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Technical Memorandum

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Subject: **Discharge Flowrate Calculations**
EPA Application Form 2C – Wastewater Discharge Information
NPDES Permit No. WA0045586 Renewal
Lehigh Cement Company Closed Cement Kiln Dust Pile Site
Metaline Falls, Washington

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1. INTRODUCTION

The Groundwater Remedy at the Lehigh Cement Company (Lehigh) Closed Cement Kiln Dust (CKD) Pile site in Metaline Falls, Washington (the Site) is subject to a National Pollutant Discharge Elimination System (NPDES) permit to discharge treated groundwater to Sullivan Creek. As part of the 2018 NPDES permit renewal application package, Lehigh submitted Environmental Protection Agency (EPA) Application Form 2C – Wastewater Discharge Information to Ecology in February 2018. EPA Form 2C included reported Maximum Daily Flow (part V.1.a of Form 2C) and Long Term Average Flow (part V.1.c of Form 2C) discharge values for the Groundwater Remedy. This technical memo describes the basis for the calculated flowrates presented in the application package.

2. BACKGROUND

At the time of the original permit application in 2006, the projected maximum daily flow was estimated to be 130,000 gallons per day (gpd), and the daily long-term average flow was projected to be 97,000 gpd. These values were estimated based on groundwater seepage calculations presented in the Engineering Design Report (EDR) submitted in 2006. The Groundwater Remedy includes a funnel and gate groundwater barrier as well as a novel in-situ carbon dioxide diffusion system buried in a "Treatment Zone" to decrease the pH of groundwater and subsequently precipitate arsenic, chromium, and lead prior to discharge to Sullivan Creek. The Groundwater Remedy was designed to be a relatively passive treatment system that relies on the groundwater elevation gradient to move water through the Treatment Zone, and therefore the actual flowrates are influenced by overall groundwater elevation and recharge trends. The flow values were re-evaluated during the most recent permit renewal process, in consideration of the fact that the groundwater flow regime for the site has changed since the treatment system was originally constructed in 2007. Observations indicate that actual flowrates are much lower than originally projected, as described in the following sections.

3. APPROACH TO ESTIMATING CURRENT FLOWRATES

This memo summarizes the results of the following methods that were used to estimate the current amount of water flowing through the Treatment Zone and into Sullivan Creek:

- Batch release test (for estimating maximum daily flow);
- Bernoulli equation for energy balances (for estimating maximum flow);
- Recharge observations immediately following the batch release test (for estimating long-term average daily flow); and
- Darcy aquifer flow equation (for estimating average flow into the Treatment Zone).

These methods were used to evaluate the flowrates that could be expected for the treated water discharge, as real time flow monitoring is not practical given the buried outlets and slow flow velocities observed through the Treatment Zone. The calculated flowrates reported in the 2018 permit renewal application package are a culmination of the flow evaluations described here and compiled at the end of this memorandum.

4. ESTIMATING MAXIMUM DAILY FLOW

4.1 Batch Release Test

A batch release test was conducted during a regular monthly site visit in November 2017, after the water had been building up in the Treatment Zone and had stabilized at approximately 2.25 feet above the discharge pipe elevations. The batch release test was conducted using the following protocol:

1. Initial water levels were measured using a water level meter in the Treatment Zone wells. An initial water level reading was also recorded with the pressure transducers connected to the programmable logic controller (PLC).
2. The valve at TZOutlet-1 was opened fully at 13:17 on November 28, 2018.

3. After opening the valve, water levels were monitored frequently via the PLC to estimate discharge rates.
4. The valve at TZOutlet-1 was closed fully at 12:57 on November 30, 2018.
5. Final water level measurements were collected using a water level meter at the time of valve closing. A final reading was also collected via the PLC.

After valve opening, the pressure head above the outlet pipes within the treatment system decreased quickly. For each time step, the discharge volume was calculated by multiplying the water level change by the approximately 5,200 square foot (ft^2) surface area of the Treatment Zone. The volume calculation also accounted for the porosity of the gravel (0.3). Based on these observations, the discharge rate rapidly decreased from approximately 32 gallons per minute (gpm; immediately after valve opening) to 2 gpm (48 hours after valve opening) when the pressure head had decreased significantly. The discharge rates were plotted as a function of available head (Figure 1).

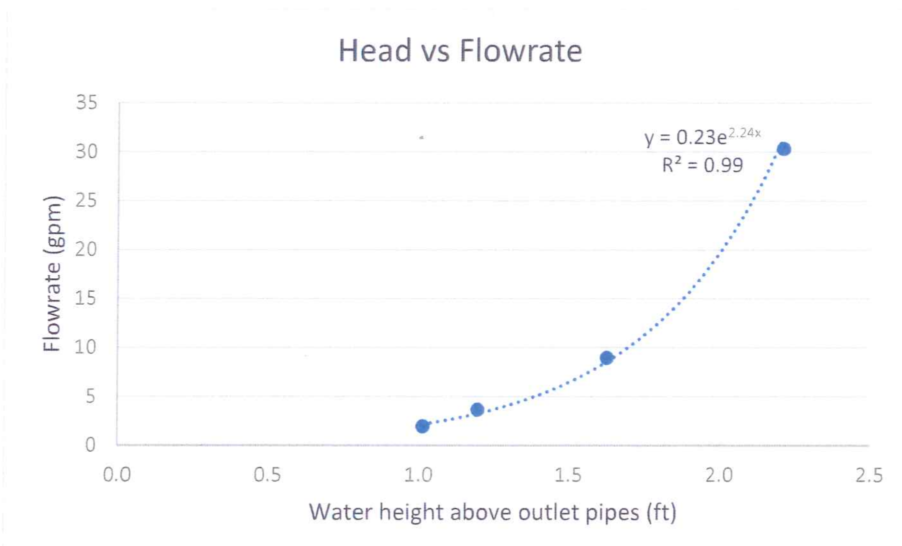

Figure 1

Based on a regression of the head and flowrate data, the head and flowrate have the following empirical relationship:

$$\boxed{\text{Flowrate in gpm} = 0.23e^{(2.24\, x\, \text{Water head in feet})}}$$

Equation 1

The theoretical maximum flow condition would be observed at the time where the water table within the Treatment Zone is approximately 2.5 feet above the outlet pipes. This is based on the distance of freeboard between the top of the outlet pipes and the spillover elevation of the buried weir. Using Equation 1, the theoretical maximum flow at 2.5 feet of head was estimated to be 60.5 gpm.

4.2 Bernoulli Equation

Using the conservation of energy principal, the Bernoulli equation (Equation 2) was applied at an outfall pipe to evaluate the theoretical discharge through the pipe given a maximum static head of 2.5 feet of water in the Treatment Zone.

$$E_1 = E_2 + h_{L,f} + h_{L,m}$$

Energy in the Treatment Zone = Energy at the end of the outfall pipe + friction losses from pipe + minor losses from fittings

Where:

$$E_1 = z_1 + p_1 + \frac{v_1^2}{2g}$$

$$E_2 = z_2 + p_2 + \frac{v_2^2}{2g}$$

Where:

z = *Elevation head*
p = *pressure*
v = *velocity*
g = *acceleration of gravity (32.2 ft/sec²)*

Equation 2

This flowrate calculated by using the Bernoulli equation corresponds to the flowrate at a given static head, and therefore the 2.5 maximum freeboard was used to evaluate a theoretical instantaneous upper limit flowrate that would decrease rapidly as the Treatment Zone drains. Losses from pipe friction and minor losses from fittings were also included in the Bernoulli equation calculation. The pipe friction losses, $h_{L,f}$, were estimated using the Darcy-Weisbach equation (Equation 3).

$$h_{L,f} = \frac{fLv_2^2}{2Dg}$$

Where:

f = *friction factor*
L = *length of discharge pipe (ft)*
v_2 = *water velocity (ft/sec)*
D = *outlet pipe diameter (ft)*
g = *acceleration of gravity (32.2 ft/sec²)*

Equation 3

Minor losses from fittings, $h_{L,m}$, were calculated using Equation 4.

$$h_{L,m} = \sum K \frac{v_2^2}{Dg}$$

Where:

K = *loss coefficients*
v_2 = *water velocity (ft/sec)*
D = *outlet pipe diameter (ft)*
g = *acceleration of gravity (32.2 ft/sec²)*

Equation 4

The equation used the following parameters:

- One Treatment Zone discharge valve was fully open;
- The discharge pipe diameter is 2" and the pipe material was Schedule 80 PVC;
- The length of the horizontal discharge pipe was 3 feet;
- The discharge pipe is level;
- The Treatment Zone and the end of the discharge pipe were open to the atmosphere ($p_1 = p_2 = 0$);
- The standing water within the Treatment Zone was stagnant ($v_1 = 0$); and
- Minor losses including pipe entrance, pipe exit, flow-through tee, and gate valve ($K_{total} = 2.1$).

Equations 2 through 4 were iteratively solved to obtain the velocity through the discharge pipe, v_2. This value was then plugged into Equation 5, as V, to calculate the discharge flowrate.

$$Q = VA$$

Where:

Q = *volumetric discharge flowrate* (ft^3/sec)
V = *discharge velocity* (ft/sec)
A = *cross-sectional pipe area* (ft^2) *of 2" pipe*

Equation 5

At 2.5 feet of static head within the Treatment Zone, the theoretical maximum discharge rate was calculated to be 0.13 cubic feet per second (cfs), or 59.0 gpm.

5. ESTIMATING LONG-TERM AVERAGE DAILY FLOW

5.1 <u>Recharge Observations Collected Immediately Following the Batch Release Test</u>

The groundwater treatment system contains two pressure transducers which transmit water level readings to the PLC interface, accessible by the site computer. After the outlet valve was closed following the November 2017 batch release test, the system was closely monitored during the period of recharge. Pressure transducer readings were recorded at least daily for five days during this time.

The recharge rate of the treatment system could be considered an estimate of the approximate long-term average discharge rate into the creek. If the system valves were to remain open, theoretically the discharge rate would eventually equilibrate with the recharge rate. (Water levels would likely be at some height below the top of the weir and not at the maximum condition).

Figure 2 shows the water level elevations recorded within the Treatment Zone after the valve was opened (indicating discharge) and after the valve was closed (indicating recharge).

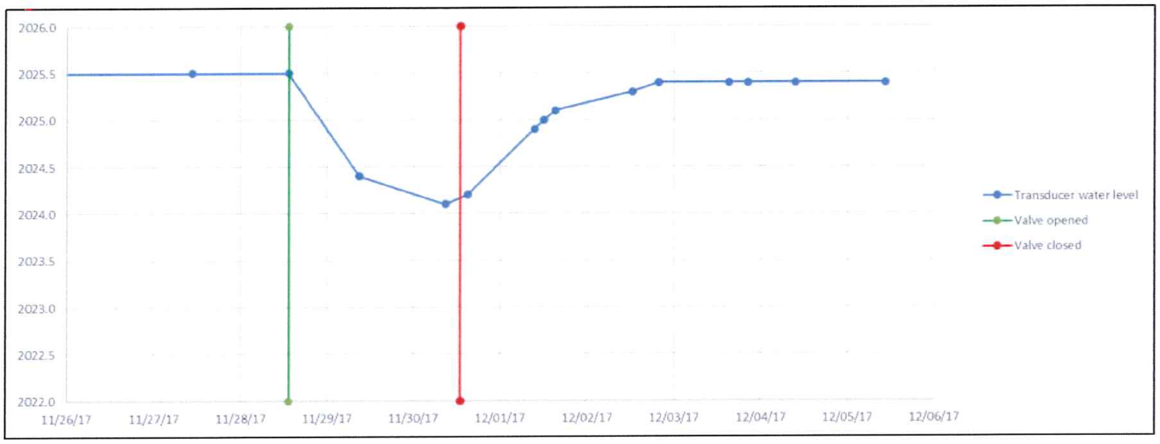

Figure 2

The rate of recharge was analyzed by calculating the change in water volume within the system per increment of time (Table 1). The volume was calculated using the surface area of the Treatment Zone and the porosity of the gravel.

	Time since valve closed (hrs)	GW elevation (ft amsl)	Water height above pipe (ft)	Available volume (ft3)	Available volume (gal)	Change in volume (gal)	Estimated flowrate (gpm)
11/30/2017 8:53	-4.1	2024.1	1.1	1704	12749	-	-
11/30/2017 14:55	2.0	2024.2	1.2	1859	13908	1159	3.2
12/1/2017 9:22	20.4	2024.9	1.9	2944	22021	8113	7.3
12/1/2017 12:05	23.1	2025.0	2.0	3099	23181	1159	7.1
12/1/2017 15:05	26.1	2025.1	2.1	3254	24340	1159	6.4
12/2/2017 12:17	47.3	2025.3	2.3	3564	26658	2318	1.8
12/2/2017 19:40	54.7	2025.4	2.4	3719	27817	1159	2.6
12/3/2017 15:05	74.1	2025.4	2.4	3719	27817	0	0.0
12/3/2017 20:26	79.5	2025.4	2.4	3719	27817	0	0.0
12/4/2017 9:35	92.6	2025.4	2.4	3719	27817	0	0.0
12/5/2017 10:25	117.5	2025.4	2.4	3719	27817	0	0.0
						Average	**5.18**

The recharge rate, and therefore the long-term daily average flowrate, was estimated to be 5.2 gpm.

5.2 Darcy Equation

The Darcy aquifer flow equation for groundwater flow was used to estimate the average amount of groundwater captured by the funnel and gate walls and flowing into the Treatment Zone (See Equation 6).

$$Q = KAi$$

Where:

Q	=	*groundwater flowrate (ft^3/min)*
K	=	*horizontal hydraulic conductivity (ft/min)*
A	=	*area normal to groundwater flow (ft^3)*
i	=	*groundwater gradient (ft/ft)*

Equation 6

Appendix B of the site EDR, dated 30 June 2006, included Darcy flow calculations which estimated the amount of groundwater that would be captured by the treatment system funnel and gate walls once they were constructed in 2007.

A cross-sectional area perpendicular to groundwater flow in the approximate location of the existing treatment system was evaluated. Using groundwater elevations from November 2004, the groundwater gradient was calculated to be 0.035 ft/ft. The calculations also assumed that the entrance to the funnel, or the length of the cross-sectional area, would be 750 feet. Based on these assumptions, the estimated flowrate in the EDR was calculated to be between 26-110 gpm.

The calculation from the EDR was repeated using 2017 data to estimate current average groundwater flows. Monitoring wells MW-8, MW-9, APC100/APC100B, PM-8, PM-5, PM-1, and PM-18 are located within the funnel and gate capture zone upgradient of the Treatment Zone (see Figure 3). Monthly water levels for these wells from the past two years were evaluated and plotted in Figure 4.

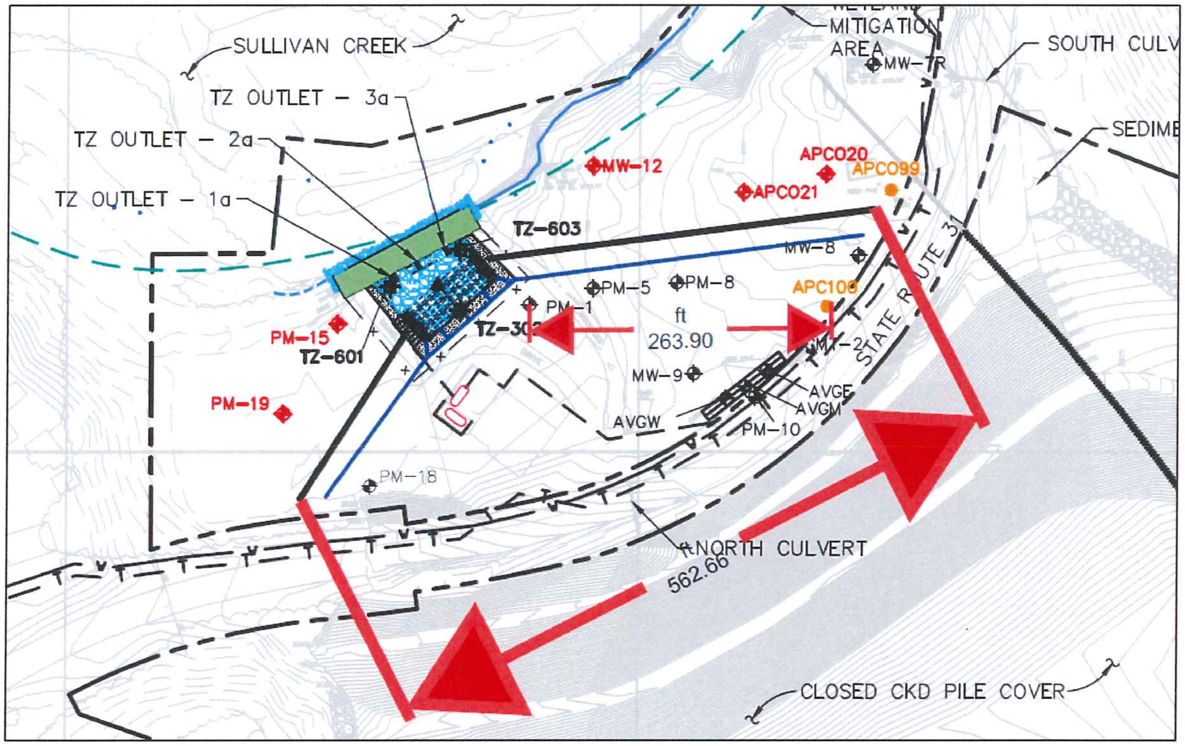

Figure 3

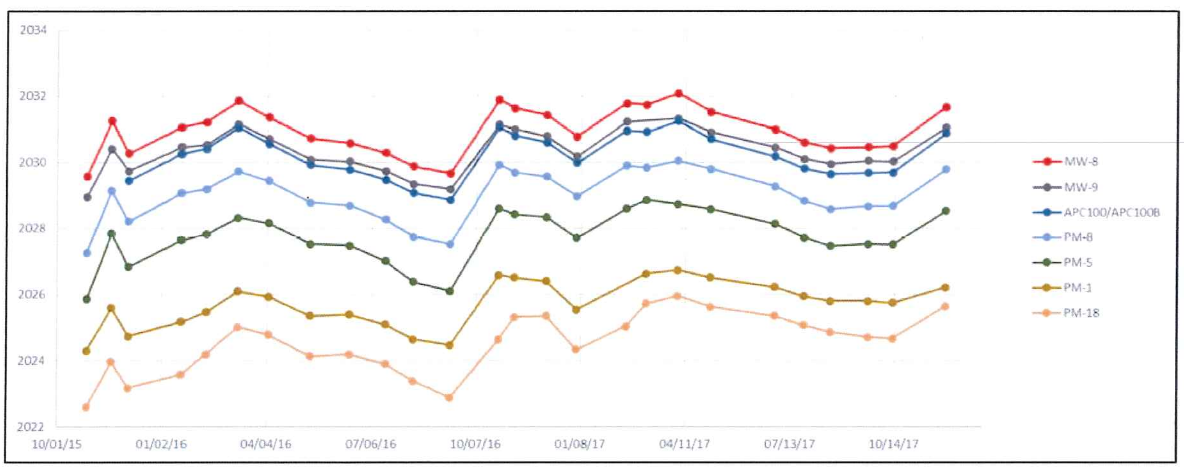

Figure 4

In the EDR the horizontal hydraulic conductivity, K, for the soil in the area around Monitoring Well MW-8 is assumed to be 1.68 x 10^{-2} ft/min, which corresponds to a fine sand. The hydraulic conductivity value for MW-8 was chosen for the updated calculations because this well is within the existing funnel.

The updated groundwater gradient, i, was calculated by using the average difference in water levels at wells APC100/APC100B and PM-1 (4.4 ft). This value was then divided by the horizontal distance between the wells (264 ft). The updated gradient is therefore 0.0167 ft/ft.

The updated cross-sectional area of groundwater flow, A, was calculated by multiplying the average saturated depth (9 ft) within the funnel by the actual length of the constructed funnel mouth shown in Figure 4 (563 ft). The updated cross-sectional area is therefore 5,064 ft^2.

Using the Darcy Equation, the approximate long-term average groundwater flowrate was calculated to be 1.42 ft^3/min, or 10.6 gpm.

6. SUMMARY

Table 2 below summarizes maximum and long-term average daily flows estimated using the four methods discussed in this memo.

Table 2		
MAXIMUM DAILY FLOW		
	(gpm)	(gpd)
Batch release test	60.5	87,107
Bernoulli equation	59.0	84,998
	Average	86,052
	Reported value (rounded)	**86,000**
LONG-TERM AVERAGE DAILY FLOW		
	(gpm)	(gpd)
Batch test recharge	5.2	7,460
Darcy flow equation	10.6	15,266
	Average	11,363
	Reported value (rounded)	**11,400**

Averaging the results obtained from various methods, the reported maximum daily flow value was 86,000 gpd, and the reported long-term average daily flow value was 11,400 gpd. These results seem reasonable based on-site observations and are proposed for use in the 2018 NPDES permit application.

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