THE HIGH PERFORMANCE BIOFILTRATION CONCEPT: THE "WORKHORSE" TECHNOLOGY OF DISTRIBUTED TREATMENT SYSTEMS

David Venhuizen[1]

Abstract: The high performance biofiltration concept is a variant of recirculating sand filter technology, itself one of the most robust and versatile wastewater treatment technologies, greatly favoring it for small-scale treatment systems. This concept was developed in part by optimizing the long-understood principles of that underlying technology and in part by observing operation of systems in the field. The resulting process practically maximizes the efficiency of the biofiltration process, and it minimizes or renders more expeditious the maintenance of the system. The high performance biofiltration concept is an even more highly robust process, resistant to upsets due to shock loads and variable operating conditions without needing frequent adjustments. It approaches the ideal for small-scale treatment processes of a drop-in-the-hole-and-walk-away type of process, requiring only monitoring of alarms (and appropriate attention when one goes off, of course) and relatively minimal routine maintenance activities.

These characteristics make this concept an ideal candidate to be the "workhorse" technology of distributed treatment systems, in which total treatment capacity required by an overall system of any size would be provided in a number of distributed, smaller-scale treatment units. With multiple treatment centers to police and maintain, that concept demands that the distributed treatment units incur low O&M liabilities in order for this strategy to be cost efficient and practical. This paper reviews how the high performance biofiltration concept fills that need.

A BRIEF HISTORY OF THE CONCEPT

The high performance biofiltration concept is the evolutionary end product of intermittent sand filter technology. The concept of intermittent sand filtration derived from observations made in the mid-1800's at "sewage farms" in sandy soils. The drainage from these areas, onto which wastewater was intermittently dosed, was greatly purified. This led to using sand beds especially constructed for wastewater treatment. (Dymond, 1981) In those operations, the sewage was "flood-dosed" onto the fields. Thus it is not surprising that the initial design concept for the sand filter treatment process used fairly fine sand and entailed flooding the filter with the entire daily loading all at one time. This allowed a full day for the wastewater to infiltrate and pass through the filter bed.

In one of the earliest recorded efforts to systematically study this process, these operating parameters were fairly well standardized, as was filter bed depth, at four feet. The major process innovation of this era was a "formal" underdrain system that promoted complete drainage of the filter bed. The hydraulic loading rate (HLR) onto filter beds designed and operated in this manner were typically limited to about 1-2 gal/ft^2/day (4-8 cm/day) to avoid frequent clogging of the bed. The slow infiltration rate allowed by the fine sand bed resulted in clogging being largely a surface phenomenon, so periodic raking or tilling of the bed to break up the "cake" that formed on the surface, along with occasional removal (scraping) of the surface layer, became the standard maintenance protocol. (Clark and Gage, 1909)

[1] Principal, David Venhuizen, P.E., 5803 Gateshead Drive, Austin, Texas 78745, waterguy@ix.netcom.com

The “modern” intermittent sand filter treatment process has its genesis at the University of Florida, where studies of this process were conducted in the 1940’s and 50’s. In these studies, it was found that by splitting the daily loading into two doses, treatment effectiveness—measured in BOD_5, TSS and nitrification—was enhanced, and the filter bed was able to accommodate a higher loading rate without leading to more rapid clogging of the filter. This led to further studies of more frequent dosing, up to one dose per hour, in which it was found that significantly higher loading rates could be accommodated when loading the filter bed multiple times per day. (Grantham, et al., 1949, Furman, et al., 1955)

Another major observation was the effect of media size on allowable loading rate, and the interaction of this parameter with loading frequency. For filters employing larger media, treatment performance was observed to improve with more frequent loadings, even when higher HLR’s were applied. One example was a filter containing media with an effective size of 1.04 mm—somewhat larger than the 0.2 to 0.4 mm effective size typically employed in sand filters to that point—which demonstrated a 96% BOD_5 removal efficiency when loaded once per hour at an HLR of 13.8 gal/ft^2/day (55.2 cm/day), a huge increase over the rates typically employed to that point. This contrasted with a removal efficiency of only 70-80% at HLR’s ranging from 4.02 to 9.76 gal/ft^2/day (16.1-39.0 cm/day) when the same filter bed was only loaded twice per day. (Furman, et al., 1955)

The next major process innovation was the recirculation concept. Introduced by Hines and Favreau in 1975, recirculation was originally used to control odors, by diluting septic tank effluent before applying it to open-bed sand filters. (Hines and Favreau, 1975, Teske, 1979) It was immediately observed that recirculation also improved treatment efficiency, allowing the filter bed to be loaded more heavily while still obtaining high quality effluent. This is due in part to the more uniform loading schedule that recirculation can impart. A major factor, however, is dilution of the filter influent strength imparted by mixing recirculation flow with forward flow. For example, in observations reported by Swanson and Dix (1987) and by Venhuizen, et al. (1998), average organic loading rate was high, but with recirculation ratios ranging up to 7:1, the organic *strength* of the *applied* influent was greatly reduced. The benefits of this, in particular with regard to enhancing nitrification through a filter bed, are detailed by Piluk (1988).

Nitrogen reduction was the next frontier for this technology. Guided largely by conceptual investigations by Roland Mote at the University of Tennessee, Rich Piluk at the University of Maryland, and Dee Mitchell at the University of Arkansas, the ability of this process to cleverly manipulate the nitrogen cycle was harnessed to eliminate a considerable portion of the nitrogen in the wastewater. In addition to several research efforts (e.g., Sandy, 1987, Piluk, 1988) this concept was subjected to an extensive field study of 5 systems (4 homes and a grocery store) that operated year-round on Washington Island, Wisconsin, over a two-year period (1992-94) of once- or twice-weekly monitoring. This effort confirmed that removal rates for total nitrogen in the range of 60-90% could routinely be attained, provided proper conditions were maintained. Observations also determined that the vast majority of nitrate in recirculated effluent was denitrified in the septic tank when recirculation flow was routed through it, indicating that the attached-growth filters conceived by Mitchell (1991) and Piluk (1988) to serve as the denitrifying chambers would be superfluous for that purpose, given sufficient hydraulic retention time (HRT) in the septic tank. (Venhuizen, 1994) This observation was confirmed in concurrent work by Piluk and Peters (1994) in Anne Arundel County, Maryland.

The Washington Island project also confirmed the observations of the Florida studies (and several other efforts—e.g., Swanson & Dix, 1987, Sandy, 1987, Piluk, 1988) that treatment efficiency can be maintained when employing coarser media if loading is frequent and uniform over the diurnal cycle. Three of these systems employed a very coarse gravel media in the size range of 1/4”-3/8” (6-9.5 mm) yet still provided superior performance. Dosing frequencies of about 30 minutes were employed in these systems. (Venhuizen, 1994)

The benefit of uniform loading of the entire filter bed surface in coarse media filters, using a spray distribution system, was also demonstrated. While fine media filters present sufficient resistance to infiltration that a rapidly loaded dose would spread over the bed surface even if loaded at a single point, coarse media filters will cause no such spread from the point of application, at least not until significant clogging had developed at that point. As was accidentally discovered in the Washington Island project, when that occurs, then adjacent areas would receive all the flow until they too clogged, and so on, until this “progressive failure” resulted in the whole bed becoming clogged. (Venhuizen, 1994) The partial aeration of the wastewater imparted by spray distribution is also beneficial to the treatment process, as detailed by Piluk (1988).

The final innovation, yielding the high performance biofiltration concept, was a refinement in the recirculation process. Recognizing that gravity recirculation schemes employed at that time represented a major compromise of the uniform dosing regime demanded by coarse media filters receiving high HLR’s, the Washington Island project employed pumped recirculation systems, to assure there would be no long “gaps” in recirculation flow, and thus in dosing intervals. This, however, proved to impart its own hazards when one of the recirculation pumps failed—and was not noticed, thus not repaired, for over a month—and critically compromised one of the systems. This hazard was circumvented by using a gravity recirculation scheme in conjunction with an “effluent bypass valve”, as detailed in the following section. (Venhuizen, et al., 1998)

DESCRIPTION OF THE PROCESS

A general representation of the high performance biofiltration treatment process which has resulted from all these experiences is illustrated in Figure 1. The basic intermittent biofiltration process is “… ideally suited to rural communities, small clusters of homes, individual residences and business establishments. [This process] can achieve advanced secondary or even tertiary levels of treatment consistently with a minimum of attention.” (Anderson, et al., 1985) It is precisely because this technology operates reliably in such a trouble-free manner that it is so well suited for the distributed treatment system environment, where close and frequent attention to system operation is simply not affordable. While competent oversight is of course required to minimize problems and correct those that occur, the level of oversight required to assure excellent performance is much less intensive than for other types of systems in common use.

The heart of the high performance biofiltration treatment process is of course the biofilter bed. The balance of the system is designed to allow the biofiltration process in the filter bed to be as stable—thus as consistent and reliable—as practical, especially when employing “elevated” forward flow HLR’s, and to enhance the ease of system operations and maintenance.

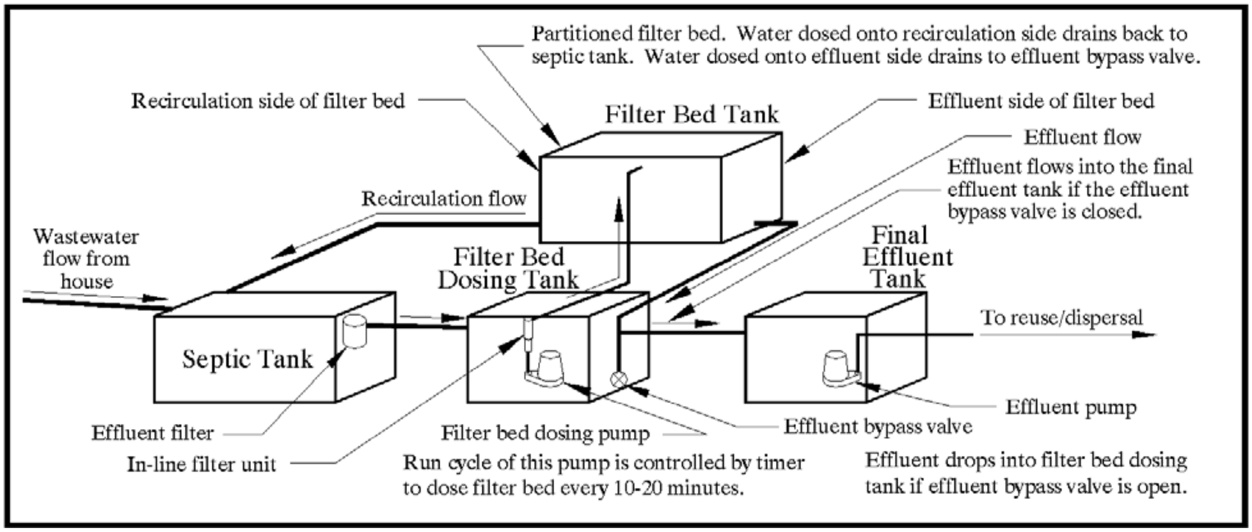

Fig. 1. High Performance Biofiltration Treatment System Concept

A key feature is good pre-clarification of wastewater prior to it being dosed onto the filter bed. Besides the obvious benefit of lowering solids loading on the filter bed, so forestalling bed clogging, Dr. George Tchobanoglous has reviewed how reducing particle size is highly beneficial to the biofiltration treatment process, as microbes “digest” smaller particles more quickly, freeing them to feed on particles coming in with the next dose. (Tchobanoglous 1999)

Basic pre-clarification is achieved by an appropriately sized septic tank fitted with a high quality effluent filter. Pre-clarification, and particle size reduction, is enhanced by a pre-filtration device, such as a Sim/Tech™ filter unit with a 600 micron filter sock, fitted onto the discharge line of the filter bed dosing pump.

Outflow from the septic tank enters the filter bed dosing tank. A pump in that tank runs intermittently to dose wastewater onto the filter beds. The dosing cycle is set up to load water onto the filter bed at a total HLR equal to the design forward flow rate plus the design recirculation flow rate, applying this water in small doses, frequently spaced in even increments of time over the diurnal cycle. In combination with the equalization volume contained in the filter bed dosing tank and the action of the effluent bypass valve, each described below, this assures that *the filter bed is loaded with the same hydraulic load on the same schedule every day*, regardless of the timing of flows into the treatment system. This enforces *true hydraulic steady-state operation* on the biofiltration treatment process, which maximizes its treatment efficiency and renders the process even more stable and robust.

After being sprayed over the top of the filter bed, the water drains by gravity down through the filter media. Note that the essential treatment action within this bed is a passive process. Power is required only to move water to the top of the filter bed. The only result of a power loss is that the filter bed dosing tank must contain the volume required to hold wastewater flowing into it during the power outage. The treatment process would resume, with no impacts on treatment quality, once the power is restored (unless the power outage lasts for days, in which case there would be a very brief "restart" period). This is in stark contrast to other technologies, most particularly activated sludge, in which power inputs are essential to maintain the treatment process, so power outages immediately and significantly compromise treatment.

The media in the filter bed may be composed of coarse sand or fine gravel—various sizes have been observed to provide good performance—or of various "artificial" media, such as geotextile fibers, foam cubes, or polystyrene beads. While the proponents of these artificial media claim they can accommodate higher HLR's than sand/gravel media, the biofilter bed—with *any* type of media—remains a fairly lightly loaded version of an attached-growth media process, so mean cell residence time is very long and holding time of the wastewater within the filter bed is relatively long. (Emrick, et al., 1997) These conditions allow the diverse biota of the filter bed, typically including many trophic levels of microorganisms and marcoorganisims (Calaway, et al., 1952, Calaway, 1957), an excellent opportunity to remove, assimilate and/or transform pollutants in the wastewater. This all renders the process inherently resistant to upsets that would cause degraded effluent quality, as suffered by other processes commonly used in the on-site environment. (Converse & Converse, 1998, Sextstone, et al., 2000) This central characteristic of biofilter processes is a major reason why they are inherently stable and robust, in particular relative to suspended-growth processes such as activated sludge, and especially in the face of variable inflow, a ubiquitous characteristic of small-scale wastewater systems.

Long-term operation is also less problematic than it is for competing technologies. The major failure mode of an intermittent biofilter is clogging of the filter bed. When clogging proceeds to the point that wastewater does not pass through the filter bed at the rate it is loaded onto it, maintenance must be performed to restore function. This condition builds up slowly over a long period (Anderson, et al. 1985), so infrequent observation of the system is needed in order to "catch" the clogging process before it proceeds to a point that it would significantly compromise performance. By insightful design, corrective procedures can be executed in short order, restoring the filter bed to full function without requiring it to be taken out of service or replaced.

As Figure 1 shows, effluent from the filter bed is split into a recirculation flow and an effluent flow. As noted, the recirculation flow is routed through the septic/recirculation tank, ending up back in the filter bed dosing tank. The effluent flow drains to the effluent tank, or drains back into the filter bed dosing tank to create a second recirculation flow, depending on the status of the effluent bypass valve, as reviewed below.

The preferred way to accomplish the flow split is by splitting the filter bed area into a recirculation side and an effluent side. The split of flow to each side in the filter bed used for normal domestic wastewater is a 2:1 or 3:1 ratio; that is, two or three gallons flows onto the recirculation side for each gallon that flows onto the effluent side. This can be accomplished by installing either twice as many or three times as many spray heads over the recirculation side as are installed over the effluent side. A typical filter bed for a home-sized system, utilizing gravel media and built in a modified septic tank, with a 2:1 flow ratio is shown in Figure 2.

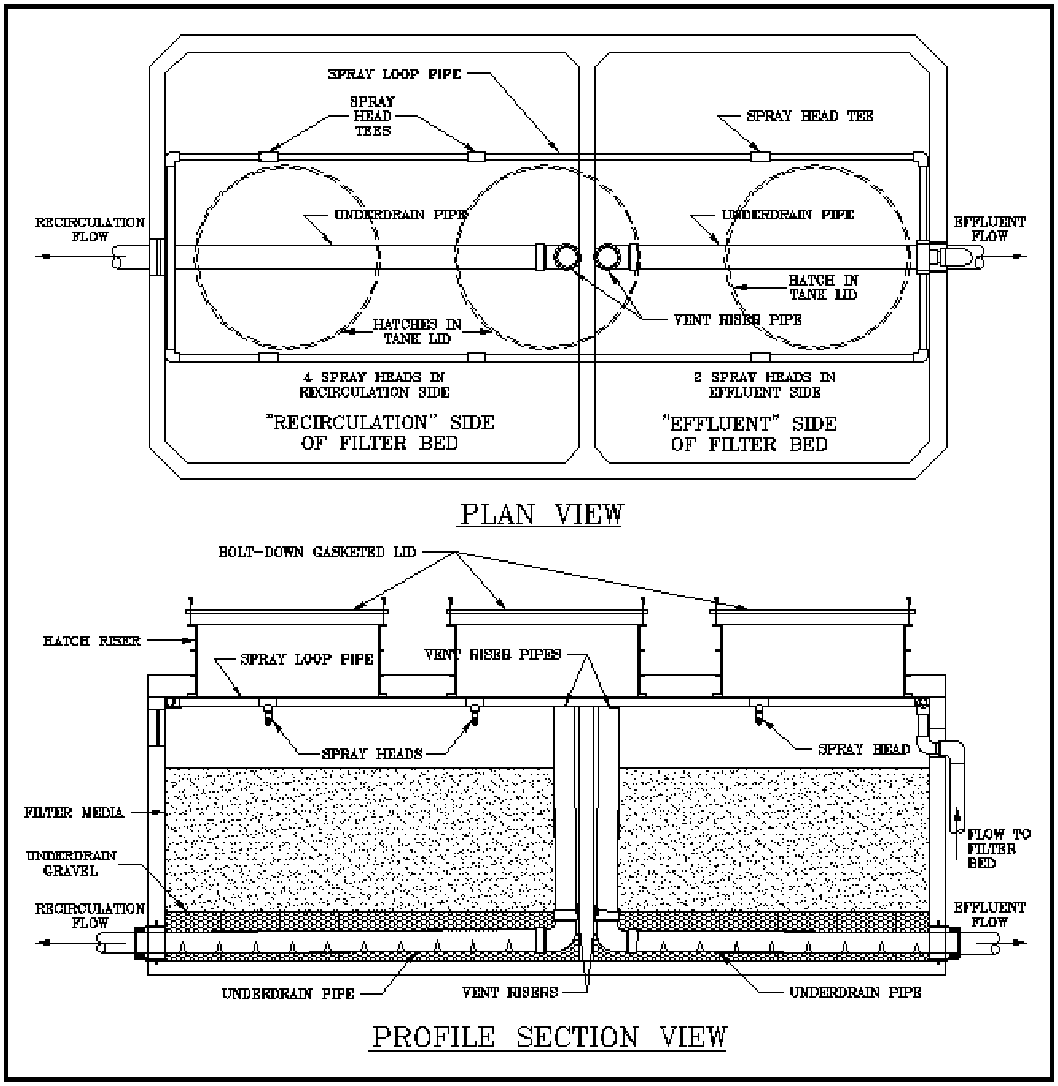

Fig. 2. Filter Bed Detail

An alternative way to create a recirculation flow and an effluent flow is with a flow splitter on the outlet pipe of a single-chamber filter bed. The flow ratio is determined by the setup of the splitter box. This method of splitting the flow is a "second choice" however because splitter boxes can be problematic. Installers relate that they must be very carefully set to ensure they split the flow evenly, and they get out of alignment if they "settle" over time or if they are accidently disturbed in the course of maintenance in and around the filter bed. However, when using manufactured units that are only available as single-chamber filter beds—e.g., AdVanTex™ or E-Z Treat™—this is the available means for incorporating them into the high performance biofiltration concept.

A flow ratio of 2:1 in the filter bed requires a minimum design recirculation rate of 3:1; that is, 3 times the daily design flow is loaded onto the filter bed—the design forward flow rate plus the design recirculation rate, which is 2 times the daily design flow. This is so because, with a 2:1 flow ratio, 1/3 of the total flow onto the filter bed flows onto the effluent side, so no more than

1/3 of the total flow onto the filter bed could drain to the final effluent tank. This requires total flow to the filter bed to be at least 3 times the daily design flow so that the daily design flow could exit the system over a 24-hour period.

Likewise, for a flow ratio of 3:1, the minimum recirculation rate would be 4:1 – the total volume loaded onto the filter bed would be 4 times the daily design flow. Thus, the higher the flow split, the higher the total HLR onto the filter bed must be. While some advocate higher recirculation rates, when the "base" HLR's are "high"—as can be supported by the high performance biofiltration concept (see further discussion below)—at some point a high flow rate through the filter bed would become problematic, possibly resulting in "excessive" wash-through of solids, especially in coarse media filter beds. It is for this reason that flow ratios of 2:1 or 3:1 are favored for systems receiving normal domestic wastewater. However, as discussed below, the recirculation rate can be increased to any rate above the required minimum if that is would be beneficial to the treatment goals of a particular system.

The effluent bypass valve, illustrated in Figure 3, operates on water level in the filter bed dosing tank. As long as water level in this tank remains above a certain depth, hydrostatic pressure on the buoyancy chamber keeps the ball stopper of the valve pressed tightly against the throat, and the valve remains closed. When water level drops to the depth of the top of the permanent liquid depth—equal to the bottom of the equalization volume—the buoyancy chamber floats down and the ball stopper falls away from the throat, opening the valve.

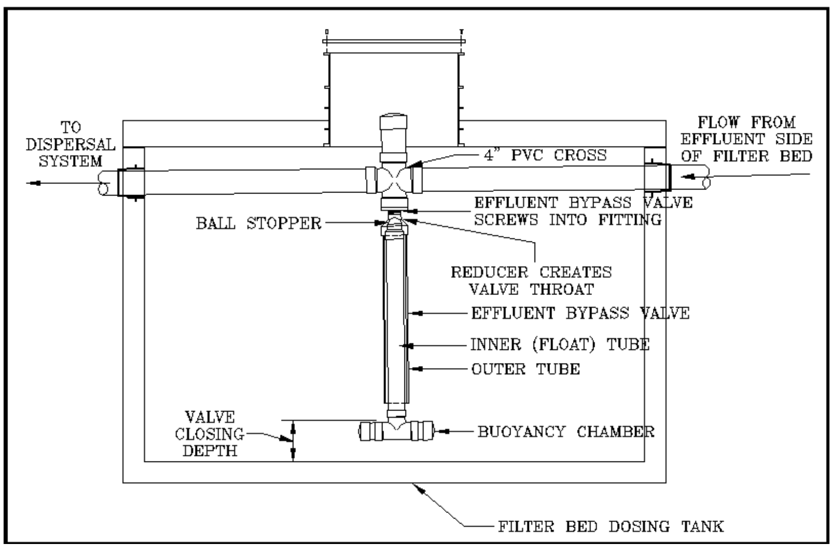

Fig. 3. Effluent Bypass Valve

Using a filter bed with a flow ratio of 2:1 and setting up the filter bed dosing pump to achieve a design recirculation rate of 3:1, the daily design flow is the maximum amount that could drain into the final effluent tank on any given day, since this would be the volume applied to the effluent side of the filter bed. The design flow rate would pass through as long as the effluent bypass valve remains closed. It would remain closed as long as the influent flow rate is equal to or greater than the design flow rate, since under these conditions at least as much water would be coming into the filter bed dosing tank over any given time period as is being evacuated from it, so the water level would not be drawn down to the "open" level of the effluent bypass valve. This situation is illustrated on the left side of Figure 4.

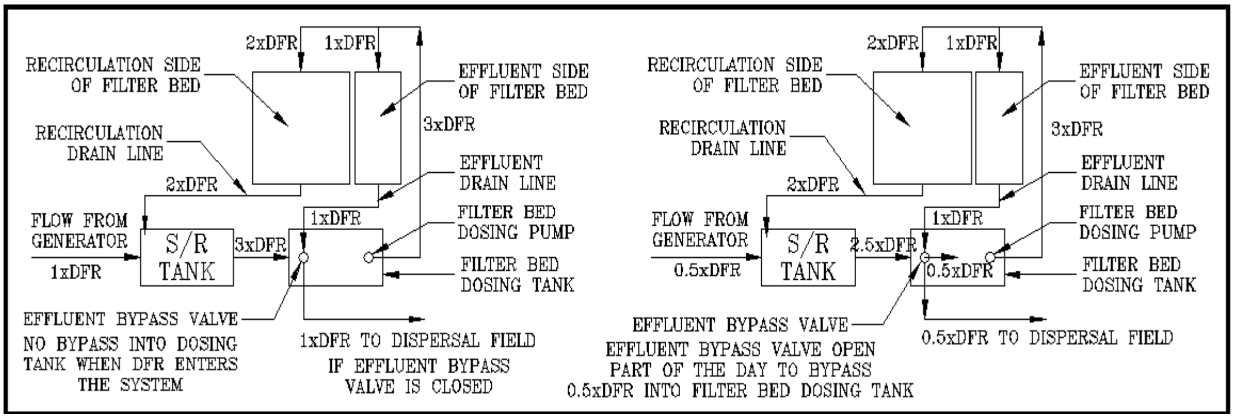

Fig. 4. Effluent Bypass Valve Impact on System Flows

However, when forward flow is less than the design flow rate, water level in the filter bed dosing tank would be drawn down, since more water would be evacuated from the filter bed dosing tank than would be entering it. When the water level drops to the depth at which the effluent bypass valve opens, then drainage from the effluent side of the filter bed would drop back into the filter bed dosing tank instead of draining to the final effluent tank. That situation is illustrated on the right side of Figure 4. This has two impacts on the overall treatment process.

First, when forward flow is consistently below the design flow rate, there would be a dilution of the wastewater in the filter bed dosing tank from the effluent side drainage that enters when the effluent bypass valve opens. As discussed below, this can be used to good effect in systems that receive wastewater of higher strength than normal domestic wastewater.

Second, opening the effluent bypass valve creates a situation in which all of the water loaded onto the filter bed returns to the filter bed dosing tank, some of it through the recirculation loop as it always does, and the rest through the effluent bypass valve. The water level in the filter bed dosing tank can therefore never drop any significant level below the depth at which the effluent bypass valve opens. Thus, the filter bed dosing tank can never run out of water, no matter how little flow enters the system. This is what allows the filter bed to always be loaded at hydraulic steady state regardless of the system's inflow characteristics. It also allows the system to keep on working without compromising the treatment process in applications where the flow generation is intermittent—e.g., vacation properties, park facilities.

The design flow rate might, of course, be exceeded on any given day. This is accommodated by the equalization volume in the filter bed dosing tank. Figure 5 displays a filter bed dosing tank profile drawing, showing this volume. On days when more than the design flow rate enters the system, the excess is stored in the equalization volume. Storage may also occur on a shorter time scale during the day. Since flow into most small-scale treatment systems is episodic, the short-term average rate of forward flow into the system will be above the daily design flow rate during periods of high water use, while the flow out of the filter bed dosing tank in each dose—and thus averaged over time—is always constant. It is because the "excess" flow is stored, to be drawn down when the inflow rate decreases, that the dosing rate can remain constant. The effluent bypass valve remains closed until the stored "excess" water is drained away, lowering water level in the dosing tank to the depth where the valve opens.

The amount of equalization storage required is based on experience. For typical residences, it has been observed that a volume equal to 2/3–3/4 of the daily design flow is sufficient to provide storage of surges in forward flow without exhausting the equalization volume. The required fraction may vary depending on how "conservative" design flow criteria are. As system size increases—being fed by more houses—the peaks tend to be a lower percentage of the average, so larger systems could use relatively smaller equalization volumes. For systems that have differing peaking characteristics than single-family houses—e.g., a lodge or camp that has greater occupancy on weekends—the designer should provide an analysis to determine the appropriate amount of equalization storage vs. design flow rate.

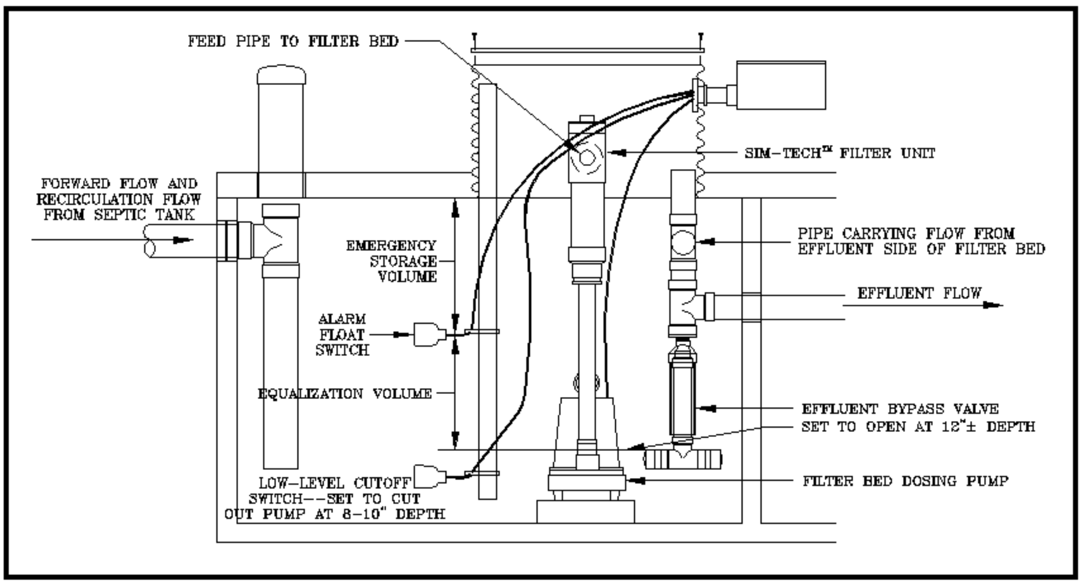

Fig. 5. Filter Bed Dosing Tank and Pump System

As noted, a 3:1 recirculation rate with a filter bed having a 2:1 flow split, or a 4:1 recirculation rate with a filter bed having a 3:1 flow split, is a typical setup for systems receiving normal domestic wastewater. The same filter bed flow ratio can be used as well for higher strength wastewaters, employing a higher recirculation rate, along with a "de-rating" of the forward flow HLR onto the filter bed. The higher recirculation flow plus flow into the filter bed dosing tank through the effluent bypass valve—enforced at points during each diurnal cycle by the increased recirculation rate—assures that the filter bed would be loaded with a more "diluted" wastewater. Given the appropriate adjustments in forward flow HLR and recirculation rate, the filter bed would be loaded with a wastewater having an organic strength in line with what would be loaded onto it under a "normal" setup if the raw wastewater were normal domestic wastewater.

Results from the Washington Island project (and also of several other investigations of this technology—e.g., Swanson and Dix, 1987) indicate that an average forward flow loading rate of

7 gallons/ft^2/day or more (for normal domestic wastewater) can be supported when utilizing gravel media. (As noted, proponents of "artificial" media claim higher HLR's can be accommodated.) It was realized, however, that splitting the filter bed into a recirculation side and an effluent side offered the opportunity to use a different design HLR on each side. As long as nitrification is maintained, it does not really matter if the quality of the recirculation side effluent degrades a bit, as it just flows back through the septic/recirculation tank. So the design criteria chosen for the high performance biofiltration concept when utilizing gravel media "push" the recirculation side HLR to 8.5 gallons/ft^2/day and, to further enhance the effluent quality from the effluent side, decrease the HLR on that side to 5.5 gallons/ft^2/day.

With a 4:1 recirculation rate on the design flow rate, applied to a filter bed with a 3:1 flow ratio, these forward flow HLR's dictate a total HLR onto the recirculation side of about 34 gallons/ft^2/day and onto the effluent side of about 22 gallons/ft^2/day. Long term total HLR's in this range were supported by the Washington Island systems (and those in other investigations—e.g., Swanson and Dix, 1987), with excellent performance results. As stated above, for higher strength wastewaters the "de-rating" of forward flow HLR, in conjunction with a higher recirculation rate, still results in a total HLR onto the filter bed similar to these rates.

Use of a higher recirculation rate may also have merit when flow into the system is highly irregular. Using the example of a filter bed with a 2:1 flow ratio, for which the minimum recirculation rate would be 3:1, when an elevated recirculation rate is used, then more than the daily design flow could flow out of the treatment system. For example, if a 4:1 recirculation rate were employed, the flow loaded onto the effluent side of the filter bed during a 24-hour period would be 4/3 of the daily design flow rate, so this is the amount that could flow out of the system during a day, if that much flow were to enter the system in a day. This ability to evacuate more than the design flow rate in a 24-hour period may have utility for a system in which the peak and minimum flow days are highly skewed around the average, or "design", flow rate. That is a luxury afforded by a treatment process that can readily accommodate fluctuating loads.

As noted previously, the systems evaluated in the Washington Island project utilized a pumped recirculation concept rather than the gravity recirculation scheme employed in the high performance biofiltration concept. This difference in the treatment concept may call to question the validity of basing design parameters on results obtained in the Washington Island project. However, the two concepts, each illustrated in Figure 6, are essentially equivalent in function.

In the pumped recirculation concept, the total recirculation flow remained the same regardless of the amount of flow into the system, since it was provided by a pump that was run by a timer. The actual recirculation rate would vary over any given time period, depending on the actual amount of forward flow into the system. In the high performance biofiltration concept, the total recirculation flow also remains the same regardless of the amount of forward flow since the filter bed dosing pump is run by a timer and the split between the recirculation side and the effluent side of the filter bed is always the same. So in both concepts, the amount of flow back through the septic/recirculation tank is constant, based on the timer setup. The actual recirculation rate over any time period would vary with the actual amount of forward flow in exactly the same way, so that the level of dilution of the forward flow would vary with the actual amount of forward flow in exactly the same way. Under either concept, filter bed effluent flows to the effluent tank after having, on average, passed through the system the same number of times—the routing is just a bit different.

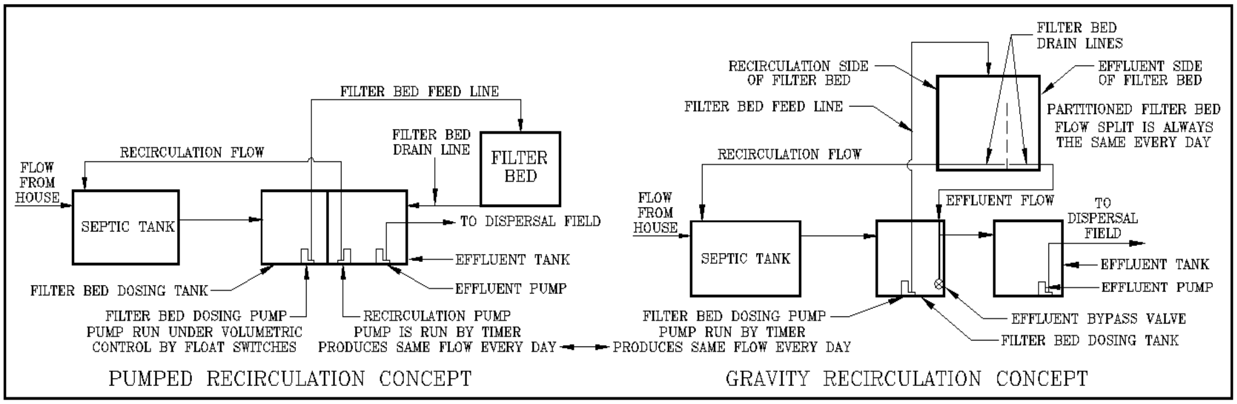

Fig. 6. Pumped and Gravity Recirculation Concepts

When using the pumped recirculation concept, however, flow rate onto the filter bed would vary with the actual forward flow, and the timing of the filter bed doses would also be affected. This is because the filter bed dosing pump was started and stopped by float switches—that is, run under volumetric control. With no forward flow, a maximum dosing interval would be determined by the recirculation flows. When forward flow entered the system, this would add to the flow entering the filter bed dosing tank, the dose volume would build up more quickly and a shorter dosing interval would result. In the high performance biofiltration concept, those variations are eliminated so that true steady-state hydraulic loading of the filter bed is obtained. This being the case, performance of the high performance biofiltration concept may be superior to that obtained using the pumped recirculation concept. Therefore, the Washington Island project data can be used as a valid indication of the performance to be expected from the high performance biofiltration treatment concept.

For organics and solids removal, the Washington Island project data indicate that removal rates would typically be in the 95%+ range, resulting in effluent concentrations typically <15 mg/L for BOD_5 and TSS, with <10 mg/L not at all unusual. (Venhuizen, 1994) These results were maintained over a range of forward flow HLR's on the filter bed and of organic loading rates on the system, and without regard to variability of influent strength or inflow rates. The systems also recovered, with no operator intervention, from various "anomalous" events—and in the case of one system, periods of extremely high organic loading—in short order, with minimal periods of degraded effluent quality. All this confirms that this treatment concept is indeed very stable and robust. The "anomalies" were either addressed by the high performance biofiltration concept—e.g., removing the recirculation pump—or could have been avoided with more reliable components or by more timely O&M procedures.

This underscores that while O&M liabilities of this process are low and generally not time-critical, they still must be carried out in a reasonably timely fashion. The list of those liabilities is brief. Once the system is set up and the filter bed dosing pump timer has been set to provide the appropriate recirculation rate, the system essentially "operates itself" without needing active intervention. Control systems set off alarms when a component may need attention—e.g., a failed pump—and arrangements must be made for timely response. All parts, however, typically perform with very high reliability. The only routine maintenance procedures are:

- Periodically checking that all components are functioning properly;
- Changing filter socks in the Sim/Tech™ filter units 3-4 times per year;
- Cleaning effluent filters in septic tanks, suggested to be done annually;
- Pumping septic tanks and dosing tanks at need, typically on intervals of several years;
- Cleaning the filter bed when clogging starts to occur, expected to be required once every 5-10 years (for gravel media filters).

NITROGEN REDUCTION CAPABILTY

As noted previously, enhanced nitrogen removal is obtained by routing recirculation flow through a septic/recirculation tank prior to reentering the filter bed dosing tank. The nitrogen cycle in the high performance biofiltration treatment concept is illustrated in Figure 7. Nitrification in the filter bed is typically fairly complete, so most of the nitrogen in the filter bed effluent is in the nitrate form. When recirculated through the anoxic environment of the septic/recirculation tank, the nitrate is biologically transformed to nitrogen gas, which bubbles off into the atmosphere.

The degree of nitrogen removal depends on the ratio of recirculation flow to effluent flow—as that flow is not denitrified—and on the effectiveness of the denitrifying environment in the septic/recirculation tank. With this being the case, the theoretical "limit" on nitrogen removal rate would be the flow ratio between the recirculation side and effluent side of the filter bed.

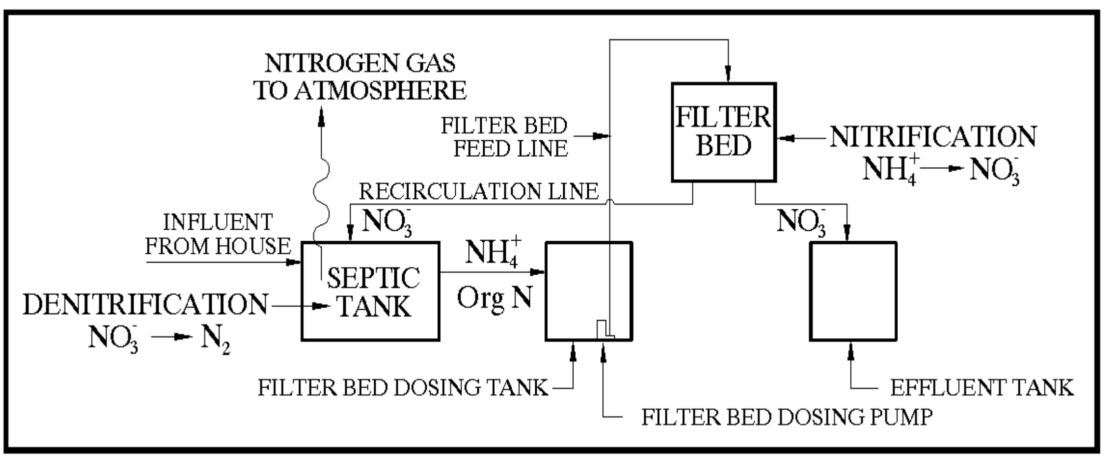

Fig. 7. Nitrogen Cycle in the High Performance Biofiltration Concept

For example, if the flow split were 2:1 – that is, two gallons flows onto the recirculation side of the filter bed for every one gallon that flows onto the effluent side – then the removal rate "limit" would be 2/3, or 67%, since 2/3 of the total flow onto the filter bed would flow through the recirculation loop, where the nitrate it carries could be denitrified. Likewise, if the flow ratio were 3:1, then 3/4, or 75% removal, would be the "limit". This would be valid as long as the design flow rate is continuously maintained. At lower influent flow rates, the actual recirculation rate would increase, so the removal rate may also increase. However, this would also result in the ratio of recirculation flow to forward flow into the septic/recirculation tank being greater, which at some point may compromise anoxia in the septic/recirculation tank, decreasing the degree of denitrification obtained. The removal rate could be further limited due to incomplete nitrification in the filter bed. The overall nitrogen removal rate might be increased above this "limit", however, due to denitrification occurring within the filter bed. Thus the actual nitrogen removal rate may vary somewhat, based upon actual operating conditions.

The systems in the Washington Island project exhibited total nitrogen removal rates in the range of 60-90%. The percentage removal generally increased with increasing total nitrogen content of the system influent, and it exhibited some correlation with recirculation rate. These findings are explored in detail in another paper in these proceedings. Total nitrogen concentration in the effluent averaged less than 15 mg/L. Again, these removal rates and effluent concentrations were maintained without regard to variability of influent total nitrogen concentrations or system inflow rates, attesting again to the stability and robustness of this treatment concept.

EXAMPLE SYSTEMS

Single-Home System/Small Office Building

This example illustrates the manner in which this technology is deployed to serve a single-family home or a small commercial generator with a similar design flow rate. The system configuration is shown in Figure 8. If used for a single-family home, the design flow rate for that set of tanks could be as high as 420 gpd. In Texas, that accommodates a 6-bedroom house (the sizing formula is design flow rate = 60 x the number of bedrooms+1).

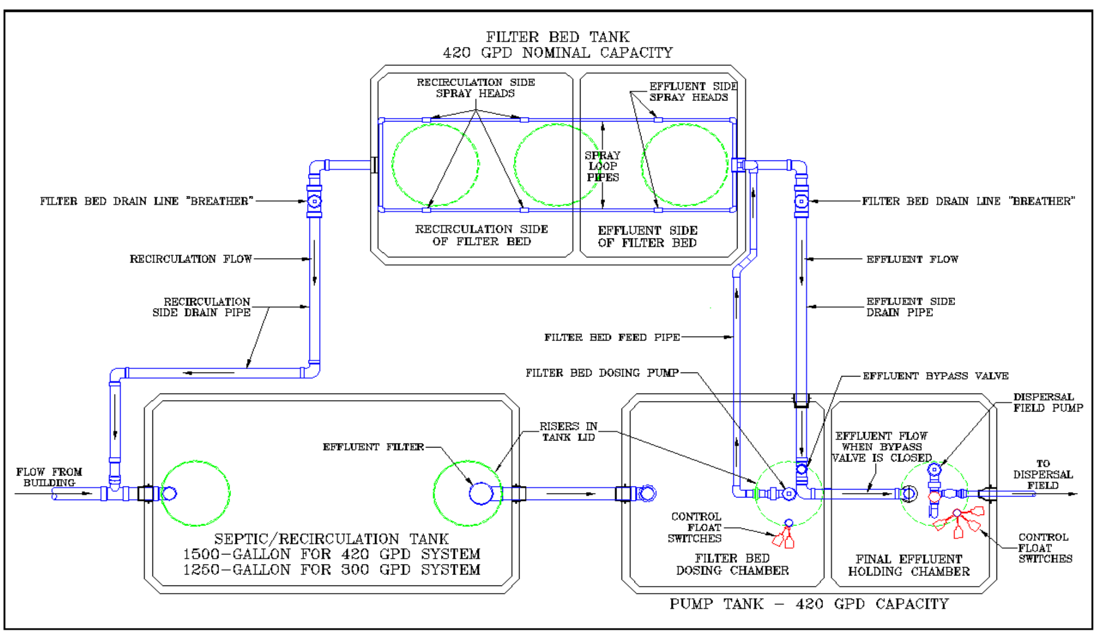

Fig. 8. House-Size Treatment System Plan

The same tanks are being used in a system to serve the Hays County Precinct 4 office building in Dripping Springs, Texas. That system offers an example of "de-rating" the filter bed and increasing the recirculation rate when influent strength would be higher than normal domestic wastewater. The flow from the county building is estimated at 300 gpd, and it would be flush water dominated. The estimated raw wastewater strength is 400 mg/L of BOD_5.

This design "de-rates" the nominal 420 gpd filter bed to 300 gpd, decreasing the HLR's to about 70% of the "nominal" rates. The Texas standards define normal domestic wastewater as having a BOD_5 level of up to 300 mg/L. This implies that, with 70% "de-rating", the system can accommodate an influent strength of 300/0.7 = 429 mg/L. The filter bed used has a flow ratio of 2:1—4 spray heads on the recirculation side, 2 on the effluent side—so as detailed previously the minimum recirculation rate would be 3:1. This is increased to 4:1 for this system, to dilute the strength of the wastewater applied to the filter bed. This 4/3 increase in recirculation rate matches the expected 4/3 increase of influent strength, from 300 to 400 mg/L. The total HLR onto the filter beds would be 300/420 x 4/3, or about 0.95 times the nominal rate. These modifications will result in the filter bed receiving an influent with characteristics and at a flow rate very similar to those in a 420 gpd system receiving normal domestic wastewater.

Elysium Condominium Project

This project is located in Austin, Texas, and consists of 5 homes and 2 small commercial buildings, one a realtor's office and one an organic cookie bakery and small bistro. The treatment system layout is illustrated in Figure 9, and a picture of the installed system is shown in Figure 10. The design flow rate generated by the users was estimated at 1,550 gpd. The filter beds are housed in modified septic tanks, the same design as illustrated in Figure 2 and Figure 8. As Figure 9 shows, there are 4 filter beds in this system, each rated at 420 gpd, providing a total design capacity of 1,680 gpd. The project was installed in 2000 and has been operating largely trouble-free ever since. Maintenance is provided by a licensed maintenance company.

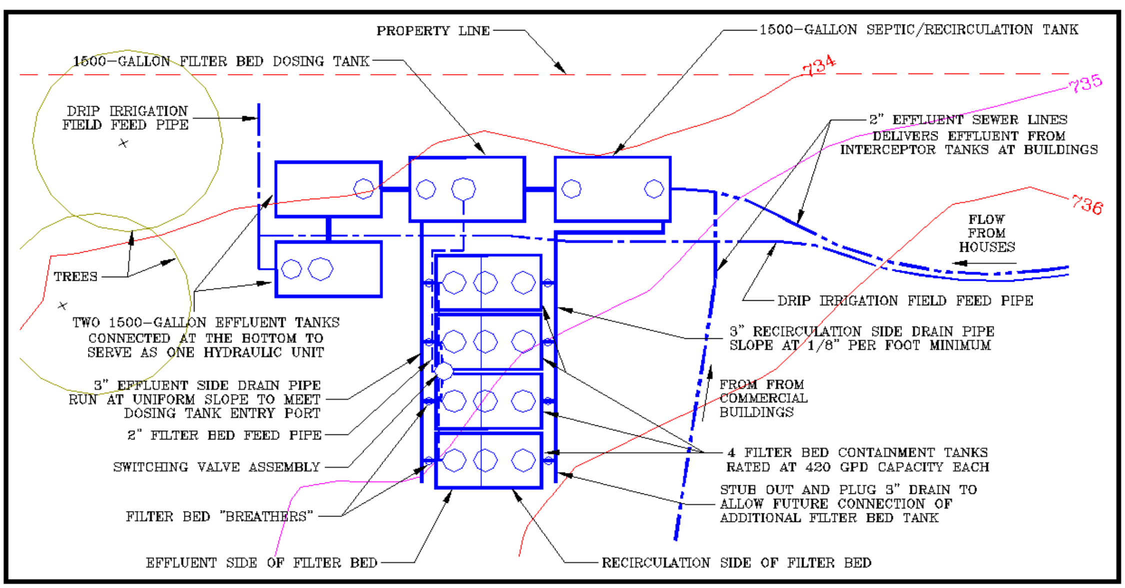

Fig. 9. Elysium Treatment Center Plan

Fig. 10. Elysium Treatment Center Photo

Primary treatment is provided in interceptor tanks located near the houses and commercial buildings, and effluent from these tanks flows through effluent gravity sewers to the treatment center. There it mixes with flow from the recirculation sides of the filter beds in the septic/recirculation tank, then flows into the filter bed dosing tank. The filter beds are fed sequentially, one at a time, through a Hydrotek switching valve. The filter beds "breathe" through vented ports installed in the drain pipes, venting into the soil around them. One of the commercial buildings is in the background in Figure 10, showing that this treatment center can be installed in very close proximity to buildings without odor problems. The final effluent flows into a set of holding tanks, from where it is dosed into drip irrigation fields. In Figure 10, the grass "stripping" shows that a portion of the drip irrigation field was installed around tanks.

In this climate, no thermal protection of the tanks is required to prevent freezing, or even to keep wastewater temperatures from dropping so low that nitrification/denitrification is compromised. The owners were not concerned about aesthetics at this treatment center location, so the tops of the filter bed tanks were left exposed, as shown in Figure 10. If aesthetics is a concern, or the system is installed in a climate where thermal protection is needed, the tanks can be surrounded by planter beds or covered by a berm and/or insulated cover, as illustrated in the next example.

Santuario de Chimayo

The Santuario de Chimayo is a major Roman Catholic pilgrimage destination, reported to be the largest in the western hemisphere, located in Chimayo, New Mexico, about an hour's drive north of Santa Fe. Wastewater derives mainly from the toilet building used by the pilgrims, so it is flush water dominated, and highly urine-dominated at that, so a high degree of nitrogen reduction is required in the treatment process in order to protect the local groundwater from nitrate pollution. The other major challenge faced by a treatment system for this project is the highly variable wastewater flow rate. There is an extreme peak during Holy Week, high flows through late spring, summer and early fall—peak-week, average-day flow was estimated at 2,500 gpd and average daily flow in the peak period was estimated at 2,000 gpd, but actual flows have been somewhat below that due to lower than expected pilgrimage volume the last two years—and little flow through the winter, when average daily flow is estimated to be about 300 gpd.

The high performance biofiltration treatment concept was chosen to cope with these constraints. The treatment center layout is illustrated in Figure 11, and a photo of the installed center—before the covers were installed on the filter bed containment tanks—is shown in Figure 12. Flow from the restroom building and a restroom in a gift shop runs by gravity into primary septic tanks, located remotely from the treatment center. A pump station there doses primary effluent to the treatment center, where it flows through two septic/recirculation tanks, then into the filter bed dosing tanks. These are two tanks connected at the bottom, providing the required capacity while utilizing off-the-shelf septic tanks, modified to suit.

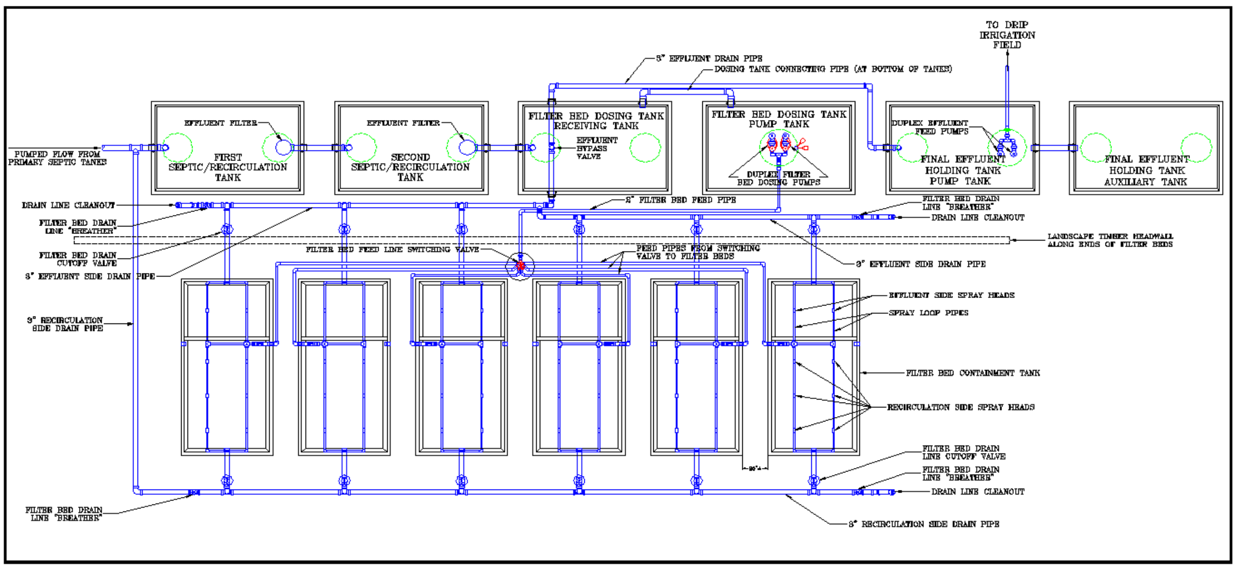

Fig. 11. Santuario de Chimayo Treatment Center Plan

The filter beds are housed in a modified two-chamber septic tank manufactured by a local tank maker. These filter beds have a flow ratio of 3:1—6 spray heads on the recirculation side, 2 on the effluent side. Filter bed sizing was based on observations of the Washington Island project system that experienced very high organic loading rates through the summer. The design organic loading rate (OLR) imparted by the peak-week, average day flow (2,500 gpd at a presumed average influent strength of 400 mg/L BOD_5) was set at a rate somewhat below the peak period OLR on that Washington Island system. This dictated the need for the six filter beds utilized in this design. Even though this design may "push" the OLR on the system under peak period conditions, the very low flow through the winter allows the system to essentially "rest" for a few months, as the HLR—and thus the OLR as well—drops very low through that period.

A filter media of the desired size—1/8"-1/4" washed rock—could not be located in Santa Fe and environs. All rock in that size range was found to be very "dirty" and would have required considerable further screening and washing to provide a suitable media. This forced the use of a media somewhat larger than desired, with an average size of about 3/8”—similar to the largest media used in the Washington Island systems, so it was used with high confidence that it would deliver proper performance. This media is shown in the photo in Figure 13, which also shows the spray pattern over the filter bed. In this system also, the filter beds are dosed sequentially, one at a time, through a Hydrotek switching valve.

Fig. 12. Santuario de Chimayo Treatment Center Photo

Fig. 13. Santuario de Chimayo Filter Bed

The filter beds are surrounded by soil, supported on one side by a headwall between them and the rest of the system tanks, as shown in Figure 12. They were topped with wood and shingle covers, with 2" rigid insulation board clad to the underside of the roof deck. Although this does not provide an airtight seal, adequate freeze protection is provided, and no odors emanate from the filter beds when wastewater is sprayed over them. In any case, the treatment center is located well away from the active use areas of this site.

The effluent from this treatment unit is distributed in a subsurface drip irrigation system to irrigate landscaping installed in and around the parking areas, a valuable water conservation measure in this arid climate. Considering plant uptake and in-soil denitrification, the regulatory system stipulated that the treatment system effluent should contain a total nitrogen concentration of <20 mg/L. When the system was started up in the spring of 2006, fairly complete nitrification by the filter beds was established in short order. However, because the wastewater was so urine-dominated, sludge was very slow to build up in the septic/recirculation tanks, and the recirculation flow was sufficiently oxygenated that anoxic conditions could not prevail in the septic/recirculation tanks. This was dealt with by adding dog food to these tanks, creating an artificial "sludge" layer to impart a greater oxygen demand. Also, because forward flows proved to be somewhat below the estimates, the total flow rate onto the filter beds was decreased, thus decreasing the recirculation rate and consequent transfer of oxygen through the septic/recirculation tanks with the recirculation flow. These measures were expected to allow significant denitrification to occur. To date, further reports of nitrogen removal performance are not available.

Other than this issue, a malfunction of the effluent bypass valve—remedied in short order by replacing the valve—has been the only problem reported in the treatment unit. That this system accommodates the highly variable loading imposed on it is another testament to how versatile and reliable the high performance biofiltration concept has proven to be.

REFERENCES

Anderson, D.L., R.L. Siegrist, and R.J. Otis. 1985. Technology assessment of intermittent sand filters. U.S. EPA, Office of Research and Development, Municipal Environmental Research Laboratory, Cincinnati.

Calaway, W. T. 1957. Intermittent Sand Filters and their Biology. *Sewage Works Journal*, Vol. 29, No. 1:1-5.

Calaway, W. T., W. R. Carroll, and S. K. Long. 1952. Hetertrophic Bacteria Encountered in Intermittent Sand Filtration. *Sewage and Industrial Wastes*, Vol. 24, No. 5:642-653.

Clark, H. W. and S. Gage. 1909. *A Review of Twenty-One Years of Experience Upon the Purification of Sewage at the Lawrence Experimental Station*, Fortieth Annual Report of the State Board of Health of Massachusetts, Public Document No. 34.

Converse, J.C, and M.M. Converse. 1998. Pump chamber effluent quality following aerobic units and sand filters serving residences. *On-Site Wastewater Treatment*, Orlando, FL, Vol. 8, ASAE Pub. 03-98:388-402.

Dymond, R. L. 1981. Design Considerations for Use of On-Site Sand Filters for Wastewater Treatment. M.S. Thesis, Institute for Research on Land and Water Resources, Pennsylvania State University.

Emrick, R.W., R.M. Test, G. Tchobanoglous, and J. Darby. 1997. Shallow intermittent sand filtration: microorganism removal. *The Small Flows Journal*, Vol. 3, no. 1, pp.12-20.

Furman, T. deS., W. T. Calaway, and G. R. Grantham. 1955. Intermittent Sand Filters—Multiple Loadings. *Sewage Works Journal*, Vol. 21, No. 6:1002-1015.

Hines, M. H. and R. E. Favreau. 1975. Recirculating Sand Filters: An Alternative to Traditional Sewage Absorption Systems. *Home Sewage Disposal*, ASAE Pub. Proc-175:130-136.

Mitchell, D. T. 1991. Personal communication.

Piluk, R. J. 1988. Field Study of an Innovative On-Site Wastewater Disposal System for Nitrogen Reduction. M.S. Thesis, Graduate School of the University of Maryland.

Piluk, R. J. and E. C. Peters. 1994. Small Recirculating Sand Filters for Individual Homes. *On-Site Wastewater Treatment*, Vol. 7, ASAE Pub. 18-94:310-318.

Sandy, A. T. III. 1987. Nitrogen Removal Using a Batch Recirculating Bottom Ash Filter. Problem report submitted in partial fulfillment of requirements for M.S., College of Engineering, West Virginia University.

Sexstone, A., M. Aiton, G. Bissonnette, K. Fleming, K. Kinneer, K. Hench, T. Bozecevich, B. Cooley, and E. Winant. 2000. A survey of home aerobic treatment systems operating in six West Virginia counties. *Small Flows Quarterly*. EPA National Small Flows Clearinghouse, Morgantown, W.V., pp. 38-46.

Swanson, S. W. and S. P. Dix. 1987. Onsite Batch Recirculation Bottom Ash Filter Performance. *On-Site Wastewater Treatment*, Vol. 5, ASAE Pub. 10-87:132-141.

Tchbanoglous. G. 1999. Keynote address at 1999 NOWRA Conference, and subsequent personal communication.

Teske, M. G. 1979. Enhanced Treatment for Surface Discharge. *Individual Onsite Wastewater Systems*, Proc. of the Sixth National Conference:137-145.

Venhuizen, D., J. H. Wiersma, and J. A. Williams. 1998. Washington Island Project: Evolution of the Denitrifying Sand Filter Concept. *On-Site Wastewater Treatment*, Orlando, FL, Vol. 8, ASAE Pub. 03-98:470-479.

Venhuizen, D. 1994. *Demonstration Systems Performance Analysis—Final Report*. Town of Washington Wastewater Management Facility Plan.