acton plant.

This recommendation is contingent on the assumption that the Acton facility's drying beds are not a problematic source of odour. No data was available for this potential source as it has not been previously assessed, and thus the modelled odour impacts do not include any contribution from the existing drying beds. It is recommended that an investigation of the drying beds be undertaken to confirm no controls are required.

It is understood that expansion of the Acton WWTP is expected to be started by 2010. If and when construction of expansion or upgrade facilities is undertaken for Acton WWTP, it is recommended that odour controls be designed into any primary odour sources. A larger facility with a corresponding increase in total air emissions may elevate the off-site impacts sufficiently to cause a problem where one does not now exist, unless the average level of control is increased. Retrofitting controls to the existing facilities is not warranted at this time.

C.1.6 After Odour Controls Frequency Plot

Because it was determined no remedial measures are need at Acton's WWTP, the frequency plot will not change.

C.1.7 Cost Estimates

No additional cost will be needed in order to reduce odours generated at Acton's WWTP.

Table C.1 – Summary of 2005 Odour Assessment

Source	Odour Emission Rate (OU/s)	Percent of Total (%)	H_2S Emission Rate (g/s)	Percent of Total (%)
Primary Clarifiers	38	9%	4.61E-08	30%
Aeration Tanks	317	80%	0	0%
Secondary Clarifiers	10	3%	0	0%
Grit Tank	20	5%	1.07E-07	70%
Sludge Pit	12	3%		
TOTAL	**396**	**100%**	**1.53E-07**	**100%**

Table C.2 – Acton WWTP: Baseline Scenario Dispersion Modelling Source Inputs

Source	Type	Source Group	Pollutant Unit Emission Rate		Variable Emission	H_2S Emission Rate (g/s)
Primary Tank 1	Area	Primary Tanks	Odour	0.263 OU/s/m^2	No	2.31E-08
Primary Tanks 2-3	Area	Primary Tanks	Odour	0.263 OU/s/m^2	No	2.31E-08
Aeration Tanks 1-2	Area	Aeration Tanks	Odour	0.771 OU/s/m^2	No	0.00E+00
Aeration Tanks 3-6	Area	Aeration Tanks	Odour	0.771 OU/s/m^2	No	0.00E+00
Secondary Clarifier 1	Area	Secondary Tanks	Odour	0.029 OU/s/m^2	No	0.00E+00
Secondary Clarifier 2	Area	Secondary Tanks	Odour	0.029 OU/s/m^2	No	0.00E+00
Secondary Clarifier 3-4	Area	Secondary Tanks	Odour	0.029 OU/s/m^2	No	0.00E+00
Grit Tank	Area	Headworks	Odour	1.354 OU/s/m^2	No	7.79E-08
Sludge Pit	Area	Headworks	Odour	2.540 OU/s/m^2	No	2.88E-08

Table C.3 – Acton WWTP: Baseline Scenario Maximum Off-Property Impact on Sensitive Receptors Predicted by AERMOD

Source Group	Max. 10-Min Odour Conc. (OU)	Max. Conc. Receptor (UTM Projection X,m)	Max. Conc. Receptor (UTM Projection Y,m)	Max. Conc. Hour (yyyymmddhh)
All	1.22	578915	4831400	1996122215
Aeration Tanks	0.90	578915	4831400	1996112908
Headworks	0.27	579158	4831383	1996051704
Primary Tanks	0.14	579132	4831388	1996121015
Secondary Tanks	0.03	579144	4831386	1999120208

Table C.4 – Acton WWTP: 99th and 98th Percentile Odour Impacts

ACTON WWTP	**Current Baseline:** *Maximum at Sensitive Receptors (OU)*
99th Percentile 10-Minute Odour Concentration	0.77
98th Percentile 10-Minute Odour Concentration	0.64

Interpretation: 99th Percentile – At the most-impacted sensitive receptor, odours are expected to be at or below 0.77 odour units 99% of the time.

Figure C.1 - Acton WWTP Baseline Scenario Frequency Plot: Percent of Time 5 OU Exceeded