

TECHNOLOGY AND DESIGN ADVANCES IN PASSIVE TREATMENT SYSTEM FLUSHING

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Abstract: A common goal of passive mine drainage treatment is the removal of metals such as iron and aluminum from contaminated water. However, these metals form solid particles that can clog pipes and limestone aggregate, increasing operational costs and decreasing treatment system longevity. To combat this problem, a wide variety of flushing systems have been installed in passive treatment systems. Flushing systems usually consist of a network of perforated pipes buried in limestone, which drain via valved header pipes. Periodically, the valves are opened to allow large amounts of water to flush through the system and, ideally, remove accumulated solids. This theoretically extends the useful life of passive systems by restoring porosity. Unfortunately, flushing system design is poorly understood and most systems are not designed using scientific or engineering principles. Four existing systems were examined from an engineering standpoint and an engineering method for designing these systems was developed. The purpose of this paper is to provide a survey of flushing technologies currently being used, detail one method of using engineering principles to design flushing networks, and discuss the implications of this analysis on future flushing technologies.

Additional Key Words: mine drainage treatment

¹Paper was presented at the 2004 National Meeting of the American Society of Mining and Reclamation and The 25th West Virginia Surface Mine Drainage Task Force, April 18 – 24, 2004. Published by ASMR, 3134 Montavesta Road, Lexington, KY 40502.

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Introduction

Vertical flow pond (VFP) technology emerged in 1994 as a way to treat net-acidic mine drainage contaminated with both iron and aluminum (Kepler and McCleary, 1994). These systems, also called Successive Alkalinity Producing systems (SAPS), generally contain standing water that flows downward through an organic substrate layer followed by a limestone layer. Perforated pipes placed in the limestone collect the water and discharge it to subsequent treatment cells.

The purpose of the organic layer is to consume oxygen and create a reducing environment, converting any ferric iron (Fe^{3+}) to ferrous iron (Fe^{2+}) and allowing it to pass through the limestone in dissolved form. However, due to limestone dissolution and increasing pH, aluminum (Al^{3+}) still reacts to form aluminum hydroxide solids in the treatment system. Over time, these solids can accumulate in the limestone and decrease the permeability of the treatment system, causing hydraulic failure (Rose et al., 2004).

To combat this problem, the design of most VFPs includes the ability to flush large quantities of water through the system. The flushing systems that have been installed range from the inclusion of a valve on the existing underdrain system to a completely separate and multiple-level flushing system equipped with several flush zones and valves. Figure 1 shows a schematic drawing of separate underdrain and flush plumbing in a VFP. However, VFP flushing systems generally have been installed without a robust engineering analysis or design method. In addition, there is little funding available to perform field studies of installed systems in order to evaluate flushing effectiveness.

In 2002, a Flushing Workshop was held at the Ebensburg District Mining Office of the Pennsylvania Department of Environmental Protection. The purpose of this conference was to discuss the effectiveness of various designs. As a result, a design method for flushing system was developed (Lagnese, 2002).

The purpose of this paper is to discuss this design method and how it is being used in current flush system design and to discuss the future of flushing technology and areas where more research is needed.

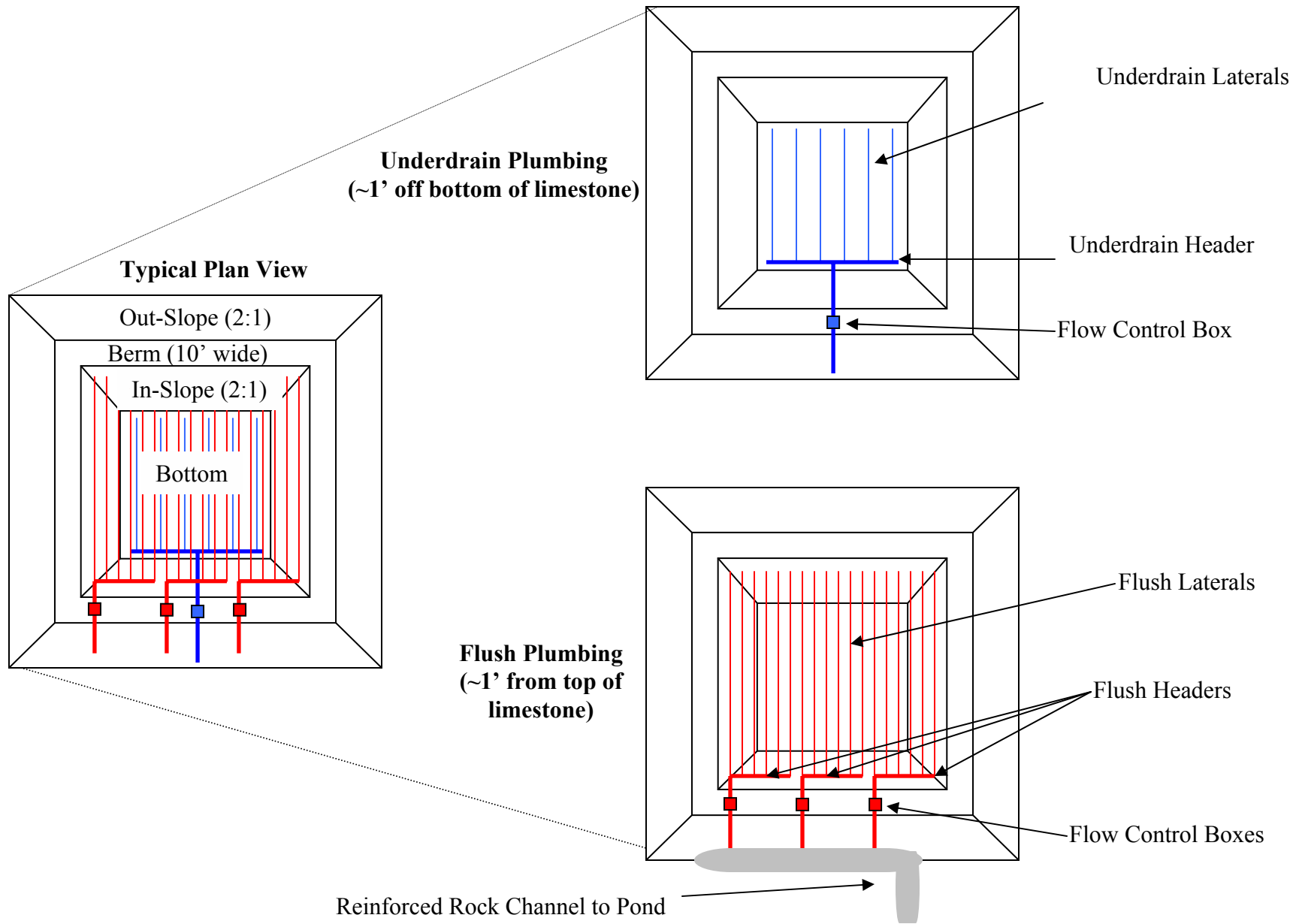


Figure 1: VFP Schematic Plan View

Problems Associated With Vertical Flow Ponds

In a recent study of several VFP treatment systems, several potential problems that could result in system failure were identified (Rose, 2002). Altering the design and/or placement of the flush plumbing could lessen some of these problems. A few of the systems had failed or had experienced short-circuiting problems related to the compost layer. In some cases, flush events had caused preferential flow paths to develop in the compost (Rose, 2002). In these cases, troughs or vortexes in the compost occurred over flush laterals. In other cases, VFPs experienced hydraulic failure caused by the accumulation of aluminum solids near the top of the limestone (Rose, 2002).

Gravity Flow Design Method

During flushing, it is the velocity of the water moving past the limestone and precipitated particles that creates shear forces. As velocity increases, the magnitude of the shear force also increases. The magnitude of the shear force required to move a particle depends upon the characteristics of the particle, including size, chemical or physical attraction to the limestone aggregate, cohesiveness and other factors. The nature of the aluminum particles precipitated in VFPs, and thus the required shear force to move these particles, is unknown. It is likely that these characteristics vary based on the influent chemistry and may change as the particles age. Because of these uncertainties, maximizing the overall flushing velocity in order to move as many particles as possible is desirable.

The superficial flushing velocity, “ V_s ”, will be used throughout this paper.

$$V_s = Q_f/A_f \quad (1)$$

As shown in Equation 1, the superficial velocity is equal to the flush flow rate, Q_f , over the entire surface area that is being flushed, A_f . For a VFP with only one flush zone, V_s is the rate that the water level drops during flushing events.

There are two ways to increase the superficial velocity; by decreasing the area being flushed (A_f) and by increasing the flush flow rate (Q_f). Decreasing the flush area (A_f) can be achieved by dividing the flushing underdrain system into several separate sub-systems, a recommended design approach. Each sub-system should include its own header pipe and flush valve, and the sub-systems should be flushed one at a time. Flushing each sub-system separately allows for a greater retention time in the subsequent settling pond and prevents the VFP from draining too much of the standing water at once (which decreases the head available for flushing). Increasing the flush flow rate (Q_f) requires that the underdrain piping system be designed with the capacity to handle the desired flow. In a typical flush plumbing network, perforated lateral pipes lead to a common flush header, which discharges through a valve or flow control box (See Figure 1). Therefore, Q_f can be affected by changing the size and spacing of the perforations, lateral pipes, and/or header pipes.

The recommended approach involves designing the flush piping to allow for gravity, not pressurized flow. In the gravity flow case, the lateral and header pipes will be capable of transferring all of the flow from each contributing perforation. The pipes will not be flowing completely full and will draining only due to gravity. The flow rate will be limited by the size and number of perforations and the pipe network will not be pressurized. In pressurized flow, the pipes would be flowing completely full and water would be flowing due to the difference in elevation between the surface of the VFP and the discharge.

Equation 2 shows the flow through each perforation, modeled as a sharp-edged orifice, where Q_{orf} is the flow rate in cubic meters per second (m^3/s), d_{orf} is the diameter of each orifice in meters, and h is the available head across the orifice in meters (Cameron Hydraulic Data, 1994).

$$Q_{orf} = 2.12 * d_{orf}^2 * h^{0.5} \quad a \quad (2)$$

. When the pipe network is not pressurized, the head term is assumed to be the same for all perforations, equivalent to the difference in elevation between the surface of the pond and the perforation in the flush piping. This results in an equalized flow distribution and provides for

^a Derived from the orifice flow equation $Q_{orf} = C * A_{orf} * \{2gh\}^{0.5}$ where:

C = flow coefficient = 0.61 for sharp-edged orifice
g = acceleration constant = 9.81 m/s²
 A_{orf} = area of the orifice (m^2) = $\Pi * d_{orf}^2 / 4$
H = head across the orifice (m)

maximum flow through each perforation. The number of perforations (also referred to as orifices) required for a given flush zone area (A_f) can be determined by dividing the desired flush rate (Q_f) by the average flow capacity of each orifice (Q_{orf}). The average flow capacity (Q_{orf}) can be approximated using the average head (h) available during the flush. For example, if the flushing system is designed to reduce the water level in the pond from 1.2 m above the orifice to 0.8 m, then an average head of 1 m should be used to determine the number of orifices required.

If the lateral pipes or the header pipes limit the maximum flow rate, the result will be a pressurized flow system. The orifices will be capable of supplying the network with more water than it is able to pass. A pressurized flow network will result in an unequal distribution of flow throughout different areas in the network because the head differential on each orifice will vary with distance along each lateral. This will result in preferential flow. The pressure created in the pipe network will reduce the amount of flow that is passing through each orifice.

Designing the plumbing network begins with establishing the superficial velocity that is desired during flushing. Variables that can then be altered include the number of flush zones (and thus the area being flushed) and the size and spacing of the orifices, the laterals, and the headers. The orifices and the laterals should have the same spacing to ensure a uniform distribution and reduce preferential flow paths. Once this spacing is established, the number of orifices per lateral is known and the required capacity of each lateral can be calculated by multiplying Equation 2 by the number of orifices per lateral. The required size of the laterals is determined using the Manning Equation (Equation 3, Corbitt 1990).

$$Q_h/A_h = V = (1/n) * R^{2/3} * S^{1/2} \quad (3)$$

Where:

Q_h = flow rate in the header, m^3/sec

A_h = Cross sectional area of flow, m^2

V = velocity, m/sec

n = roughness coefficient (dependent upon pipe material, typical value 0.013)

R = hydraulic radius, m (equivalent to $1/4$ pipe diameter for pipe flowing full)

S = slope, m/m

The required header capacity is calculated by multiplying Equation 2 by the number of orifices per lateral and the number of laterals per header. Equation 3 can then be used to determine the required size of the header pipe. The design of the VFP underdrain header pipe should be based on achieving a minimum velocity of 0.6 to 0.9 m/sec (2 to 3 feet/sec) at the desired flushing rate. This will ensure that particles removed from the bed during the flush will not settle out in the pipe (Corbitt, 1990).

Construction and Cost Implications

Figure 1 shows a schematic drawing of a VFP plumbing network. Many early VFPs were constructed with only the underdrain layer of plumbing. Adding the flush layer of plumbing has implications to both the construction methods and the overall costs of the VFP.

The Johnson Run treatment system located in Elk County, Pennsylvania was constructed in 2002. The system was designed to treat up to 13 L/sec (200 gpm) of mine drainage with approximately 10 mg/L each of iron and aluminum. The average flow rate is 6 L/sec (95 gpm). The system includes the first flushing system designed using the method described in Lagnese (2002) and includes eight flushing zones with laterals placed one foot from the top of the limestone bed.

Construction bidders for the Johnson Run system provided bids for the system both with and without the flush level of plumbing. The VFP was approximately 1,860 m² (20,000 feet²) total and contained approximately 3.6 million kg (4,000 tons) of AASHTO #1 limestone. The underdrain consisted of nearly 425 m (1,400 feet) of underdrain lateral and header pipes placed at the bottom of the limestone bed. The flush system contained nearly 1,525 m (5,000 feet) of flush lateral and header pipes and also required 8 flow control boxes (one for each flush zone). The flush design was for 8 independent flush zones. The low bidder's costs for the system with and without the flushing network differed by an average of approximately \$40,000.

This increase in construction cost and material quantities can be justified if the flushing system increases the treatment capacity (in both flow rate and total life span) of the system as compared to systems that do not contain flush plumbing.

Figure 2 shows the Johnson Run VFP during construction. Flush laterals were spaced at 1.2-m (4-foot) intervals, which caused some difficulties during construction. Pipe T's were used to connect the laterals and the headers. Connecting the plumbing in this manner was difficult

because the length of the Ts was larger than the spacing of the laterals, requiring extensive pipe joining and the use of additional fittings to obtain the required spacing.

To alleviate this problem, pipe saddles can be used to connect header pipes with pipe laterals. Figure 3 shows this method of connection. These saddles are installed directly to the header pipe and do not require that the header pipes be pieced together at every lateral. Saddles can be installed at any location along the header pipe except where two joints of pipe meet. To maintain the velocity assumptions of the flushing system, the holes cut into the header pipe over which the saddles are installed must be the same diameter of the lateral pipes.



Figure 2: Johnson Run VFP During Installation



Figure 3: Pipe Saddle Connection Between Lateral Pipe and Header Pipe

Another design consideration that greatly impacts construction is the placement of the flush and underdrain plumbing in limestone. If heavy equipment passes over pipes that are not buried several feet in limestone, the pipes can be crushed. Since the Johnson Run design called for flush pipes one foot from the top of the limestone, equipment could not pass over the surface of the VFP once the flush plumbing has been placed. The construction contractor therefore chose to place all the limestone first and then excavate trenches for each lateral. An excavator was then used to place and distribute the compost (See Figure 3). The equipment retreated in this manner across the VFP until all materials had been installed. If it becomes necessary to enter the VFP with equipment to perform maintenance or to remove or add more compost, low ground-pressure equipment is recommended in order to protect the plumbing.

Evaluating Existing Flushing Systems

DeSale II and Tangascootack I Treatment Systems

The ultimate test of any flushing system is the amount of solids that are removed from the media during a flushing event. Watzlaf et al. (2002) performed the most extensive evaluation of this kind on the DeSale II vertical flow system in Butler County, PA. At the time of the flushing event, the system was operating normally and pore spaces in the limestone were estimated to be only 0.75% filled. The system has two levels of flush plumbing placed 0.6 m (2 feet) from the surface of the limestone and 1.2 m (4 feet) from the surface of the limestone. Both short-duration (under 10 minutes) and long-duration (over 4 hours) flush events were performed. The effluent flow rate and metals concentrations were monitored throughout the flushing events. The total mass of solids removed from the systems varied from between 0.2% and 1.3% of the solids that were estimated to be deposited in the systems based on influent and effluent data taken during normal operation throughout the life of the system. The long-term flushing event removed a higher percentage of solids than the short-term flushing event (Watzlaf et al., 2002).

Rose et al. (2004) conducted a similar analysis during the flushing of the Tangascootack I vertical flow system. This system was constructed in 1998 by the USDA Natural Resources Conservation Service. Water was passing through the system very slowly due to plugging of the effluent pipe and the inability of the system to develop a head difference from the influent water surface to the effluent pipe. The limestone thickness and the depth of the flush pipes in the limestone were not consistent over the extent of the VFP. Flush pipes were located approximately 12 to 35 cm (5 to 14 inches) from the top of the limestone. The system had been flushed 69 days prior to this flushing event. The flushing event lasted approximately 20 minutes. During this time, less than 1% of the total aluminum estimated to be deposited in the system since the previous flushing event was removed. A subsequent excavation of the system revealed a white precipitate covering the aggregate in the top 0.3 m (1 foot) of the limestone bed.

The treatment systems evaluated by Rose et al. (2004) and Watzlaf et al. (2002) were not designed using the gravity flow design methods, but the average superficial flush velocity can be calculated by dividing the flush flow rate by the area being flushed. In the case of the DeSale II system, the average superficial flush velocity is approximately 3.4×10^{-5} m/sec (1.1×10^{-4}

ft/sec). In the case of the Tangascootack I system, the average V_s is approximately 4.6×10^{-5} m/sec (1.5×10^{-4} ft/sec).

In both cases, it is not known why so little of the calculated precipitate was removed by flushing. For both systems, the superficial velocity is quite low. In the case of the DeSale II system, the flush pipes are 0.6 m (2 feet) into the depth of the limestone. This depth may be too great to allow particles precipitated near the top of the limestone to travel through the tortuous paths of the limestone pores and reach the pipes, particularly during short-duration flushing events.

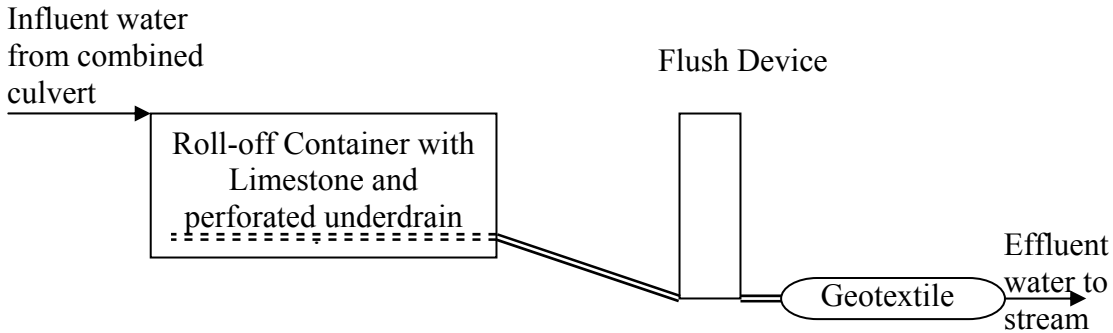
Johnson Run Treatment System

For the Johnson Run system previously described, the calculated average superficial flush velocity of the system is 2.8×10^{-4} m/sec (9.3×10^{-4} ft/sec; approximately six times greater than the DeSale II and Tangascootack I VFPS). A flushing study has not yet been performed on this system, but flushing events have taken place that did not involve flush water sampling. This will complicate any future analysis of flushing performance, but such studies should take place in order to gain more data on the effectiveness of this flushing system in removing precipitated particles.

Jonathan Run Pilot Systems

The Jonathan Run self-flushing system is a pilot-scale limestone system that uses self-flushing siphons to empty the limestone cells each time they fill. Figure 4 shows a schematic drawing of the system and Figure 5 shows a photograph of the system. Because this system flushes each time it is filled, the mass balance of solids in the system can be evaluated more easily than typical manual-flush VFPS. Each pilot system consists of a 23 m^3 (30 cubic yard) roll-off container filled with approximately 29,900 kg (33 tons) of limestone aggregate. Water is introduced into the top of the containers and a single 38-cm (15-inch) diameter custom-perforated pipe on the bottom of each container is used for flushing. The containers both discharge to a geotextile bag intended to collect the flushed aluminum solids. One container is filled with AASHTO #3 limestone and one is filled with AASHTO #1 limestone. Two sizes of

limestone were used in order to evaluate the performance of each size. Smaller limestone (AASHTO #3) is thought to generate more alkalinity because more surface areas is in contact with the water. Conversely, larger limestone (AASHTO #1) is thought to be easier to flush



because of the larger pore spaces.

Figure 4: Schematic of Jonathan Run System



Figure 5: Jonathan Run Pilot Scale Treatment Systems

The systems were installed so that the influent flow rate, and thus the flush cycle time, could be controlled. The average retention time of the systems is half of the flush cycle, since some of the water is retained for the entire time, but the last water that enters the container is almost immediately flushed out. Table 1 shows sampling results for both containers at Jonathan Run.

The average influent chemistry, which was fairly consistent, is shown. The influent contained less than 1 mg/L of iron.

Table 1 shows the results of sampling of these discharges. The results are listed in the order of increasing influent flow rate (decreasing retention time).

Table 1: Jonathan Run Self-Flusher Influent and Flush Water Chemistry Sampling Results.

Sample Description	Flow, L/sec (GPM)	Lab pH	Alk (mg/L)	Net Acid (mg/L)	Total Al (mg/L)	Dis Al (mg/L)	% Tot Al Flushed	% Flush as Particulate
Average Influent	1.58 (25)	3.5	0	304	48.8	45.2		
Box 1 Flush	0.063 (1)	5.8	23	-8	67.3	0.7	138%	99%
Box 1 Flush	0.11 (1.75)	5.7	20	21	34.1	0.4	70%	99%
Box 1 Flush	0.126 (2)	4.9	4	59	40.5	2.4	83%	95%
Box 1 Flush	0.126 (2)	5.9	20	24	56.5	0.0	116%	100%
Box 1 Flush	0.126 (2)	4.8	2	42	24.4	5.6	50%	88%
Box 1 Flush	0.189 (3)	4.5	0	99	25.8	18.1	53%	63%
Box 1 Flush	0.189 (3)	4.5	0	99	25.8	18.1	53%	63%
Box 1 Average	0.132 (2.1)	5.1	10	48	39.2	6.5	80%	87%
Box 3 Flush	0.063 (1)	4.7	3	64	23.6	0.4	48%	99%
Box 3 Flush	0.079 (1.25)	6.1	38	-12	24.4	0.2	50%	100%
Box 3 Flush	0.095 (1.5)	5.8	20	3	22.3	0.7	46%	99%
Box 3 Flush	0.11 (1.75)	6.2	21	-8	26.4	0.5	54%	99%
Box 3 Flush	0.126 (2)	6.2	38	-7	27.8	0.0	57%	100%
Box 3 Flush	0.126 (2)	6.2	21	-6	21.7	0.0	44%	100%
Box 3 Average	0.101 (1.6)	5.9	23	6	24.4	0.3	50%	99%

“Box 1 Flush” contained AASHTO #1 limestone aggregate. “Box 3 Flush” contained AASHTO #3 aggregate. Samples were taken from the system discharge pipe as a composite of the entire flush cycle and acidified in the field. The influent flow rate was measured using the timed volume method. Alkalinity was measured in the field using a Hach Digital Titrater. All other analyses were performed by G&C Laboratories of Brookville, PA using standard methods.

Samples for particulate metals were filtered in the field using a 1.0-micron glass fiber syringe filter. “% Tot Al Flushed” was calculated as the Total Al in the flush water over the average influent Al concentration. The “% Flush as Particulate” was calculated as (average influent Al concentration - the dissolved Al concentration) / average influent Al concentration.

In the system containing AASHTO #1 limestone flushing removed an average of 80% of the total influent aluminum over the range of influent flow rates tested. Flushing efficiency seemed to decrease with decreased retention time (increasing influent flow rate). Flushing of the system containing AASHTO #3 limestone removed an average of 50% of the influent aluminum. These data suggest that the use of larger limestone provides for more efficient flushing. Dissolved aluminum concentrations were generally quite low, indicating that the aluminum being flushed from the systems was in particulate form. The percentages of influent aluminum removed by the Jonathan Run systems, 50 – 80%, indicate a vast improvement over the results measured at the DeSale II and Tangascootack I systems, where approximately 1% of the total influent aluminum was removed by flushing.

There are two possible reasons for the superior flushing performance of the Jonathan Run self-flushing systems. The average superficial velocity that is experienced in the limestone is approximately 1.2×10^{-3} m/sec (4.0×10^{-3} ft/sec) based on field measurements of flush time and the known capacity of the limestone cells. This value is four times greater than the superficial velocity of the Johnson Run VFP and 25 times greater than the superficial velocity of the DeSale II and Tangascootack I VFPs.

Another factor that may be making the aluminum particles easier to flush is the frequency of the flush cycles. In a traditional VFP, flushing is performed on a frequency of no greater than monthly. In some cases, flushing is only performed once a year or when water levels indicate that permeability has decreased. Therefore, the water that passes through the system during a flushing event is only a small fraction of the water that has been treated. In the case of the Jonathan Run self-flushers, all treated water is released during each flushing event.

In traditional VFPs, aluminum particles that are formed near the top of the limestone are retained in that location until the next flush. It is likely that the nature of these solids changes over time. The particles may become attached to limestone surfaces or lose waters of hydration,

becoming more dense and thus harder to remove during a flushing event. These effects are minimized at Jonathan Run due to the frequency of the flushing events.

It is unknown how long the Jonathan Run systems will continue to flush with the high level of effectiveness that was documented early during their operation. A significant portion of the aluminum is still being retained within the limestone. Eventually, these aluminum particles may cause the system to fail due to decreased permeability, decreased retention time, or decreased alkalinity generation due to limestone armoring.

The Future of Flushing

Design Recommendations

Vigorous flushing in the zone of precipitation would help to remove precipitated particles. Based on observations from failed VFPs, the flushing pipes should be placed approximately 0.3 m (1 foot) deep into the limestone in order to protect the compost from disturbance and to reduce the distance the accumulated aluminum particles must travel to reach the flush pipes.

Placing flush pipes in the limestone rather than on the surface of the limestone and/or evenly spacing the pipe perforations could help to alleviate problems with the compost layer such as vortices and troughs. Standard perforated pipes generally have three orifices spaced around the diameter of the pipe and every 15 cm (6 inches) along the pipe. The orifice spacing is much less than the lateral spacing, creating a zone of higher flow rates over each lateral. Therefore, the orifice spacing and the lateral spacing should be the same in order to create a network of evenly spaced orifices over the entire flushing area.

Evaluating Flushing Effectiveness

In order to continue to improve flushing design and implementation, monitoring existing systems is extremely important. Periodic sampling of the influent and effluent for flow rate and metals, acidity, alkalinity, and sulfate concentrations are vital for assessing the performance of the systems and for estimating the quantity of metal precipitates that are being retained in the bed. Monitoring the water level in the VFP allows permeability problems to be identified early

and provides an indication of the effectiveness of flushing events in restoring porosity. Collecting samples of flush water during flushing events is necessary in order to evaluate how effective the flushing process is in removing the accumulated metal particles.

Monitoring these parameters would be useful for any type of flushing system. Consistent data collection and analysis would allow a direct comparison of widely different systems. Monitoring over the life of systems could result in a better understanding of the total expected life of these types of systems, which is currently poorly understood. Monitoring and continued system evaluation may also result in a scientifically-based method for recommending flushing frequency and duration.

Areas of Research

In addition to the monitoring of existing systems, there are many aspects of passive treatment flushing that would benefit from additional laboratory and field-scale experiments. Studies should investigate:

- The characteristics of the precipitated solids (particle size and density) and how the characteristics change over time;
- Where the precipitation occurs within the treatment media;
- The magnitude of velocities and shear forces needed to flush precipitated solids from a limestone bed, operated in either an upflow or downflow mode;
- The percentage of precipitated metals removed during flushing; and
- The feasibility of using chemical treatment to re-solubilize the metals prior to flushing (Lagnese, 2002).

The test results are likely to vary depending on the quality of the untreated acid mine drainage water.

Acknowledgements

Funding for the original study of flushing technologies (Lagnese, 2002) was provided by the Pennsylvania Department of Environmental Protection Growing Greener Grant Program through

a grant to the Babb Creek Watershed Association. Funding for the construction of the Johnson Run system and the Jonathan Run pilot scale systems was also funded by the Growing Greener Grant Program.

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