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Recirculating Media Filter Technology Assessment and Design Guidance

Iowa Department of Natural Resources

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NOTICE

This document has been reviewed in accordance with the Iowa Department of Natural Resources policies and procedures and has been approved for publication. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

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Application

This document provides guidance for the design of recirculating media filters (RMF). RMFs are a fixed film wastewater treatment system capable of producing better than secondary quality effluent. They are suitable treatment systems for both surface and subsurface discharge. Recirculating media filters should be restricted to domestic strength waste applications.

Performance

Effluent quality from recirculating sand filters in the upper Midwest will typically produce effluent with single-digit BOD, TSS and ammonia. The following table shows typical effluent quality from well designed and operated recirculating sand filters.

Parameter	Typical Effluent Concentration, mg/L		Typical Removal Rate
	Summer	Winter	
BOD	2 - 10	3 - 15	96%
TSS	2 - 10	2 - 10	96%
NH3	ND - 5	1 - 20	87%
P	3 - 5	3 - 5	50%
DO	3 - 5	6 - 12	n/a

Recommended Design Parameters

- Primary treatment is required prior to the recirculating media filter
- Recirculation tanks sized for 1.0 X daily flow
- Domestic strength waste only
- Applicable to 25,000 gpd or less
- Effluent screens to 1/8" opening
- Hydraulic loading rate 5 gpd/ft^2 or less
- Recirculation rate Provide for up to 4:1
- Organic loading rate 0.005 lb BOD/ft^2/day or less
- Media Effective Size 1.5-2.5 mm
- Media Uniformity Coefficient UC < 2.0
- Media depth 24 inches
- Minimum of 2 filter cells
- Each cell served by 2 pumps in alternation
- Dosing frequency 48 per day or more

• Pressurized distribution	
• Orifice size	1/8"
• Orifice spacing	2 feet
• Lateral spacing	2 feet
• Dose volume	Less than 2 gal.orifice/dose

Design Process
The general design procedure outlined in this manual follows these steps:

<u>Step 1 - Determine design requirements</u>
a. Characterize design flow rates
b. Characterize influent wastewater makeup
c. Determine effluent discharge location and limits

<u>Step 2 - Size pretreatment unit</u>
a. Septic tank size, number and layout
b. Tank configuration
c. Effluent screens

<u>Step 3 - Size Recirculation Tank</u>

<u>Step 4 - Size Sand Filter and Distribution System</u>
a. Select hydraulic and organic loading rates
b. Determine filter size that satisfies both hydraulic and organic loading rates
c. Determine optimal filter layout
    i. Length
    ii. Width
    iii. Lateral and orifice spacing
    iv. Select nominal pump flow rate
    v. Determine number of cells
    vi. Determine number of zones
d. Select media gradation
e. Select media depth

<u>Step 5 - Size dosing pumps and controls</u>
a. Select range of recirculation ratio
b. Determine number of pumps needed
c. Select dosing volume per orifice
d. Provide operator with recommendations on pump cycle times, dose volumes and frequency based on flow, wastewater strength and system performance.

Step 6 - Determine size, number and location of filter underdrain collectors
a. Select liner material
b. Select number, size and type of underdrains
c. Select drain perforation size, shape, location on the pipe, and spacing
d. Select underdrain bedding media gradation and depth

Step 7 - Size flow splitter elements
a. Size recirculation pipe to splitter
b. Determine type of flow splitter
c. Size splitter elements

Step 8 - Size downstream elements
a. Disinfection (if applicable)
b. Outfall pipe, or
c. Soil absorption system

Step 9 - Determine hydraulic profile and set elevations

I. INTRODUCTION

A. Scope

The Iowa Department of Natural Resources (DNR) has commissioned this manual in order to broaden the number of treatment options considered for managing wastewater within Iowa's small rural communities. Current rules and regulations do not address recirculating sand, gravel or other media filters. This manual is intended to expedite the design and review process for these technologies by:

- Summarizing existing research and performance data;
- Acting as a guide to determining the applicability of recirculating media filters;
- Advising the designer as to the selection and sensitivity of design parameters;
- Providing an overview of the design process; and
- Providing three example designs for populations of 25, 100, and 250 people.

The manual has application for:

- Treatment of Domestic Waste Only; and
- Population Equivalents from 25-250 people.

The following assumptions on waste quantity and strength have been used throughout the manual:

- Design influent BOD of 250 mg/l or less;
- Design influent TSS of 250 mg/l or less;
- Design influent TKN of 40 mg/l or less; and
- Design Hydraulic Loadings of 100 gpcd

This manual is intended for use by Owners, Consulting Engineers, DNR review engineers and associated DNR personnel, as well as funding source personnel to provide guidance to the successful design for the use of recirculating media filters within Iowa. The design approach contained within this manual should be construed as a minimum basis of design. Nothing within this manual should be construed or viewed as eliminating additional alternative treatment systems, or alternative design approaches with respect to recirculating media filters, provided that adequate justification and data from actual installations is submitted.

B. Terminology

Definitions of some terms used in this evaluation report are as follows:

ADW	Average Dry Weather Flow Rate. ADW is average daily flow when groundwater is at or near normal and a runoff condition is not occurring. The period of measurement for this flow should extend for as long as favorable conditions exist up to 30 days, if possible
AWW	Average Wet Weather Flow Rate. AWW is the daily average flow for the wettest consecutive 30 days for mechanical plants, or for the wettest 180 consecutive days for controlled discharge lagoons
Ammonia	A naturally occurring inorganic form of nitrogen I combination with hydrogen. Total ammonia includes unionized ammonia (NH_3) as well as ionized ammonium (NH_4^+) The proportion between ionized and unionized ammonia depends on the pH and temperature of the solution. Ammonia is both toxic to aquatic animal life and a source of nutrition to plants.
Ammonification	The decomposition of organic nitrogen to ammonium by decomposing organisms.
Biochemical Oxygen Demand	The biochemical oxygen demand (BOD) of domestic and industrial wastewater is the measure of the amount of molecular oxygen required to stabilize the decomposable matter present in water by aerobic biochemical action as determined by a standard laboratory procedure.
Denitrification	The process of biologically converting nitrate/nitrite (NO_3^-/NO_2^-) to nitrogen gas.
Infiltration	The water entering a sewer system (including service connections) from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include, and is distinguished from, inflow.
Infiltration/Inflow	The total quantity of water from both infiltration and inflow without distinguishing the source.

Inflow	The water discharged into a sewer system (including service connections) from such sources as, but not limited to, roof drains, cellar, yard and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers and combined sewers, catch basins, storm water, surface runoff, street wash waters, or drainage. It does not include, and is distinguished from, infiltration.
MWW	Maximum Wet Weather Flow. MWW is the total maximum flow received during any 24 hour period when the groundwater is high and a runoff condition is occurring.
Nitrification	The process of biologically oxidizing ammonia (NH_4^+/NH_3) to nitrate/nitrite (NO_3^-/NO_2^-).
Pathogen	A disease producing microorganism
PHWW	Peak Hourly Wet Weather Flow Rate. PHWW is the total maximum flow received during one hour when the groundwater is high, runoff is occurring and the domestic, commercial and industrial flows are at their peak.
Sanitary Sewer	A sewer intended to carry only sanitary or sanitary and industrial wastewater, from residences, commercial buildings, industrial plants, and institutions.
Suspended Solids	Those solids that either float to the surface of, or are suspended in water, sewage, or industrial waste which are removable by a laboratory filtration device.
Total Kjeldahl Nitrogen	The sum of the organic and total ammonia nitrogen present.
Total Nitrogen	The sum of organic nitrogen, total ammonia nitrogen and nitrate + nitrite nitrogen.

Abbreviations of some terms used in this report are as follows:

BOD	BOD_5, the five-day biochemical oxygen demand
cfs	cubic feet per second
DNR	Department of Natural Resources (State of Iowa)
EPA	United States Environmental Protection Agency (Federal)
FOG	Fats, Oils and Grease
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
HRT	hydraulic retention time
lb/day	pounds per day
lb/cap/d	pounds per capita per day
MGD	million gallons per day
mg/L	milligrams per liter
ND	not detectable
NH_4-N	ammonia nitrogen
NO_3-N	nitrate nitrogen
RGF	Recirculating Gravel Filter
RSF	Recirculating Sand Filter
STEG	Septic Tank Effluent - Gravity
STEP	Septic Tank Effluent - Pumped
TKN	Total Kjeldahl nitrogen
TN	Total nitrogen
TSS	total suspended solids
WWTF	Wastewater Treatment Facility

C. Discharge Performance Capability

In general, recirculating media filters are capable of producing a very high quality effluent, rivaling that from activated sludge systems. A well-designed and operated recirculating sand filter (RSF) treating pre-settled domestic strength wastewater in a climate similar to that of Iowa can be expected to produce effluent in the range of that shown in Table 1-1. Detailed data on which these ranges are based is presented in Chapter 3 and in the Appendix.

Table 1-1
Expected Effluent Quality From Recirculating Sand Filter System

Parameter	Typical Effluent Concentration, mg/L		Typical Removal Rate
	Summer	Winter	
BOD	2 - 10	3 - 15	96%
TSS	2 - 10	2 - 10	96%
Ammonia-nitrogen	ND - 5	1 - 20	87%
Phosphorus	3 - 5	3 - 5	50%
Dissolved Oxygen	3 - 5	6 - 12	n/a

The above data is valid for effluent wastewater temperatures as low as 3 degrees C in the winter months. The performance of an individual system is influenced by a variety of design and operational issues, each of which will be discussed in this design guidance.

Bacteria levels, as characterized by fecal coliforms, are reduced in media filtration treatment, but not typically to the levels required for surface water discharge. A separate disinfection process typically follows a media filter that discharges to surface water to ensure compliance under all conditions.

II. PROCESS DESCRIPTION

A. Background of Recirculating Sand and Gravel Filters

Media filtration is a term that generally describes an aerobic, fixed-film bio-reactor used to stabilize pre-treated, domestic strength wastewater. Rather than a strictly physical process as implied by the "filtration" moniker, media filtration in this context employs a combination of physical, chemical and biological processes to produce a high-quality effluent that may meet requirements for discharge to surface waters, depending on receiving water criteria, and for sub-surface soil dispersal.

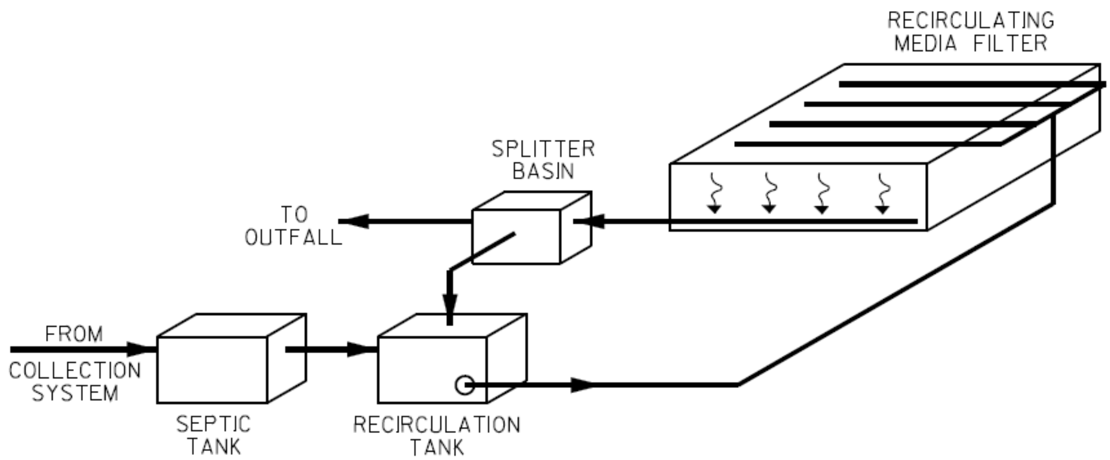
The "media" can be any of a number of physical structures whose sole purpose is to provide a surface to support biological growth. Commonly used media have historically included rock, gravel, and sand of various sizes. Newer variations include textile media, open cell foam, peat, coir and chipped tires. Research projects have evaluated crushed recycled glass, sintered glass, and boiler ash as potential sources of media. This technology evaluation will focus on the most common materials.

The category of treatment referred to as media filtration includes a number of variations on the process. They can be broken down into subcategories based on how many passes through the filter the wastewater makes, whether the filter surface is open to the air or buried, and the relative size and type of the media (sand, gravel, textile or other).

In all cases, pretreatment of the wastewater to reduce the BOD and suspended solids content of raw sewage is required. Once settling is accomplished, the pre-treated wastewater is applied to the filter surface in small doses, to alternately load and rest the media. As wastewater percolates down through the filter bed, it comes into contact with the bacterial film growing on the media. The filtrate is contained by an impermeable liner, and collected in an underdrain. The underdrain pipe directs the filtrate to a flow splitting structure, in which a portion of the flow can be diverted back to the recirculation tank for additional treatment, with the rest discharged as effluent. Where total nitrogen removal is desired, recirculation back through the settling tanks provides contact between the nitrate-laden filtrate and carbon-bearing influent in the presence of bacteria.

A schematic of typical media filtration systems is shown in Figure 2-1.

Figure 2-1
Recirculating Sand Filter Schematic



The following subsections provide a more complete description of each of the elements of a media filtration system.

B. Application

Recirculating media filters are suitable wastewater treatment technologies for both surface and subsurface discharge. They can be used for discharge to surface waters where effluent limits are at least:

- 10 mg/l BOD
- 10 mg/l TSS
- 2 mg/l ammonia (summer)
- 10 mg/l ammonia (winter)

With supplemental treatment, RMF systems can also meet effluent limits for fecal coliforms. Removal of phosphorus down to 1 mg/l may also be possible by addition of metal salts to the primary treatment tank, but little full-scale data exists to confirm that.

RMF systems are excellent systems for treatment prior to subsurface dispersal in a trench, mound, or drip irrigation system. The additional degree of treatment provided in an RMF allows for a reduction in the sizing of the dispersal component, as it allows the native soil to act as a conduit for dispersal of the treated effluent rather than as a medium for treatment.

C. Typical Size Requirements

RSF systems serving design flow rates of up to 25,000 gpd can generally be accommodated on sites of 1 acre or less. Depending on the strength of the influent wastewater, the type of collection system and the type of primary treatment, the actual filter bed will typically only require between 500 and 5,000 square feet for wastewater flows ranging from 2,500 to 25,000 gpd. The remaining area is required for septic tanks, access roads, earthen berms, and for a buffer between adjacent development. This low land requirement makes RSF systems an attractive option compared to pond systems, which require several times more land than an RSF system.

A gently sloping site is ideal for an RMF system. Typical headloss requirements are 6-10 feet, but can also be accommodated on a level site with the appropriate earthwork.

D. Relative Cost

It is not possible to provide meaningful cost data that applies equally to all applications and maintains its timeliness. It can generally be stated that an RMF system will be similar in terms of overall life-cycle cost to an aerated lagoon system, while providing a higher quality effluent and requiring less land. Compared to a small activated sludge system, an RMF can provide effluent quality that rivals that of a mechanical treatment system for about two-thirds the cost. While the capital cost savings may only be about 10%, the real savings come in the cost of operation and maintenance. Power costs and the cost of the labor to properly operate and maintain the RMF system is typically half that for an activated sludge system.

E. Process Description

1. Primary Treatment by Septic Tanks

Primary treatment is required prior to all forms of recirculating media filtration. The goal of primary treatment is to prevent fouling of the filter from suspended solids and from excessive bacterial growth due to BOD overloading. To accomplish this, the primary treatment process must provide a quiescent environment to promote settling of suspended particles, as well as allowing for contact between the influent wastewater and naturally occurring anaerobic bacteria. This bacteria may be both in suspension as well as in the settled layer at the bottom of the primary treatment unit. Successful primary treatment can occur in many forms, but this guidance will focus on the two most common forms, septic tanks and lagoons. Appendix A contains a detailed discussion on the sizing, layout and maintenance considerations associated with primary treatment for alternative small community wastewater treatment systems.

2. Filter Dosing

After primary treatment, the wastewater flows into a dosing tank, also frequently referred to as a recirculation tank. This compartment is effectively a wet well in which the primary treatment unit effluent is blended with water that has already been passed through the sand filter. It houses two or more timer-controlled submersible dosing pumps that are used to move water up to the surface of the filter. Once at the surface of the filter, the water is allowed to percolate down through the filter where it comes into contact with the treatment organisms living on the filter media.

Intermittent application of wastewater, or filter "dosing", is required so that the filter has time to allow the wastewater to percolate through, and then re-aerate. The aerobic bacteria responsible for treatment need air in the pore space of the media in order to obtain oxygen. If a filter were constantly dosed, the aerobic bacteria would not thrive, and the bacterial culture would change over to anaerobic or facultative organisms. Anaerobic reactions are much less efficient, produce odorous gases, and are not desirable. For this reason the dosing tank needs to be large enough to store a portion of the incoming flow as well as recirculated flow while the filter re-aerates.

Water in the distribution piping must not be allowed to freeze between doses. To prevent this, water must be allowed to rapidly drain from the pipes. This can be accomplished by not using check valves downstream of the filter dosing pumps, and by drilling the first and last orifice of each lateral on the bottom of the lateral.

3. Filter Media and Wastewater Distribution

Media is the material or product used to provide support for the attached microbial growth that will provide the aerobic biological treatment. It is not, as the name might imply, used primarily to provide physical filtration of influent solids, although some filtration does occur. Media is among the most important elements of a recirculating media filtration system, and is among the most costly.

An ideal media will have the following properties:

- High surface area to volume ratio
- Large enough voids to allow for rapid air infiltration and to minimize fouling
- Good weathering properties, including
  - UV resistance if exposed to sunlight
  - Physical wear and soundness
  - Low solubility in water and acidic conditions
- Be cost-effective and locally available

These generic properties allow for a variety of materials to be used in a recirculating media filter. The vast majority of the experience is with sand and gravel media. Research has also been done using crushed recycled glass (Emerick 1997; Elliot, 2001; Hu and Gagnon, 2005), boiler ash and slag (Sack, 1989), peat (Apfel, 1991; Boyle, 1995; Solomon, 2000) and other granular material. Peat systems are commercially available and are no longer considered an emerging technology in some states.

Private manufacturers have also brought proprietary products to the market, such as textile, open cell foam and coir (ground coconut hulls) media. When considering proprietary media systems, the designer should evaluate the media with the same parameters used for granular media. These include allowable loading rate (either surface area of filter or specific surface area of media, whichever is more appropriate to the filter type), durability, maintainability, expected performance, and cost.

Due to the many different types of proprietary media available now and in the future, it is not possible to give uniform loading rate guidelines for these systems. Rather, the design should require the media manufacturer to provide sufficient documentation as to the performance of their media under the manufacturer's suggested loading conditions. The designer should look for documentation that the desired level of performance was achieved at a similar temperature to what the designer expects his application to experience.

Because of the evolving nature and limited data set of these proprietary media systems, this manual will focus on non-proprietary, sand and gravel media filters. The difference between a recirculating *sand* filter and a recirculating *gravel* filter is largely a matter of semantics. Although "gravel" implies a coarser filter media than does "sand", the typical gravel and sand filter media are very similar. Geologists use the Krumbein or Wentworth Scales to define classes of soil grains by size. Under these scales, "sand" refers to particle sizes up to 2 mm, and "gravel" refers to particles larger than 2 mm. For the purposes of this document, recirculating sand filter (RSF) media will refer to media having an Effective Size ($D_{10,}$ the diameter at which 10 percent by weight of the material is finer; also ES) of up to 2 mm, and recirculating gravel filters (RGF) will refer to media having a D_{10} larger than 2 mm. Most of the media in use falls near to the 2 mm dividing line between sand and gravel, and includes particle sizes both above and below this size.

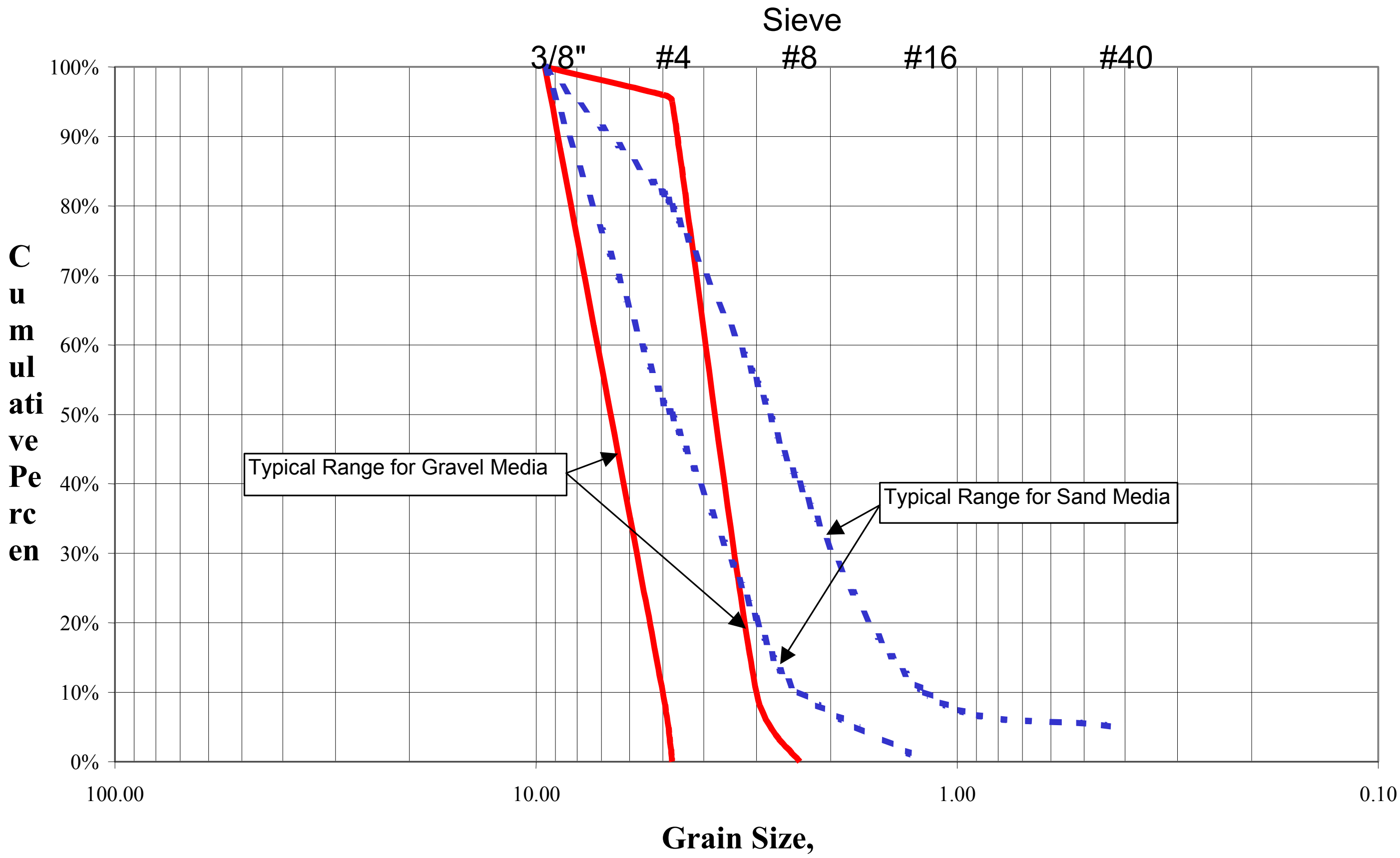
a) Grain Size

Some of the earliest work on recirculating media filtration was performed by Hines and Favreau in the 1970's using sand media having a D_{10} of 0.3 mm (Loudon, 1984). A variety of studies comparing treatment performance and fouling of media for varying effective size have followed, and include Boyle, 1995; Darby, 1996; and Zaplatikova, 2006. In general, these studies have found that media size has the greatest impact on performance for single pass and infrequently dosed filters. In these cases, fine-grained media (0.25 – 0.3 mm) will provide better treatment than coarser media due to the high surface area to volume property of fine-grained soils. This difference in performance was reduced by increasing dosing frequency and by providing recirculation. Once it was demonstrated that similar performance could be expected from a variety of media sizes, media selection became based more on extending the longevity of a filter run and minimizing maintenance than on treatment performance.

Since this early work, recirculating sand filter media size has generally increased to 1.5 – 2.5 mm D_{10}, with some jurisdictions such as the state of Oregon moving toward a fine gravel media having D_{10} of 3 to 5 mm (Bergstrom 1995).

Figure 2-2 shows typical grain size distribution curves for sand and gravel filter media.

Figure 2-2
Typical Grain Size Distribution Curves for Sand and Gravel Filter Media



Effective Size, a key element in the selection of media is the absence of fines. Most successful media specifications require that less than two percent by weight of the media be able to pass through a #40 sieve, and less than four percent passing a #16 sieve.

b) Uniformity

The other key characteristic of granular media is its uniformity. To prevent the accumulation of smaller particles within the void spaces of larger particles, which would lead to clogging of the filter, all research has recommended a relatively uniform, or poorly sorted, media. The degree of uniformity is described by the Uniformity Coefficient (UC), which is the ratio of the D_{60} to the D_{10}. The lower this number, the more uniform the media. The highest allow UC is typically 4, with many specifications requiring a UC of 2.5 or less. In general, the lower the uniformity coefficient, the less prone to fouling the media will be, but the cost of the media will likely increase due to the additional volume of raw material that must be screened to manufacture the media.

c) Depth

Whereas much of the earlier guidance on recirculating sand and gravel filters suggested a media depth of 36" or more, more recent research has found that lesser depth is necessary (Anderson, 1985; Darby, 1996). The majority of the biological activity has been found to occur in the upper 9 to 12 inches of the bed (Anderson, 1985). Others reported results using a filter depth of 15" that were comparable to those from previous studies using deeper filter beds (Darby, 1996). As media is one of the more expensive elements of a media filtration system, any ability to safely minimize the quantity will result in significant cost savings. Based on these studies, a filter bed depth of 24" has been commonly used in Wisconsin, Massachusetts, Rhode Island and other states. It provides for some safety factor, and would allow for removal of several inches of fouled media, if necessary, without replacement.

d) Selection

Virtually any granular media will successfully support biological growth that will treat wastewater with some degree of success. There is no one right size and gradation. All, however, offer tradeoffs, and it is the role of the designer to select the best fit for a particular application.

The following general relationships with respect to media size have emerged as a result of much research and actual experience. These relationships apply to granular media between 0.3 mm and 5 mm in size.

As media size increases,

- Time to fouling increases;
- Maintenance decreases;
- Allowable hydraulic loading rate increases (filter area becomes smaller);
- Media life may be extended; and it is
- Less prone to freezing.

But…

- Higher recycle rates may be necessary, resulting in greater power consumption; and
- Better distribution of water may be necessary.

e) *Wastewater Distribution*

It is of further importance that, once a media size and gradation have been selected, the designer must apply a method of distributing the wastewater that is appropriate for that specific media. Fine media will allow fewer distribution points to be used because the slower percolation rate will cause the wastewater to pond and spread out naturally over the filter bed. One example of this type of distribution system is the use of a few, large spray nozzles which discharge the water in the center of a filter bed. This has the advantage of making the majority of the filter surface accessible for weeding, raking and tilling.

Coarse media will not allow wastewater to pond and spread as readily, and will require the wastewater to be distributed evenly over the surface of the filter bed. This distribution is frequently accomplished by a network of perforated plastic pipe lain on or above the surface of the filter bed. The pipes convey water pumped by the dosing pumps and carry it to all points on the filter surface. Water is then applied to the filter surface through a series of orifices, or holes drilled into the pipe. While achieving uniform distribution, the pipes can get in the way of maintaining the filter surface. For this reason, the pipes may be connected with removable couplings, allowing the manifold pipes to be removed when the filter bed requires maintenance.

4. Liner and Underdrain

In order for wastewater to be recirculated, it must be collected after filtration so that it can be mixed with incoming septic tank effluent and sent back to the filter for additional contact with the treatment organisms. Therefore all recirculating sand and gravel filters must have an impervious bottom so that partially treated wastewater does not escape. Single family sand filters are often constructed in concrete tanks, but community scale filters typically use earthen sidewalls with a synthetic liner placed at the bottom and up the sides. The liner material most commonly used is 30-mil PVC. The liner should be placed on a prepared soil subgrade that is free from sticks, roots and the like. A 2-3 inch layer of clean sand is often placed over the subgrade before liner placement to cushion the liner.

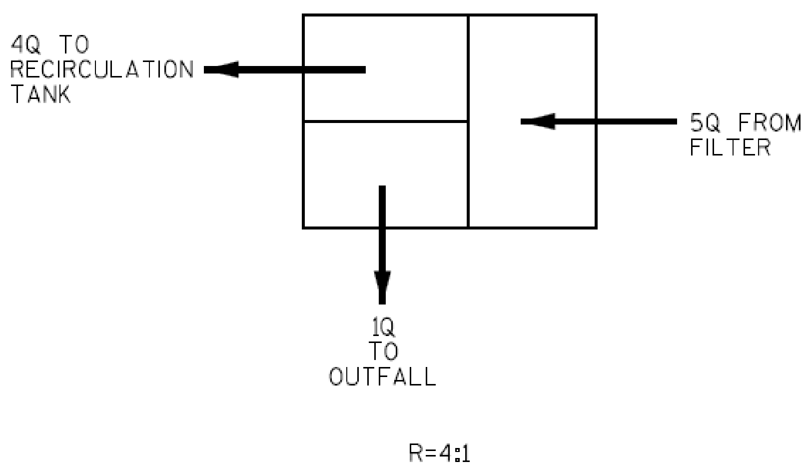
Perforated collection pipe laid on top of the liner is typically used to convey filtrate which collects on the liner back to a flow splitting structure. The underdrain pipe is typically vented to the surface to allow in air, and is often bedded in clean stone of larger diameter than the filter media. The underdrain media should be large enough to not blind the underdrain pipe openings, and should be sized to support the overlying treatment media. Geofabric shall not be placed between layers of media. Early designs using geofabric to separate media layers exhibited high rates of failure due to fouling of the fabric.

5. Flow Splitting and Recirculation

The operator must determine the degree of treatment needed in order to meet a permit limit. Additional treatment can be obtained by recirculating the filtrate back to the dosing tank, from which it will make an additional pass through the filter. The portion of the flow routed back to the dosing tank relative to that portion of the flow discharged as effluent is quantified as the recirculation ratio (R). Recirculation ratios typically range between 3:1 and 7:1, with 4:1 being typical.

Figure 2-3 illustrates this concept for a R = 4:1, or simply 4.

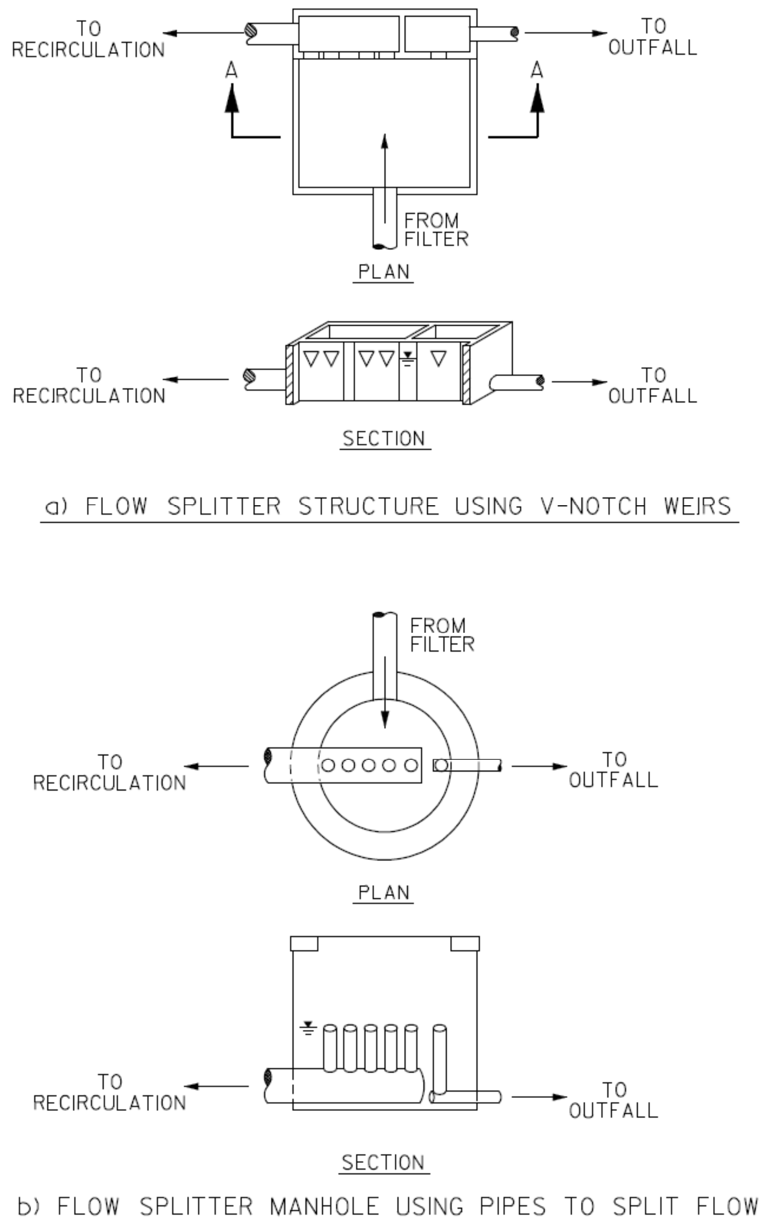
Figure 2-3
Recirculation Ratio



Each pass through the filter media provides additional contact time with the treatment organisms and results in a higher degree of treatment. The total number of passes through a filter is determined by the recirculation ratio, R, and is equal to R+1. While a higher recycle ratio generally provides a better treatment, it requires more energy to pump the water through the filter each additional time. There can also be harmful effects of recirculation ratios that exceed 7:1 or 8:1. A high R can deplete alkalinity due to complete nitrification, and thus drive pH below acceptable levels. Low pH can allow filamentous organisms to form and clog distribution orifices. High recycle rates in the winter can also lead to heat loss, which can inhibit nitrification. It becomes the operator's responsibility to determine the best balance between reliable treatment (more recycle) and efficient operation (less recycle).

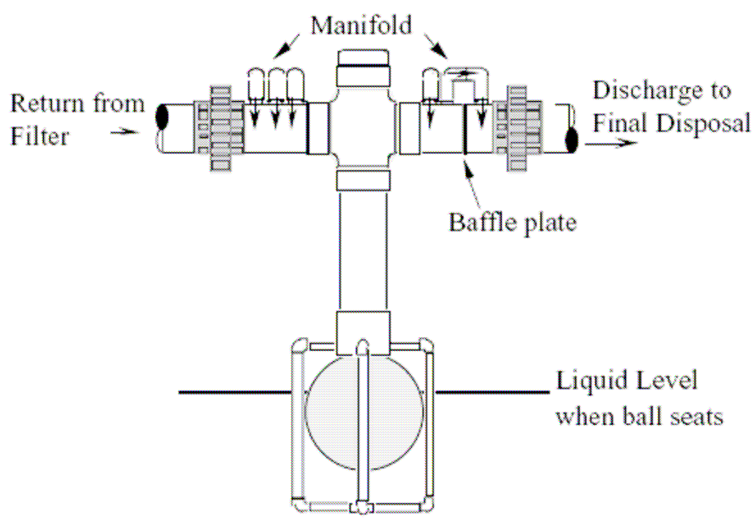
Control over the recycle rate is done with a flow splitting structure or valving located between the filter and doing tank. The ideal flow splitter will give the operator the ability to determine the recirculation ratio, and thus be able to exercise some control over the degree of treatment and energy demand. Flow splitting can be accomplished with weirs or overflow pipes as shown in Figure 2-4.

Figure 2-4
Examples of External Flow Splitting Structures



Another simple type of flow splitter is the recirculating splitter valve designed and sold by Orenco™, as shown in Figure 2-5. This simple valve allows for variable recirculation ratio while ensuring that during low-flows, the majority of the filtrate is returned to the dosing tank. This allows for the dosing pumps to maintain their normal timed dosing cycles to keep the filter wet without concern of causing a low level alarm due to lack of water. Once a minimum liquid level is reached in the recirculation tank, the ball float seats against the base of the valve, and the recirculation ratio for the remaining filtrate is determined by the number of open pipes overflowing from the manifold into the recirculation tank.

Figure 2-5
Recirculating Splitter Valve
(Ball and Denn, 1997)



Coupled with the ability to control the timer settings for the dosing pumps, the recirculating splitter valve is the simplest means for controlling the recirculation rate and for providing for low-flow recycle.

III. PERFORMANCE

A. Performance Data

Data was evaluated from a number of operating facilities in order to provide a more detailed evaluation as to the performance ability of recirculating media filters, and to provide a basis for comparing results between different styles of filter systems.

The data set includes 27 operating community-scale recirculating sand filters in Iowa and Wisconsin. The facilities range in size from about 4,000 gpd to 180,000 gpd, and in age from two years to 20 years of age. The individual data sets for each facility ranged in size from 3 to 1,039. Individual data sets for the Iowa facilities range in size from 1 to 33 points.

A variety of designs were used, including both open and buried filters. The overall data for each parameter will be presented for the entire sample group, and then will be broken down into subgroups for comparison.

1. BOD Removal Data

a) *Sand Media*

The Iowa data set is included in Table 3-1.

Table 3-1
Effluent BOD Performance from Iowa RSFs

	Mean, mg/l	**95% C.I. - Low**	**95% C.I. - High**	**Mean, May-Oct**	**Mean, Nov-Apr**
Welton	6.6	3.6	9.6	4.5	8.5
Burlington	6.0	4.3	7.7	5.8	6.5
Randalia	4.5	1.7	7.3	3.3	5.3
Panama	7.0	5.2	8.7	5.4	8.5
Country-Aire	5.5	0.0	12.7	0.5	10. 6
Marathon	6.2	5.0	7.4	4.9	7.5
Average	**5.2**	**3.3**	**8.9**	**4.1**	**7.8**

Table 3-2 directly compares the average effluent performance with respect to BOD of the Iowa and Wisconsin facilities.

Table 3-2
Effluent BOD Comparison, Iowa and Wisconsin

	Mean, mg/l	95% C.I. - Low	95% C.I. - High	Mean, May-Oct	Mean, Nov-Apr
Iowa	5.2	3.3	8.9	4.1	7.8
Wisconsin	8.3	6.9	8.9	6.2	9.3

Table 3-2 indicates that RSFs in both Wisconsin and Iowa have enjoyed a similar degree of success in terms of effluent BOD performance.

b) <u>*Gravel Media*</u>

As discussed earlier, gravel media filters are in similar in all ways to the sand filter media, with the effective diameter being slightly larger (D_{10} = 3-5 mm for gravel vs. D_{10} = 1.5-2.5 mm for sand). The state of Oregon was an early adopter of recirculating gravel filters (RGF). Table 3-3 presents effluent BOD performance for select Oregon RGF's (Bergstrom, 1995).

Table 3-3
Effluent BOD From Select Oregon and Washington RGF's

		Effluent BOD, mg/l			
			Range		
Facility	**Influent BOD, mg/l**	**Mean**	**Low**	**High**	**Percent Removal**
Oregon					
Fischer Forest Park	134	12	3	35	91%
Falls City	109	13	3	72	88%
Alsea	161	19	11	27	88%
Mill City	125	10	4	24	92%
Dexter	NR	10	5	34	-
Hebo	138	5	1	11	96%
Westport	127	9	5	36	93%
Elkton	141	7	1	22	95%
Elbe	303	5	2	16	98%
Orcis Village	146	4	1	10	97%
Washington					
South Prairie	200	8	3	28	96%
Mean	158	9	4	29	94%

The data from these facilities correlate very well with the data from the Wisconsin and Iowa RSF facilities in terms of effluent BOD concentration and overall removal rates.

2. TSS Removal

a) Sand Media

Table 3-4
Effluent TSS Performance from Iowa RSF's

	Mean, mg/l	95% C.I. - Low	95% C.I. - High	Mean, May-Oct	Mean, Nov-Apr
Welton	11.1	8.3	13.4	12.0	10.0
Burlington	3.1	2.9	3.3	4.4	3.0
Randalia	6.2	3.0	9.4	4.5	7.3
Panama	4.1	0.0	8.3	4.1	10.7
Country-Aire	0.5	0.2	1.1	1.0	0.1
Marathon	6.5	4.2	8.8	3.6	9.4
Average	**5.3**	**3.1**	**7.4**	**4.9**	**6.75**

Table 3-5directly compares the average effluent performance with respect to BOD of the Iowa and Wisconsin facilities.

Table 3-5
Effluent TSS Comparison, Iowa and Wisconsin RSF's

	Mean, mg/l	95% C.I. - Low	95% C.I. - High	Mean, May-Oct	Mean, Nov-Apr
Iowa	5.1	3.1	7.5	4.8	5.8
Wisconsin	5.5	4.4	5.9	5.1	5.0

Table 3-5 indicates that RSFs in both Iowa and Wisconsin have also enjoyed a similar degree of success in terms of effluent TSS performance. There is no discernible difference in the performance of either group of RSFs.

b) Gravel Media

With a larger effective diameter, gravel media also has a larger pore space, which would be expected to be less efficient in capturing solids. Table 3-6 shows the TSS removal performance of 11 Oregon and Washington state RGF's. As one might expect, effluent TSS was more than double that seen for the Iowa and Wisconsin RSF's, and overall removal efficiency was lower. While the performance of the coarser media with respect to solids removal appears to be worse than for finer media, it is still within an acceptable range for most secondary effluent limits.

Table 3-6
Effluent TSS performance from Oregon and Washington RGF's

Facility	Influent TSS, mg/l	Effluent TSS, mg/l			Percent Removal
		Mean	Range Low	Range High	
Oregon					
Fischer Forest Park	30	10	1	31	67%
Falls City	64	23	2	98	64%
Alsea	127	47	25	68	63%
Mill City	49	11	1	25	78%
Dexter	NR	22	4	129	-
Hebo	71	4	1	10	94%
Westport	44	7	2	23	84%
Elkton	32	6	2	16	81%
Elbe	103	4	1	10	96%
Orcis Village	116	4	1	12	97%
Washington					
South Prairie	38	17	1	64	55%
Mean	67	14	4	44	78%

3. <u>Ammonia Removal</u>

Removal of ammonia from wastewater is accomplished by biochemical oxidation of ammonia nitrogen (NH_3 and NH_4^+) to nitrite (NO_2) and finally to nitrate nitrogen (NO_3). This transformation is known as nitrification, and is accomplished in two steps by bacteria of the family *Nitrosomonas* and *Nitrobacter*. These slow-growing organisms favor surface growth rather than suspended growth, which is why fixed film processes such as granular media filtration can exhibit very good rates of ammonia removal.

Alkalinity is consumed in the conversion of ammonia to nitrate, and may be the limiting factor in the ability to nitrify a particular wastewater. 7.1 grams alkalinity as $CaCO_3$ are consumed per gram of NH_4-N oxidized. The depletion of alkalinity leads to a drop in the pH of the wastewater, which can lead to inhibition of the nitrifiers at pH of less than 6.5. Areas with soft water supplies may not naturally contain sufficient alkalinity for full nitrification to result. For example, an influent TKN concentration of 45 mg/l would require that at least 320 mg/l of alkalinity be present in order for full nitrification. If not naturally available, addition of alkalinity may be required for ammonia removal to occur.

a) *Sand Media*

Data regarding ammonia removal for the Iowa RSF data set is limited to only two facilities, containing a total of 64 data points.

Table 3-7
Effluent NH3 Performance from Iowa RSFs

	Mean, mg/l	95% C.I. - Low	95% C.I. - High	Mean, May-Oct	Mean, Nov-Apr
Country-Aire	12.0	8.7	15.4	9.6	15.8
Marathon	19.3	17.0	21.6	18.4	20.1
Average	**15.6**	**12.8**	**18.5**	**14.0**	**18.0**

Unlike BOD and TSS performance, Table 3-8 indicates that there is a difference in terms of effluent ammonia concentration between the Iowa and Wisconsin RSF sample groups.

Table 3-8
Effluent NH3 Comparison, Iowa and Wisconsin

	Mean	95% C.I. - Low	95% C.I. - High	Mean, May-Oct	Mean, Nov-Apr
Iowa	**15.6**	**12.8**	**18.5**	**14.0**	**18.0**
Wisconsin	**3.8**	**2.9**	**5.1**	**1.7**	**6.5**

b) *Gravel Media*

The data from the Oregon and Washington RGF's indicate that the coarser media can provide effective ammonia removal, similar to that experienced in Wisconsin.

Table 3-9
Ammonia Removal Performance from Oregon and Washington RGFs

	Influent	Effluent Ammonia, mg/l					
Facility	**Ammonia, mg/l**	**Mean**	**Range Low**	**Range High**	**Mean, May-Oct**	**Mean, Nov-Apr**	**Percent Removal**
Oregon							
Fischer Forest Park	29	6	1	14	4	7	79%
Falls City	22	4	2	9	5	2	82%
Alsea	25	4	2	5	4	3	84%
Mill City	33	4	<1	22	5	6	88%
Dexter	NR	3	<1	27	4	2	-
Westport	40	6	<1	15	8	3	85%
Elkton	48	7	1	16	7	7	85%
Elbe	55	3	<1	10	4	1	94%
Washington							
South Prairie	85	5	<1	34	5	6	94%
Mean	42	5	1	17	5	4	86%

4. Pathogen Reduction

Removal of pathogenic indicator organisms depends on the final discharge plan. If discharge is to soil, pathogen removal will be completed there and thus should not be considered as a parameter of concern in the final effluent. If surface water discharge is the plan, then pathogen reduction is required.

The reduction of pathogenic organisms in wastewater treatment facilities down to permit-required levels is typically achieved in a tertiary disinfection process, such as chlorination or ultraviolet disinfection. A granular media filter provides some pathogen reduction, but not to the levels required by NPDES permits, which are typically on the order of a geometric mean of 200-400 colonies of fecal coliform bacteria per 100 ml.

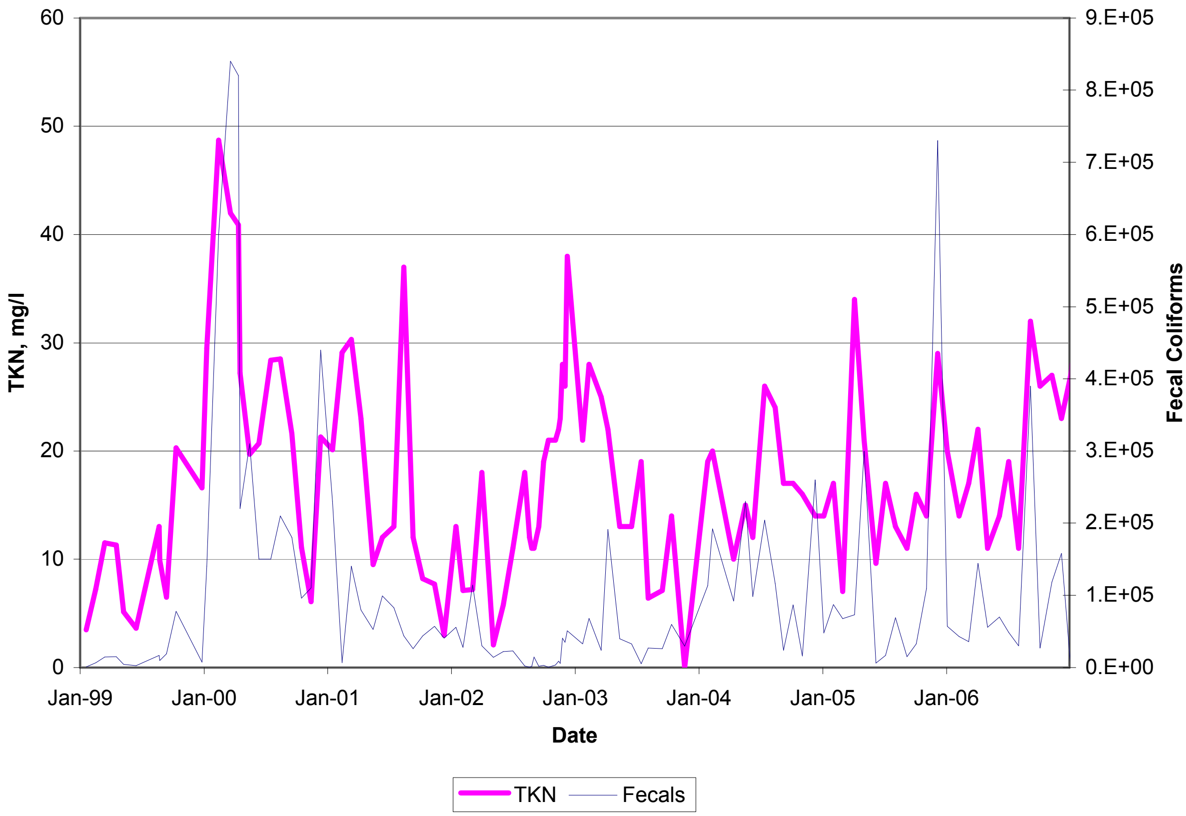
There is little full scale operating data available on pathogen reduction in RMF facilities. The literature reports that a removal of 2 to 4 logs is typical for this type of media filter, but is dependent on media size and type as well as hydraulic loading rate. Bacterial removal rates will increase as hydraulic loading rate decreases and as dosing frequency increases (Emerick, 1997). Converse (1999) found that effluent fecal coliforms in single-pass media filter effluent were less than 200/100 ml 76% of the time. Media filters that are required to meet a pathogen limit typically employ a separate disinfection process waters to ensure compliance under all conditions.

There are mixed results regarding media size, with earlier research indicating that finer media yielded better pathogen removal. More recent studies, however, found no significant difference between a sand media having D_{10} = 0.65 mm and a gravel media with D_{10} = 3.3 mm. It is felt that the use of a higher dosing frequency can minimize any impacts of media size (Emerick, 1997). For the purposes of virus inactivation, it was found that dosing frequencies of less than 12 doses per day resulted in a steadily decreasing viral removal rate, while above 12 doses per day, additional removal continued to occur, albeit at a lower rate of increase (Emerick, 1999)

One facility for which non-disinfected effluent bacteria data is available is the Indian Heights RSF operated by the Ho-Chunk Nation near Wisconsin Dells, WI. An RSF discharging to a drainfield has been monitored monthly for effluent fecal coliform bacteria for at least the past eight years. The data shows that effluent fecal coliforms have ranged between 400 MPN/100 ml to upwards of 10 million. It suggests that the filter is achieving some removal of bacteria most of the time, but not to a reliable degree and not to one that would comply with an NPDES permit limit.

The Indian Heights data also suggests that when the plant is nitrifying well, it is also doing a relatively good job of pathogen reduction. Periods when the effluent pathogen concentration is at its peak correspond to periods where nitrification was suffering. The converse is not necessarily true, as there were periods of elevated TKN, most notably in the summer of 2001 and the winter of 2003-2004 where elevated TKN did not correspond to an elevation in fecal coliforms.

Figure 3-1
Relationship Between Effluent TKN and Fecal Coliform Counts



It can be concluded that while some degree of pathogen reduction can occur in a recirculating sand or gravel filter, it is not sufficient to reliably meet an NPDES permit limit without a separate disinfection process. Pathogen removal is optimized with more frequent dosing (at least 12 cycles per day) and a low hydraulic loading rate. It can be assumed that when a filter is exhibiting good performance as evidenced by good nitrification, that it can be assumed that pathogen reduction is also being optimized.

B. Loading Rate Considerations

1. Hydraulic Loading Rates

Hydraulic loading rates (HLR) were the earliest guidelines developed for the sizing of recirculating sand and gravel filters. The HLR is calculated based on the daily forward flow divided by the filter surface area in use. It does not represent the instantaneous application rate that can be affected by recirculation rate and dosing frequency and duration.

Experience has shown that HLR's for septic tank effluent onto recirculating sand filters will result in rapid clogging at rates above 5 gallons per day per square foot of filter surface (gpd/ft^2) (Boyle, 2001). A typical design guideline for HLR is from 3-5 gpd/ft^2 for coarse sand and fine gravel media (Anderson, 1985).

There is a tradeoff between loading rates, which drive filter size, and filter run time. While a HLR higher than 5 gpd/ft^2 may work, it will be accompanied by a higher level of maintenance required to keep the filter unclogged.

The guidelines discussed thus far relate to the HLR's of septic tank effluent, implying pre-settled domestic strength wastewater. It is intuitive that clear water containing no organic material would pass through a filter more rapidly, as the void spaces would contain no biomass to slow down the rate of percolation. At the other extreme, a wastewater containing a high level of soluble organic matter will promote bacterial growth in the media, which left uncontrolled will eventually clog the void space in the media, allowing less water to pass through. It is therefore imperative that any discussion of hydraulic loading rate also consider the organic loading rate, as the amount of organic material requiring stabilization will have a greater effect on the ability of a filter to percolate water over an extended period of time. Organic loading rates will be discussed separately in a following section of this document.

2. Dosing Rate and Frequency

A subset of the hydraulic loading rate, which refers to the design daily forward flow, is the rate of instantaneous application, which is governed by the dosing frequency and duration. In general, many studies have shown benefits from increasing the frequency of dosing cycles. A study reported that removal rates in filters dosed 12 times or more per day exceeded removal rates for filters dosed 1-4 times per day (Darby, 1996). They also found that as dosing frequency increased from 4 to 24 times per day, COD removal increased from 79.3 to 93.3 percent. Another study found that for a given hydraulic and organic loading rate, increasing the number of doses per day from 48 to 96 improved BOD removal rates from 92% to 97% (Hu and Gagnon, 2005). The benefits of more frequent dosing have been found to be more pronounced for higher hydraulic loading rates, coarser media, and less uniform sand (Darby, 1996).

The benefit of increased dosing frequency can be attributed to less hydraulic pressure being put on the media to flush water through the pore spaces at a steady state rate. Wastewater is allowed to percolate more slowly, resulting in a greater contact time and a thin film flow over the biomass (Darby, 1996).

An additional benefit to a shortened dosing interval is that the instantaneous effluent flow rate more closely matches the influent flow rate. With fewer, larger volume doses, the effluent flow rate is more directly influenced by the instantaneous rate of dosing, which can result in larger downstream treatment facilities for flow measurement and disinfection.

3. Organic Loading Rates

As an aerobic biological process, it is intuitive that organic loading is a major factor in the design of a recirculating sand filter. Much of the early empirical design parameters centered on a hydraulic loading rate, based on an assumption of domestic strength wastewater. The earliest guidelines recommended that organic loading fall between 0.003 and 0.005 lb BOD/day/ft^2 (Anderson, 1984).

This recommendation appears to have been made without regard to nitrification. Assuming that the recirculation, dosing and percolation of wastewater through the media can contribute a finite quantity of oxygen to the wastewater, it is reasonable to assume that the organic loading rate for BOD removal only is higher than that allowable for applications requiring both carbonaceous BOD removal as well as nitrification. Studies have shown that an effluent ammonia concentration of less than 5 mg/L is possible with organic loadings less than 0.002 lb BOD/day/ft^2 in the summer, and less than 0.0012 lb BOD/day/ft^2 in the winter (Boyle, 2001).

Data collected at several Wisconsin RSF's have shown that summer effluent ammonia levels of less than 5 mg/l are attainable at organic loading rates of less than 0.004 BOD/day/ft^2. Winter ammonia performance data is more attributable to temperature consideration than to organic loading rates, and will be discussed in the subsequent section.

The basic relationship is that as organic loading increases, the filter run time until ponding decreases. A balance, however, must be struck, so that sufficient food is available to build and sustain a bacterial culture for treatment. Slightly better BOD removal efficiencies for an organic loading rate of 0.004 lb BOD/ft^2/day (97%) than for a rate of 0.002 lb BOD/ft^2/day have also been reported (Hu and Gagnon 2005).

Most studies have continued to compare hydraulic loading rates and their impact on filter run time, but the real influence must be recognized to be the effect of the sustained organic loading rate. For example, a fine media filter loaded at 5 gpd/ft^2 ran for over 150 days without clogging, while the same media loaded at a rate of 14 gpd/ft^2 clogged between 45 and 80 days (Darby, 1996).

Organic loading rates are a function of the waste strength and the hydraulic loading rate. The waste strength should be that of the wastewater being applied to the filter surface after pre-treatment or collection from a STEP or STEG system. Table 3-10 llustrates the variation in organic loading with respect to waste strength and hydraulic loading rate.

Table 3-10
Organic Loading Rate in lb BOD/day/sf for Varying Waste Strength and HLR

Hydraulic Loading Rate, gpd/sf	**Wastewater Strength, mg/l BOD**		
	100	**200**	**300**
1	0.0008	0.0017	0.0025
2	0.0017	0.0033	0.0050
3	0.0025	0.0050	0.0075
4	0.0033	0.0067	0.0100
5	0.0042	0.0083	0.0125
6	0.0050	0.0100	0.0150

The shaded values indicate loading rates above 0.005 lb BOD/sf or above 5 gpd/sf which are generally not recommended.

There is less published data on organic loading rates for larger media. Several states, including Washington and Massachusetts, use a loading formula of the form:

$$\text{HLR (gpd/sf)} = \frac{1150}{\text{BOD of septic tank effluent (mg/l)}}$$

when sizing a filter bed using 3-5 mm gravel media. (Washington Department of Health, 1989, Commonwealth of Massachusetts, 2006)

The implication of this formula is that an expected septic tank effluent BOD of 230 mg/l will allow a hydraulic loading rate of 5 gpd/sf. The corresponding organic loading rate for this scenario is 0.009 lb BOD/sf, which is higher than the maximum value of 0.005 lb BOD/sf often cited for recirculating sand filters. The loading rate of 0.009 lb BOD/sf is also used in the state of Oregon for RGFs (Bergstrom, 1995). This rate appears to be the maximum allowable for gravel filters, and is allowable due to the larger void spaces in the media being less prone to fouling.

C. Temperature Considerations

Wastewater temperatures in an RSF in the north-central US will typically vary from about 20 degrees C in the summer down to about 3 degrees C in the winter. Cooling of the wastewater occurs as it is brought into contact with the media, which is at or about the ambient temperature in an open RSF or RGF. High recirculation ratios contribute to a lower temperature by providing additional opportunity for cooling each time the waste is applied to the media. The cooler wastewater and ambient temperatures of the winter slow down the biological activity of the biomass living on the filter media, potentially impacting treatment efficiency.

Media filters with the distribution laterals covered by several inches of coarse stone are more common for small systems. The coarse stone still allows air into the filter media, but provides some insulation and protection from the wind. Temperature loss and hence winter ammonia removal tends to be superior in a covered filter.

Proprietary media filters will also experience less of a temperature variation due to the media being contained in an enclosure allowing for the retention of heat. Temperature data was not available for the proprietary media filters, but they are not expected to show as much variation as that seen in RSFs and RGFs.

The following sections will examine the impact of temperature on the removal of specific pollutants.

1. BOD Removal

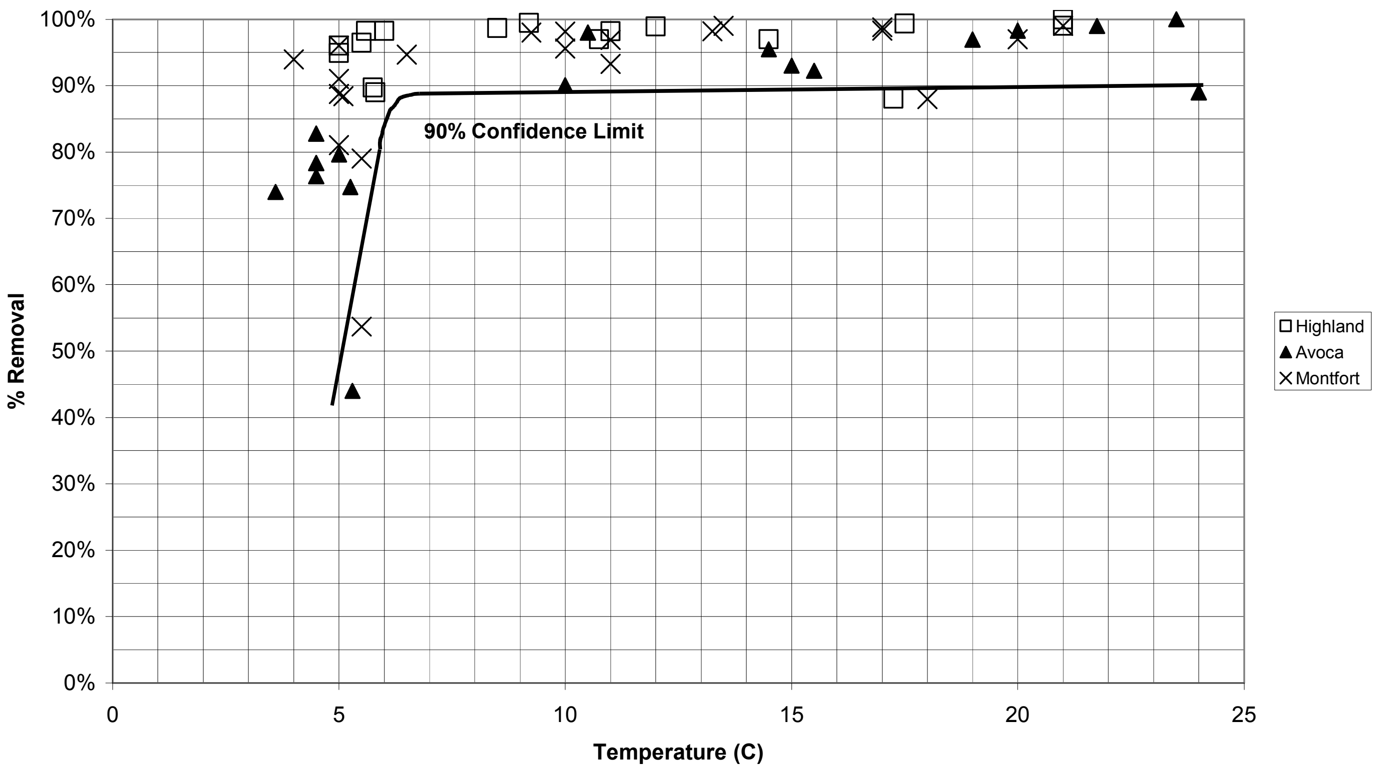
Table 3-2 shows a nominal influence of temperature on effluent BOD concentrations. Both the Iowa and Wisconsin facilities showed an increase in average winter BOD of about 50% above the summer levels. Due to the excellent overall performance of these systems, this only means an increase of 2 to 3 mg/l.

2. Ammonia Removal

It is well documented that nitrifying bacteria are temperature sensitive organisms. In reviewing the data presented earlier in this section it can be seen that the variation of effluent concentration from summer to winter is the greatest for ammonia as compared to BOD and TSS. The ratio of winter to summer effluent ammonia concentrations averages 3.8. However, individual facilities have been observed to vary from below 1.0 mg/l to above 20 mg/l from summer to winter.

Theoretically nitrification ceases at temperatures below 10 degrees C. Research on cold weather RSFs has shown that nitrification will continue unimpeded down to wastewater temperatures of about 6 degrees C. Once the temperature drops below that level, nitrification starts to become impacted, but does not cease entirely. Nitrification at levels of 70-80% has been observed at temperatures between 3 and 5 degrees C. Figure 3-2 illustrates this relationship.

Figure 3-2
RSF Ammonia Removal Efficiency



IV. IDNR BACKGROUND AND REQUIREMENTS

Under the assumptions and constraints of this manual, the DNR is the jurisdictional entity that provides oversight and approval of wastewater treatment system design and operation. As defined within §567 IAC 64, the DNR provides that oversight through the issuance of permits to construct and NPDES operational discharge permits. These permits must be obtained and authorized before any wastewater treatment system can become operational.

The reader of this manual is directed to review the requirements, as outlined within §567 IAC 64, for the currently enforced rules and regulations regarding wastewater construction and operation in Iowa.

Criteria for monitoring of any discharge are statutorily identified within §567 IAC 63. This criterion is based upon method of discharge, either continuous or controlled and the size of the facility with respect to population.

As identified within §567 IAC 64, as well as the current wastewater treatment design standards, recirculating media filters are not currently identified as a treatment process. Therefore, there is no current design standard. It is the intent of this manual to provide a non-codified standard of design and criteria for establishing constraints of implementation of recirculating media filter treatment systems.

The Reliability requirements shall be met by all designs. The designer shall contact IDNR to determine what level of reliability for the intended receiving water.

V. DESIGN GUIDANCE

As can be seen from the preceding sections, there are numerous variations on recirculating media filtration systems, all of which can be successful, but all of which also offer some trade-offs. No single filter design or operating parameter was found to adequately predict the performance of sand filters (Darby, 1996). There is a complex interdependency in the design variables; the selection of one variable will likely impact the selection of others. There is no one right way to design a recirculating sand filter. What is paramount is that the design approach be consistent so that each aspect is complementary to the others. A designer can get into trouble by mixing and matching these design approaches without regard to the interrelationship of the design variables.

There appear to be two primary approaches taken to the design of a recirculating sand filter. The first, typically used for small installations (population 250 and less), features covered header systems with closely spaced distribution headers and orifices. The design process revolves around the selection of small (fractional horsepower) submersible pumps. Small pumps are favored because they are lower in cost and involve use of small diameter pipes, small electric wire sizes, and are easily managed by a single operator. Each pump is dedicated to serving a portion of the filter. The filter is then sized and laid out to match the capacity of the pumps, which then determines how many identical pumps are needed to meet a design flow.

The second approach, typically used for larger systems (50,000 gpd and up), features fewer but larger pumps on a common manifold. Each pump can then deliver flow to any part of the filter. The design of this style of filter is typically driven by the filter size, and then pumps are selected to match the flow requirements. This type of system may require more sophisticated controls, including electrically actuated drainback and low-flow recirculation valves.

Bearing that in mind, this design guidance will focus on the design approach that is commonly used for small facilities. The design examples that follow will utilize this method. They are not the only parameters that can produce a successful system, but the burden of demonstrating the efficacy of an approach outside of these guidelines will fall upon the designer.

A. Design Process Overview

The general process used by this manual to design an RSF will be in accordance with the following steps:

Step 1 - Determine design requirements
a. Characterize design flow rates
b. Characterize influent wastewater makeup
c. Determine effluent discharge location and limits

Step 2 - Size pretreatment unit
a. Septic tank size, number and layout
b. Tank configuration
c. Effluent screens

Step 3 - Size Recirculation Tank

Step 4 - Size Sand Filter and Distribution System
a. Select hydraulic and organic loading rates
b. Determine filter size that satisfies both hydraulic and organic loading rates
c. Determine optimal filter layout
    i. Length
    ii. Width
    iii. Lateral and orifice spacing
    iv. Select nominal pump flow rate
    v. Determine number of cells
    vi. Determine number of zones
d. Select media gradation
e. Select media depth

Step 5 - Size dosing pumps and controls
a. Select range of recirculation ratio
b. Determine number of pumps needed
c. Select dosing volume per orifice
d. Provide operator with recommendations on pump cycle times, dose volumes and frequency based on flow, wastewater strength and system performance.

Step 6 - Determine size, number and location of filter underdrain collectors
a. Select liner material
b. Select number, size and type of underdrains
c. Select drain perforation size, shape, location on the pipe, and spacing
d. Select underdrain bedding media gradation and depth

Step 7 - Size flow splitter elements

a. Size recirculation pipe to splitter
b. Determine type of flow splitter
c. Size splitter elements

Step 8 - Size downstream elements

a. Disinfection (if applicable)
b. Outfall pipe, or
c. Soil absorption system

Step 9 - Determine hydraulic profile and set elevations

In addition to providing guidelines for the design of a recirculating sand filter, a set of default design parameters will be given in each section for clusters of residential developments having populations of 25, 100 and 250 people.

B. Site Selection

Recirculating media filters should be located and designed in conformance with the current Iowa DNR Wastewater Treatment Design Standards, including:

- Containment berms surrounding the filter bed that do not allow surface water run-on into the treatment area, (in accordance with the wastewater facility design standards);
- Protection against 100-year flood events; (in accordance with the wastewater facility design standards);
- Vertical separation from maximum ground water and bedrock (in accordance with the wastewater facility design standards);
- Liner systems below the media filter beds should provide the same level of containment as Lagoon systems within Iowa (in accordance with the wastewater facility design standards).

In addition, the site should be large enough to accommodate the required filter area, leaving room for backslopes and future expansion. A typical RSF of up to 25,000 gpd can generally be accommodated on sites of 1-2 acres. A gently sloping site that can provide about 6-10 feet of elevation difference is ideal, but a flat site can also be regarded to work.

C. Design Requirements

1. Design Flow

The volume of water to be treated for any application is best determined by actual wastewater flow data if it is available. For new systems or where this data is not otherwise available, the designer will have to estimate the volume to be treated. A per capita flow rate design value of 100 gpcd should be used, in accordance with current Iowa standards. Appendix A of §567 IAC 69, provides guidelines for average daily design flows for various types of commercial establishments.

2. Peak Hourly Flow Rate

The peak hourly flow rate must also be considered, primarily for hydraulic, as opposed to biological treatment, considerations. The system must be able to pass the peak flow anticipated over a 60-minute period without overtopping a tank or other adverse effects. In accordance with the current Iowa wastewater design standards, a conservative peaking factor of 4.4 can be used to obtain the Peak Hourly Flow Rate.

Based on these guidelines, the design flow rates that should be considered for the example communities are show in Table 5-1.

Table 5-1
Sample Design Flow Rates for Communities of 25, 100, and 250

	Design Flow	
Population	**AWW flow, gpd**	**Peak Hour, gpm**
25	2,500	8
100	10,000	31
250	25,000	76

3. Wastewater Loadings

RSF systems are intended for the treatment of domestic wastewaters. High strength commercial or industrial wastewaters are not appropriate for treatment in a sand filter, as the filter will be susceptible to biological clogging, or will quickly become so large so as to not be cost-effective. Typical domestic wastewater strength parameters should be used to characterize the strength of the wastewater to be treated. Table 5-2 contains typical influent wastewater characteristics for influent wastewater as well as for settled wastewater (representative of a community scale septic tank) and for influent wastewater from a STEP/STEG collection system.

Alkalinity is included, as the designer of a facility with an effluent ammonia limit will need to consider whether sufficient alkalinity is present in the wastewater for nitrification to proceed.

Nitrogen is expressed as Total Kjejdahl Nitrogen (TKN), the sum of ammonia plus organic nitrogen, and should be used where possible when characterizing influent strength. This is because the much of the organic fraction will convert to ammonia in the preliminary treatment phase, and better represents the total amount of ammonia the treatment system will ultimately have to treat.

Table 5-2
Typical Influent Domestic Wastewater Strength

Parameter	Influent	Community Septic Tank Effluent	STEP/ STEG Effluent
BOD	250	250	125
TSS	250	125	125
TKN	40	40	40
Alkalinity	Varies 50-350		

These concentrations can be combined with the design flows to develop a set of design loadings for an RSF system, as presented in Table 5-3.

Table 5-3
Average Daily Influent Design Loadings, lb/day

	Population		
Parameter	**25**	**100**	**250**
BOD	5	21	52
TSS	5	21	52
TKN	1	3	8

Loading rates to the filter media itself can similarly be determined for the facility based on the type and degree of pretreatment expected, as illustrated in Table 5-2.

4. Treatment Goals

The degree of treatment required is driven by the NPDES permit issued by the IDNR. The location of the discharge in turn drives the effluent limits allowed by the permit. Individual water quality based effluent limits are determined for each facility based on the discharge volume, as well as the flow rate, temperature and pH of the receiving stream.

D. Recirculation Tank

The recirculation tank functions as a wet well for the recirculating pumps. It has been common practice to size recirculation tanks for one day's average wet weather design flow (Bounds, 1990). The recirculation tank should be long and narrow. It can be achieved by connecting multiple precast tanks together or be a single tank. The tank volume allows for accommodating short term peak flows without greatly changing the concentration of wastewater mix in the tank, and for the increased volume of recycle flow that results from rain falling directly on the filter.

As no treatment is occurring in the recirculation tank, and it is not a mechanical device, the unit process reliability requirements of the Iowa Wastewater Design Standards do not apply, and it is not necessary to provide a redundant tank.

E. Sand Filter

1. Distribution Piping Spacing

Good distribution over the filter surface is important, particularly for filters containing coarse media. The best way to ensure even distribution is to provide closely spaced distribution laterals and orifices along the lateral. Accepted practice is to place the distribution laterals on 2-foot centers. Each lateral is drilled with distribution 1/8" diameter orifices, also on 2-foot centers. Orifices are typically drilled at the 12 o'clock position and are covered with an orifice shield. The first and last orifice should be drilled in the bottom of the pipe (6 o'clock position) to help ensure drainage of the lateral to prevent freezing. This provides even distribution over the filter surface, with one orifice for every 4 ft^2 of filter area (Ball & Denn, 1997).

2. Distribution Piping Layout

Relatively equal distribution can be obtained by designing a pressurized distribution system in which there is at least 5 feet of head over the most remote orifice. The flow rate through each orifice is given by the equation:

$$Q_O = [CA(2gH)^{1/2}][60x7.48]$$

Where:
Q_O = Orifice flowrate, gpm
C = Orifice constant = 0.63 (for holes drilled in PVC pipe)
A = Cross sectional area of orifice, ft^2
g = Acceleration due to gravity = 32.2 f/s/s
H = Head, ft of water

Which simplifies to: $Q_O = 12.4\ d^2(H)^{1/2}$

Where:
d = orifice diameter, in
H = Head, ft of water

For a residual head H of 5 feet on an 1/8" diameter orifice with area A = $8.52x10^{-5}$ ft^2, the flow per orifice is 0.43 gpm. Good distribution of flow requires that the flow from all orifices be nearly the same. Pressure loss should be minimized such that the difference in flow from the first to the last orifice on a header is less than 10%. For 1/8" orifices on 2' centers, 50 feet of length (25 orifices per lateral) is the limit for a 1" PVC pipe (Molatore, 2007). For more information on the design of pressurized distribution pipe network design, or for the design of other than 1" laterals, consult Pressure Distribution Network Design by James Converse, Small Scale Waste Management Project (SSWMP): University of Wisconsin, Madison, Wisconsin. www.soils.wisc.edu/sswmp

Practice has shown that limiting the dose volume to a maximum of about 2 gallons per orifice per dose will result in the small, frequent dosing shown to maximize treatment performance. (Ball and Denn, 1997)

For each pump, determine the maximum allowable number of orifices connected to it:

$$N_o = Q_p/Q_o$$

Where:
N_o = Number of orifices
Q_p = Flowrate of pump, gpm
Q_o = Flow per orifice, gpm/orifice

For pumps of a given size, the number of orifices served per pump is show in Table 5-4.

Table 5-4
Number of 1/8" Orifices Served by Each Pump

Pump Size, gpm	Number of Orifices
10	23
20	46
30	69
40	93
50	116

As each orifice serves 4 ft^2 of filter surface area (based on a 2 foot header and orifice spacing), each pump can then be dedicated to:

$$A_z = N_o A_o$$

Where: A_z = Area of filter zone, ft^2
N_o = Number of orifices
A_o = Filter area per orifice, ft^2

The results of this equation is presented in Table 5-5, which gives the filter area served at any one time by a single pump.

Table 5-5
Filter Area Served Per Pump, ft2 for 1/8" Orifices

Pump Size, gpm	Filter Area per Pump, sf
10	93
20	185
30	278
40	370
50	463

Selection of the hydraulic loading rate and recirculation ratio are then required in order to proceed with the hydraulic design of the header system.

3. Hydraulic Loading Rate Selection

Hydraulic loading rate is the principal design parameter for sizing the surface area of the filter bed, with organic loading rate also being checked to ensure that it is below the maximum allowable. The recommended hydraulic and organic loading rates for recirculating sand filters in Iowa are as follows:

Hydraulic loading rate:	≤ 5 gpd/sf
Organic loading rate:	≤ 0.005 lb BOD/sf

The target effluent limits should be a guide to selecting design loading rates for a particular application. Very strict limits (BOD and TSS < 10 mg/l and ammonia < 5 mg/l in winter) should be addressed by using loading rates at the low end of the typical range, while less stringent limits (Eg. 30 mg/l BOD and TSS with ammonia > 15 mg/l in winter) can easily be achieved with the maximum loading rates.

Using the guidelines presented earlier, the following steps shall be used to determine the surface area of the filter bed for a cluster of 25 people using a community septic tank:

Assume

(1)	Average Daily Flow	= 2,500 gpd
(2)	Influent BOD concentration	= 250 mg/l
(3)	Post settling BOD conc.	= 250 mg/l for community septic tank
(4)	Target effluent limits	
	CBOD	= 25 mg/l monthly ave
	TSS	= 30 mg/l monthly ave
	NH_4	= 10 mg/l summer
	NH_4	= 15 mg/l winter

Select

(5)	Hydraulic loading rate (HLR)	= 5 gpd/sf
(6)	Organic loading rate	< 0.005 lb BOD/sf

Calculate

(7) Filter surface area based on HLR

= Design flow ÷ HLR

= 2,500 gpd ÷ 5 gpd/sf

= 500 sf

(8) Check organic loading rate

= BOD loading ÷ surface area

$$= \frac{250 \text{ mg/l} \times 2{,}500 \text{ gpd} \times 8.34}{1{,}000{,}000 \times 500 \text{ sf}}$$

= 0.010 lb BOD/sf

(9) Filter surface area based on organic loading

= BOD loading ÷Organic loading rate

$$= \frac{250 \text{ mg/l} \times 2{,}500 \text{ gpd} \times 8.34}{1{,}000{,}000 \times 0.005 \text{ lb BOD/sf}}$$

= 1042 sf

(10) Recalculate HLR = Daily Flow ÷ Filter surface area
= 2,500 gpd ÷ 1042 sf
= 2.4 gpd/sf

In this example, the organic loading rate controlled the filter size. For septic tank effluent BOD concentrations of 125 mg/l or greater, the organic loading rate will require a hydraulic loading rate of less than the maximum value of 5 gpd/sf. IN all cases, but the organic and hydraulic loading rates should be less than the recommended maximum values.

Table 5-6
Organic Loading Rates Resulting From Varying BOD Concentrations and Hydraulic Loading Rates, lb BOD/sf/day

Septic Tank Effluent BOD, mg/l	HLR, gpd/sf						
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**
100	0.0017	0.0021	0.0025	0.0029	0.0033	0.0038	0.0042
125	0.0021	0.0026	0.0031	0.0036	0.0042	0.0047	0.0052
150	0.0025	0.0031	0.0038	0.0044	0.0050	0.0056	0.0063
175	0.0029	0.0036	0.0044	0.0051	0.0058	0.0066	0.0073
200	0.0033	0.0042	0.0050	0.0058	0.0067	0.0075	0.0083
225	0.0038	0.0047	0.0056	0.0066	0.0075	0.0084	0.0094

Loading rates falling within the gray shaded areas should be used only with justification to support them, such as very stringent limits or critical applications to support loadings below 0.003 lb BOD/sf/day, or where there are no ammonia limits for loading rates above 0.005 lb BOD/sf/day.

Based on this variability in design hydraulic and organic loading rates, Table 5-7 gives the sand filter sizes resulting from a variation in hydraulic loading rates in square feet for the example community sizes used in this manual.

Table 5-7
Sand Filter Surface Area (square feet) for Varying HLR

HLR, gpd/sf	Population		
	25	**100**	**250**
2	1,250	5,000	12,500
2.5	1,000	4,000	10,000
3	833	3,333	8,333
3.5	714	2,857	7,143
4	625	2,500	6,250
4.5	556	2,222	5,556
5	500	2,000	5,000

4. Filter Layout

Multiplying the area of a zone by the hydraulic loading rate yields the forward flow that can be treated by a single zone:

$$Q_z = A_z(LR)$$

Where: Q_z = Flow rate per zone, gpd
A_z = Area of filter zone, ft^2
LR = Hydraulic loading rate, gpm/ft^2

Dividing this rate into the total daily design flow rate will yield the number of zones required. Each zone should be served by two pumps that alternate, providing redundancy in the event of a pump failure. By using sequencing valves, each pair of pumps can serve several zones.

The overall minimum size of the filter is driven by the daily design flow and the hydraulic loading rate:

$$A_f = Q_d/LR$$

Where: A_f = Area of filter, sf
Q_d = Design flow, gpd
LR = Hydraulic loading rate, gpm/ft^2

There is some flexibility as to the layout of the filter. Site constraints may dictate the length to width ratio. The optimum layout of the overall filter is a square, as liner and perimeter wall material are minimized.

Once the filter dimensions have been selected, the number of laterals and individual zones can be determined. Allowing for 1 foot of clearance from the terminal orifice on each end, the lateral length (distance between first and last orifice) will be 2 feet less than the width of the filter.

$$N_L = A_f \div 2(d_l + Os)$$

Where:
- N_L = Number of laterals
- Af = Area of filter, sf
- d_l = Length of lateral, ft
- Os = Orifice spacing, ft

Regardless of loading rate, the number of zones per filter is driven by the pump flow rate, which in turn determines how many orifices can be pressurized by a single pump. From Table 4-6, select a pump size and the corresponding number of orifices served by a single pump, N_o, and calculate number of zones needed by dividing the number of laterals by the number of orifices per pump:

$$\mathbf{N_z = N_L \div N_o}$$

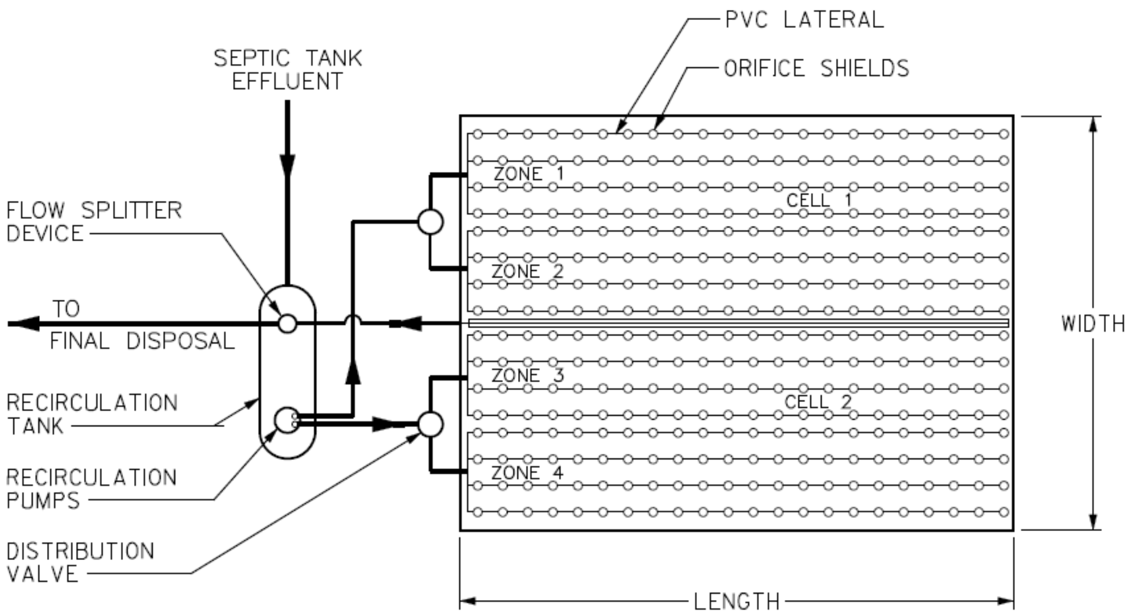
Where:
- N_Z = Number of filter zones
- N_L = Number of laterals
- N_o = Number of orifices per pump

The designer must next determine how many zones can be served by a single pump. Multiple zones can be served from a single pump through the use of an automatic distribution valve. An automatic distribution valve is mechanically actuated by the stopping and starting of a pump cycle. It sequentially rotates and selects the next zone to receive flow from the pump. Using an automatic distribution valve, a single pump can serve up to a maximum of six zones.

The unit process reliability requirements for every installation will be driven by the stream classification and determined in the current Iowa Wastewater Design Standards. Where the reliability requirement is 50% for organics and ammonia, the total required filter area should be divided into at least two filter cells. A higher reliability requirement can be met by dividing the filter into additional cells, with each filter cell fed by a dedicated pump or set of pumps. If there are multiple zones in each cell, each cell will also have its own distribution valve. Where hydraulic reliability requirements are 75%, a filter must be able to receive 75% of the design flow with one unit out of service and maintain compliance with the effluent suspended solids limit. Due to the filtering hat occurs in RMF systems, a system of two cells will be able to comply with the 75% reliability requirement.

Once the filter dimensions and configuration of cells and zones has been determined, the designer can produce a layout of the tanks and filter on the site. A generalized layout of the filter system showing the laterals, cells and zones is shown in Figure 5-1.

Figure 5-1
General RSF System Layout



To provide separation between individual cells, a physical barrier is required that will allow a cell to be rested or rehabilitated without influence from an adjacent operating cell. A 2"x12" treated board running between the cells for the length of the filter, with the top of the board installed in the media level with the top of the distribution lateral, will suffice to provide this separation.

To minimize head loss and piping cost, the recirculation tank should be located near the sand filter. Once the piping can be laid out along with relative elevations, hydraulic calculations can be run to make final pump selection. This guidance assumes the designer has a working knowledge of hydraulics and will not go into the details of pipe and pump sizing. It will only be pointed out that design is an iterative process, and that initial assumptions on pump flow rate shall be verified, and the design adjusted as needed for variations in the actual pump flow rate that may result from the selection process. Assistance from the process equipment suppliers may be helpful in determining the layout of a recirculating sand filter.

5. Media Selection

a) Fine filtering media

Media for recirculating filters should be clean, hard, durable particles free from dirt or organic matter. The media shall conform to the following requirements:

Effective Size (D_{10})	= 1.5 – 2.5 mm
Uniformity Coefficient (UC)	= 2.5 or less
Maximum particle size	< 3/8 inch
Hardness	> 3 Mohs
Solubility	< 5% in acid for particles smaller than No.8 sieve

Grain size distribution

Sieve Size	Passing by Weight
3/8”	100%
No. 4	70-100%
No. 8	5-78%
No. 16	0-4%
No. 40	0-1%

b) Coarse underdrain media

Filter underdrain pipes shall be bedded in a coarse media to allow water to flow to the underdrain collection pipes. The coarse underdrain media shall be of sufficient size to support the overlying fine filtration media without migration of the fine media into the coarse media. The coarse media shall be clean, hard durable stone. The coarse underdrain media shall be a total of 8 inches in depth, and shall consist of two layers with the following properties:

Lower 6 inches

Grain size distribution - ASTM C-33 No. 67

Sieve Size	Passing by Weight
1”	100%
3/ 4”	90-100%
3/8”	20-55%
No. 4	0-10%

Upper 2 inches
Grain size distribution - ASTM C-33 No. 8

Sieve Size	Passing by Weight
1/2"	100%
3/ 8"	50-100%
No. 4	6-84%
No. 8	0-24%
No. 16	0-1%

6. Filter Bed Depth

The depth of the fine filtering media shall be 24" at a minimum. More may be allowable but has not been demonstrated to be of significant benefit. A coarser material shall be used below the fine filtering media. The lower media (ASTM C-33, size No. 67) depth shall be great enough to cover the under drain pipes, so where 4" diameter under drains are used, a lower coarse media depth of 6-inches is sufficient. An intermediate layer (ASTM C-33, size No. 8) of 2-inches shall be between the coarse and fine media to prevent migration of fine media into the lower layer.

The resulting media requirements for filters with 6" of coarse under drain media, 2" of and 24" of fine media are shown in Table 5-8 as the HLR for the filters varies from 2 to 5 gpd/sf.

Table 5-8
Media Volumes in Cubic Yards for Fine Media

	AWW Flow, gpd								
	2,500			10,000			25,000		
HLR, gpd/sf	ASTM 67	ASTM 8	Fine	ASTM 67	ASTM 8	Fine	ASTM 67	ASTM 8	Fine
2	23	8	93	93	31	370	231	77	926
2.5	19	6	74	74	25	296	185	62	741
3	15	5	62	62	21	247	154	52	617
3.5	13	4	53	53	18	212	132	44	529
4	12	4	46	46	15	185	116	39	463
4.5	10	3	41	41	14	165	103	34	412
5	9	3	37	37	12	148	93	31	370

7. Filter Under drain

The job of the filter under drain is to convey water from the bottom of the filter to the flow splitter structure or device and to provide a conduit for air flow into the bottom of the filter and up through the media. Thus, the under drain must be open to the atmosphere at some point. It is recommended that a sampling sump be located just outside the filter(s) having a drop of at least 4-inches in the drain across the sump. This will provide a convenient location for obtaining samples to monitor filter performance and will provide the needed air inlet point. The under drain must be sufficiently sized so that water does not back up into the filter media, which can lead to anaerobic conditions. The openings in the under drain pipe must be large enough to allow water to enter freely, while preventing the under drain bedding media from blocking the openings or entering the pipe.

Some references recommend that filter under drain be spaced no more than 10 feet apart across the entire bottom of the filter (Rhode Island, 1999). Experience, however, has shown that a properly sized single drain pipe sized to convey the peak flow rate that is anticipated through the filter media, including rainfall, and that is bedded in a clean stone media as specified herein, will adequately drain the filter. The single drain pipe should be placed in the center of the filter and run the entire length. The bottom of the filter and liner should be sloped at 0.5-1% to pull water to the drain from the perimeter. Alternatively, for a flat bottomed filter, a 4" under drain spaced 20 feet on centers is commonly used.

Slotted PVC or HDPE pipe is typically used, with ¼" wide slots on 4" centers. The end of the under drain opposite the splitter structure should be directed up with two 45-degree bends and be terminated above the filter surface to provide access for cleaning.

8. <u>Monitoring Tubes</u>

Four monitoring tubes should be placed in each filter zone to two different depths, two each to the bottom of the filter and to the top of the treatment media. One each of the shallow and deep monitoring tubes shall be placed on each end of each filter zone.

Monitoring tubes that extend to the liner should be perforated only in the bottom 12 inches. These will allow the operator to determine if there is any unexpected depth of ponding on the liner. This, coupled with observations in the cleanouts at the end of drain lines, will allow determination of whether there is clogging of the under drain pipe openings.

A second set of monitoring tubes bottoming at the surface of the treatment media should be placed with at least two at each end of the filter. This will allow quick determination of any ponding starting to develop on the surface of the treatment media.

Providing the operator with the ability to make these observations can help him or her avert a catastrophic clogging incident by being able to see a problem starting and determine the cause before it becomes a major problem.

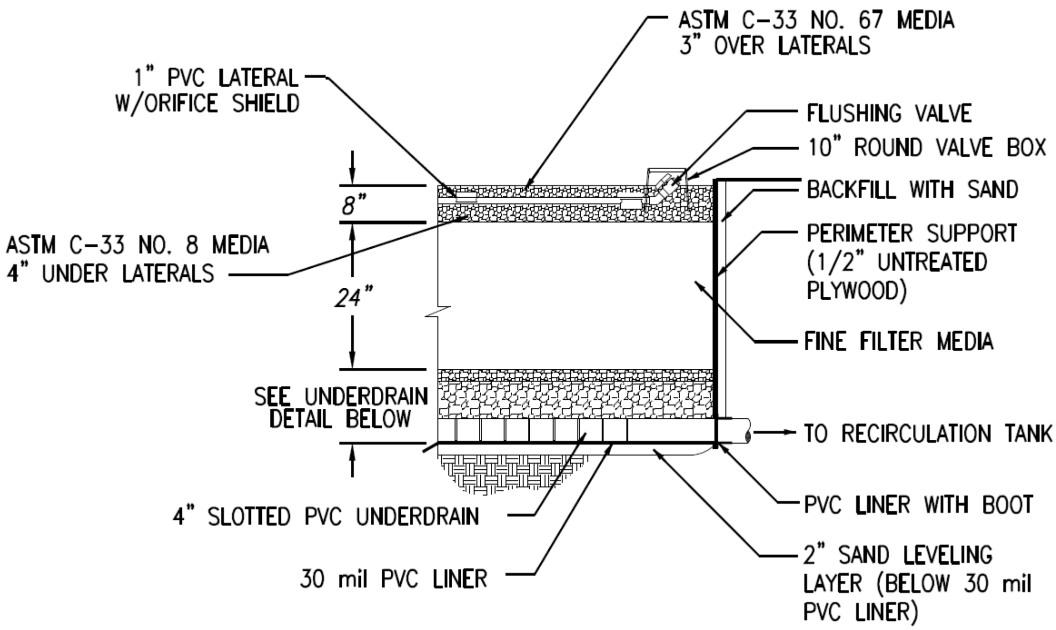
9. Filter Liner

An impervious liner is required to contain the filtrate and allow it to be collected for recirculation. 30 mil PVC is often used for this purpose. The subgrade should be prepared for liner installation by requiring the removal of all rocks, roots, and organic material. If the native soils are not sufficient, a 2-3” layer of clean sand should be placed prior to liner installation.

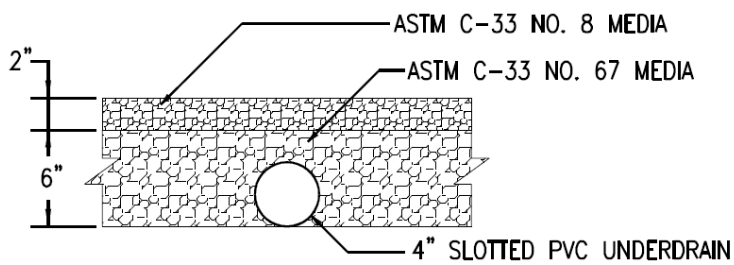
The excavation sidewalls are often ½” to ¾” untreated plywood or OSB. The liner is lapped over the sidewalls at least 18 inches and the space between the excavation and OSB is backfilled with sand to stabilize the sidewall and secure the liner.

A cross section of the completed filter, showing the liner, under drain, layers of media, and distribution piping is shown in Figure 5-2.

Figure 5-2
Cross Section of Recirculating Sand Filter



FILTER CROSS SECTION



UNDERDRAIN DETAIL

10. Recirculation Control

As discussed in Section 2, there are a variety of means to splitting flow between the recirculation tank and the effluent outfall. For the range of systems covered by this guidance, it is recommended that the recirculating splitter valve shown in Figure 2-5 be used. It provides the ability to control the recirculation rate to between 1 and 4 and provides for low-flow recycle to the recirculation tank without the use of an actuated valve.

F. Dosing Pump Controls

For systems serving populations up to 250 people, a relatively simple control system based on timers and floats should be sufficient. More sophisticated control systems can be applied, but the complexity will increase while the reliability will likely decrease.

In general, dosing cycles are initiated by timers based on the anticipated daily flow. High and low level floats provide overrides for when the flow rate is greater than or less than the anticipated flow. If the timed dosing cycles are not sufficient to keep up with the rate of influent, the water level in the recirculation tank will rise until the high level float is actuated. The high level float will initiate an additional dosing cycle or cause the control to simply switch to a shorter time off interval to help draw down the level in the recirculation tank. Once the level returns to normal, the control will resume operating at its normal setting.

A low level float can prevent the pumps from drawing the level down too far and running the pumps dry. In the event that not enough water is being returned from the filter and the timer initiates a cycle, the low level float shall cause the pumps to shut down, and not restart until there is sufficient water available to initiate a dosing cycle. The control panel shall be able to record a low- and high-level events so that the operator will know that the timer settings may need adjustment.

Initial timer settings based on the design flow of the system are done based on limiting the volume per orifice to 2 gallons per dose. As discussed earlier, this is setting will provide for frequent, short cycles which have been demonstrated to provide a higher degree of treatment.

The number of pumps that are required for each dose is based on the total flow to be pumped, including recirculation.

$$N_{pc} = \frac{Q_{rsf}}{(1440 \text{ min/day} \times Q_{po})}$$

Where:
- N_{pc} = Calcuated number of Pumps per Dose
- Q_{rsf} = Total pumped flow, gpd
  = Q_d*(R+1)
- Q_d = Daily design flow, gpd
- Q_{po} = Operating pump discharge rate, gpm

The calculated number of pumps N_{pc} is then rounded up to the nearest whole number to get the actual number of pumps N_{pa}. When more than one pump is required, it means that two or more pumps are activated at the initiation of each dosing cycle. A delay timer in the control circuit can be used so that both pumps do not start at exactly the same time, which would increase amp draw and wire size requirements. The timing sequence is then calculated as follows:

$$T_{\%} = \frac{Q_{rsf} \times 100\%}{(N_{pa} \times Q_{po} \times 1{,}440)}$$

Where:
- $T_{\%}$ = Daily Run Time, %
- Q_{rsf} = Total pumped flow, gpd
- N_{pa} = Actual number of Pumps per Dose
- Q_{po} = Operating pump discharge rate, gpm

The initial timer settings are then based on the time needed to dose a given volume per orifice per dose. Assuming an initial target dose volume of 2 gallons per orifice per dosing cycle, calculate the total volume the pumps must deliver based on the final layout of the filter.

$$T_d = \frac{N_l \times N_o \times V_d}{(N_{pa} \times Q_{po})}$$

Where:
- T_d = Pump Run Time per Dose, min
- N_l = Number of laterals per zone
- N_o = Number of orifices per lateral
- V_d = Volume per orifice per dose, gal
- N_{pa} = Actual number of Pumps per Dose
- Q_{po} = Operating pump discharge rate, gpm

The initial timer settings in minutes are then determined by Td and $T_{\%}$ as follows:

Run time per dose = T_d
Total time per dosing cycle $T_c = T_d/T_{\%}$
Rest time per dose $T_r = T_c - T_d$

The total number of cycles per day is then 1,440 min/day ÷ Tc. The number of cycles should be at least 96 cycles per day to ensure frequent dosing, but should not require more pumps starts than recommended by the pump manufacturer. Franklin Electric Motors, a manufacturer of motors used in many submersible pumps, recommends fewer than 300 starts per day for less than 1-hp pump motors. The pump control panel must then alternately energize the pump or pumps needed for each cycle.

If the number of pump cycles is greater than 300, the designer will need to increase the dose volume per orifice in order to get a longer cycle time. Should the frequency fall below 96, the designer should likewise reduce the dose volume per orifice such that additional cycles are needed.

G. Design Examples

This section will go through the preceding design process using numbers for a population of 250 people. After each parameter is developed in the example, a set of design parameters for populations of 25 and 100 people will also be provided for comparison.

RSF Design Example				**Units**
1 Calculate average daily design flow	**250**	**100**	**25**	**People**
	25,000	10,000	2,500	gpd
2 Size Pretreatment Unit				
Design detention time	2 -3			days
Total tank volume	50-75,000	20-30,000	5-10,000	gallons
Select number of tanks	2-3	2-3	2	tanks
Select volume of each tank	25,000	10,000	2,500-5000	gallons
3 Select Effluent Filters				
Design flow for effluent filters (each)	8,400	8,400	4,200	gpd
Select filter openings of 1/8"				
Number of screens needed	3	2	1	
Place effluent filters at the outlet of the second tank				
4 Size Recirculation Tank				
Size tank for one day's flow				
Minimum Tank Size	25,000	10,000	2,500	gallons

5 Sand Filter Design	**25,000**	**10,000**	**2,500**	gpd
Select desired hydraulic loading rate		5		gpd/sf
Calculate filter size needed	5,000	2,000	500	sf
Check organic loading rate				
Assume BOD removal in septic tank		10%		
BOD to filter		250		mg/l
Calculate organic loading	47	19	5	lb BOD/day
Calculate organic loading rate		0.01		lb BOD/day/sf
Limit organic loading rate to 0.005		0.005		lb BOD/day/sf
Recalculate filter size based on organic loading	10,425	4,170	1,042	sf
Recalculate effective hydraulic loading rate		2.4		gpd/sf
Filter dimensions (L x W)	50 x 210	48 x 90	22 x 48	feet
Actual filter area provided	10,500	4,320	1,056	sf

Select filter dimensions (length and width) that best fit the site and that meet the minimum filter area needed. Note that for 1” diameter PVC distribution laterals, the maximum filter length is 50 feet (48 feet of lateral plus 1 foot on either end). Filter length should be an even number to best accommodate orifice spacing. Initially set width to a multiple of 4 feet to work best for dividing filter area into cells and zones. This process may require iterations of filter dimensions and pump size to determine a geometry that can be uniformly divided into cells and zones.

5 Sand Filter Design (cont'd)	**25,000**	**10,000**	**2,500**	gpd
Select lateral spacing		2		feet
Select orifice spacing		2		feet
Select orifice diameter		0.125		inch
Select design head pressure		5		feet
Number of laterals per filter	105	45	24	
		Should be 1/2 of width		
Length of lateral	48	46	20	feet
Orifices per lateral	24	23	10	
Calculate flow per orifice		0.43		gpm
Select Nominal Pump Size	40	30	20	gpm
Calculate No. of orifices per pump	93	69	46	
Calculate number of laterals per zone	3.86	3.02	4.63	
Round down to nearest whole number	3	3	4	
Calculate number of zones in filter	35	15	6	
Minimum number of filter cells		2		
Maximum number of Zones per cell		6		
Select No. of Zones and Cells				
Cells	7	3	2	
Zones per cell	5	5	3	

- The designer can now lay out the filter on the site and size pipes and pumps to match the design conditions;
- Because the length of the laterals is not greater than 48 feet, a lateral size of 1" PVC is sufficient;
- The remaining conveyance lines from the pump to the distribution valve, and the recirculation line must be sized based on the actual hydraulic conditions.

6	**Dosing Controls**	**250**	**100**	**25**	**People**
	Select maximum recirculation ratio		4		:1
	Total flow pumped per day	125,000	50,000	12,500	gpd
	Calculate the number of pumps required per dose	2.2	1.2	0.4	Pumps per dosing cycle
	Round up to the next nearest whole number	3	2	1	Pumps per dosing cycle
	Calculate percent of pump running time	72%	58%	43%	
	Determine number of orifices per zone	72	69	40	
	Select dose volume per orifice	0.5	0.5	0.5	gallons
	Determine pump run time per dose	0.3	0.6	1.0	minutes
	Determine total cycle time cycle	0.4	1.0	2.3	minutes
	Resting time	0.1	0.4	1.3	minutes
	Total dosing cycles per day	3472	1449	625	
	No. of Doses per day, each zone	99	97	104	
	No. of Pump Cycles per day, each pump	248	242	156	

VI. OPERATION AND MAINTENANCE INFORMATION

A. Operational Concerns

1. Filter Saturation and Ponding

The organism population within an RSF multiplies to balance the organic loading rate. When food is not coming in, the process of endogenous respiration takes over in which organisms consume each other, a sort of survival of the fittest phenomenon. This process keeps the filter from building a large organic content of biological cells. If the system is too heavily loaded, biological cells and biodegradation byproducts accumulate, and the pores of the sand system may become filled with organic matter. This then begins to slow the flow through process and eventually can lead to a filter with ponding on the surface. Therefore, it is necessary to balance the application rate with the rate at which the bugs can decompose the applied material and keep the development of a large bacterial cell mass from accumulating.

As water starts to collect in ponds on the surface, it also spreads out over the surface of the media. While initially only a small area underneath the orifice of a distribution lateral will receive water, ponding will increase the amount of media utilized in the treatment process. So while ponding is a preliminary indication of clogging, isolated ponding need not cause alarm, as it also allows for better media utilization.

However, a properly operating RSF should never pond completely. There should always be sufficient area that is not covered by biomat that the water recedes within at most a minute or two. If this is not the case, the filter is not operating correctly and the nature and reason for excessive biomat needs to be investigated before anaerobic conditions set in. An anaerobic filter may also foster the growth of worms or other macrophytes. A filter cell should be taken off line and rested before it is completely ponded. Once the surface has dried, it can be raked or tilled and placed back into service.

2. Freezing

Water that is kept moving is less likely to freeze. In a coarse media filter (D_{10}>1.5 mm), water will percolate through the media fast enough to prevent freezing, even in the Upper Midwest. Some ice "shields" will form above the surface of an open filter and distribution headers, but water should continue to flow underneath the ice all winter. This ice provides insulation from the cold ambient air. Algae growth is inhibited by the cold temperature, making winter operation relatively low maintenance.

Freezing is a concern with fine or clogged media. In subfreezing ambient temperatures, ponded wastewater may cool to the point where freezing occurs. Once a filter surface freezes, it effectively prevents its use for treatment until it thaws. Allowing an entire bed to freeze would leave a community without any secondary treatment at all.

One such example of a filter that experienced freezing occurred in the Village of Knapp, WI. The original system employed a media with an effective size (D_{10}) of 1.12 mm and a Uniformity Coefficient of 1.4. The filter built up a layer of slime and was continually saturated, which led to the entire filter bed freezing up in winter. A number of other factors contributed to the failure of this system, including:

- Poor distribution of wastewater
  - Gravity flow (not pressurized)
  - Large spacing of headers (approximately 14 feet)
- High strength wastewater
  - Influent BOD of 400 mg/l
  - High organic loading rate
    - 0.0062 lb/day/ft^2 at design flow
    - 0.004 actual loading rate
- Small septic tank (24 hour HRT)

One of the keys to preventing a frozen filter is to transfer flow onto a rested and raked filter cell in the fall months while the temperature is still warm enough to establish nitrification. Frequent, smaller doses to minimize ponding will also help to avoid freezing.

3. <u>Pumps and Electrical</u>

Pumping systems should be provided with a redundant pump for each zone to provide good reliability. The dosing pumps must be able to meet the worst-case instantaneous flow rate requirement with one unit out of service. Pumps are generally controlled by timers, floats, or some type of electronic level sensor.

4. <u>Odors</u>

a) <u>Pretreatment Units</u>

Odors can originate in the septic tank, which is vented to the atmosphere. While generally not a nuisance to neighbors, carbon canisters can be installed on the vent piping to further reduce odors.

b) *Media Bed*

Odors in the sand or gravel filter media are uncommon, and are an indicator that something is wrong. As an aerobic system, the products of metabolism are chiefly carbon dioxide and water, which are odorless. Odors are produced under anaerobic conditions. They are an indicator that the dissolved oxygen in the filter is being depleted and that BOD and ammonia removal are likely being impacted.

B. Maintenance Issues

1. Staffing

An RSF facility is typically operated and maintained by a single person. Depending on the frequency of visits and sampling requirements, the average amount of time spent monitoring an RSF facility ranges from about 2-7 hours per week. For larger facilities, daily visits might be needed, or required according to the permit. On non-sampling days, operators report that the daily checkup should take about 15 minutes. On sampling days, one hour is typically needed to collect samples and prepare them for delivery to the lab. Weeks during which periodic maintenance of equipment or of the filter itself is performed will require additional hours.

For small facilities equipped with an alarm dialer, daily visits may not be necessary, but the operator is remained to consistently maintain compliance with the applicable permits.

2. Sampling

Surface discharging facilities regulated under the NPDES permit program will have influent and effluent sampling requirements spelled out in the permit. These may range from once per month to as many as three times per week for parameters such as

- BOD,
- TSS,
- ammonia-nitrogen,
- fecal coliforms,
- pH, and
- dissolved oxygen.

For facilities without a permit required sampling schedule, periodic sampling for operational control are still recommended. Such sampling can provide a benchmark level of performance for a system, allowing the operator to observe trends in performance and address a potential issue before it is allowed to progress to failure of the system. Table 6-1 contains a minimum recommended sampling protocol for RSF systems that will provide the operator with sufficient information on the performance of their system.

Table 6-1
Minimum Recommended Sampling Location and Frequency

Parameter	Location: Influent	Location: Septic Tank Effluent	Location: Effluent
BOD	Monthly	Monthly	Monthly
TSS	Monthly	Monthly	Monthly
Ammonia	Monthly		Monthly
Temperature	Monthly		Monthly
D.O.			Monthly
pH			Monthly

3. Septic Tank Effluent Screen Cleaning Intervals

It is recommended that cleaning of the effluent screens be done more frequently than recommended by the manufacturer at first, until the operator has a sense of how quickly they are prone to clogging. An initial cleaning interval of every two weeks is suggested. If clogging does not appear to be a problem after two weeks, the operator can gradually begin to extend the interval. The operator should look for signs of surcharging such as a high waterline on the wall and debris on top of the screen and overflow pipes.

Screens should be sprayed off with high-pressure water over the head end of the septic tank. Water may be from a well, or from a sump pump drawing effluent from the splitter structure. If water is not available on site, the operator may place a spare cartridge into service, and haul the dirty screen off site for cleaning. Note that the filter will likely retain some water and the operator will need a way to transport the screen in a manner that minimizes spillage from the screen. Examples include wrapping up in a plastic tarp or placing the screen in a bucket.

4. Sludge Removal

Solids will accumulate in the settling tank, particularly the first cell of a multi-chambered tank. A properly sized tank will allow for solids to accumulate for 1-5 years. During this period, the sludge will compact and anaerobically break down. An operator should monitor the level of sludge accumulation annually with a Sludge Judge or similar sampling device. A rule of thumb would be to arrange for sludge removal when sludge occupies half of the volume of the settling tank. The quality of the sludge is equivalent to a Class B sludge under 40 CFR Part 503, the federal sludge quality regulations, and can generally be land applied. Sludge should be handled by a hauler licensed under IA 68.

The scum layer that may form at the surface should also be monitored. The bottom of the scum layer should not be allowed to get close than 6 inches from the inlet to the outlet baffle or effluent screen housing. In some cases, the thickness of the scum layer may be the factor that triggers tank cleanout.

Sludge pumping contractors will typically charge by the gallon, so reducing the volume of wastewater above the sludge can save the Owner money. Decanting the liquid portion from one cell into another can be accomplished by lowering a submersible pump into the tank cell and suspending it above the surface of the sludge blanket. Another advantage of using multiple small tanks instead of one large tank is that the pump-out operation will be much more effective if the pumper truck(s) can completely remove the contents of a given tank in a few minutes so that new flow does not add to the total volume of material to be pumped. Pumping a large tank can take days. During the pumping period, continuing inflow adds to tank contents and increased the total volume to be pumped.

5. Pumps and Recirculation Tank

Water in the recirculation tank should be relatively clear and free of solids. If large solids or debris are noticed, it will be an indicator that the effluent screens have overflowed.

The pumps should be observed to operate when called to do so by the control system. The pump runtimes should be checked and recorded to verify that all pumps are receiving approximately the same amount of run time. Disparities in run times will indicate a failure to alternate or failure of a pump to run when called. Such failures should be investigated and corrected.

The manufacturer's recommendation for pump service such as oil changes, seal replacements and bearing replacements should be followed. At least one spare pump shall be maintained in reserve in the event a pump needs to be removed for service for more than one day

Pump control floats in the recirculation tank should be suspended freely in the tank. The floats should be free of debris or grease build-up, and should be sprayed off as needed.

6. Distribution Piping

The automatic distribution valve(s) should be observed to be sequencing the dosing of each filter cell and zone. Using a shovel, the operator should expose laterals at various locations on the filter surface to verify that the area under the laterals is wet. If the media under some laterals is dry, it indicates that clogging of the distribution lateral is likely to be occurring. Clogging is usually first evident at the most distant ends of the laterals, and indicates that the laterals need to be cleaned or flushed.

Lateral flushing can be accomplished simply using the pumped flow to scour out the lines. With the pump running to a zone, remove the end cap or open the valve on each lateral sequentially, one at a time, to flush each line clean. This takes only a few seconds for each line. Wear rubber gloves and take care not to get effluent on you. If end caps are used instead of a valve on each line, loosen all caps before starting the procedure. Surge the flow in each line by rapidly closing and reopening the valve or hold and remove the end cap over the end to stop and start the flow. This can help dislodge solids in the line or in slightly clogged orifices. Take care to be sure any squirt does not come toward you.

If flushing is not sufficient to dislodge the clogging, a more vigorous method of cleaning is required. High pressure jetting can be done while the lateral is off-line by running the nozzle of a pressure washer up and down the length of each lateral 2-3 times. Alternatively, a bottle brush attached to the end of a sewer snake can be used to ream solids out of the lateral.

7. Filter Media

Look for any obvious signs of ponding. For laterals bedded under the media, look for any wetness on the surface, which indicates localized fouling of the media. Where monitoring tubes have been installed, they should be observed for ponded water. Tubes penetrating to the surface of the treatment media should not show ponded water, except perhaps for a brief period after a dose. Where ponding remains for minutes after a dose, the dose volume is too large or fouling of the media is starting to occur. If either of these conditions are occurring, it is an early indication of media clogging, and the operator should consider taking the filter cell off line and allowing it to rest.

The operator should also observe the biological activity in the filter. Look for any tan to light gray gelatinous deposits around the orifices, orifice caps and stones immediately around these zones. If present, this is an aerobic floc starting to build and is an indication that the applied effluent is too aerobic. Reduce pump run time to reduce recirculation ratio. Also look for black deposits. If present, this is an indication of anaerobic overload conditions. It may mean that the organic loading rate is too high or that the recirculation ratio is too low. Sometimes some black deposits may build during cold weather and dissipate when it warms up, even if the organic loading and the recirculation ratio are both within the proper range. As long as the blackness goes away seasonally, it is not a major problem (Loudon, 2003).

8. Vegetation Control

All growth should be kept off the surface of the filter. Where influent is surface applied, this will require regular, frequent weed removal in the summer. If done frequently, the maintenance provider will deal only with small weeds having little root depth. Removal can be accomplished by raking the surface stones around to dislodge the developing weed roots. If weeds are allowed to get well started with significant roots into the stone, removal will require hand pulling, probably with follow-up work to prevent plants from getting reestablished from roots that do not come out with the initial attempt.

Where influent is applied below a few inches of stone, take care to keep stones arranged over the distribution lines to prevent any surface wetness. This will prevent most weeds from getting a start.

9. Record Keeping

The operator should keep a bench sheet for recording observations made on each visit. Items that should be recorded include:

- Weather observation (temperature, precipitation)
- Influent/effluent flow (if metered)
- Total pump run time, each pump
- Daily pump run time, each pump (calculated)
- Total pump starts, each pump
- Daily starts, each pump (calculated)
- Cells and zones in service
- Dissolved oxygen
    - Recirculation tank
    - Effluent
- Effluent pH
- Effluent temperature
- Other observations and comments

10. Site Control and Maintenance

The site should be made secure from passersby and particularly from vehicular traffic, including all-terrain vehicles, which may be attracted to the large, level surface of loose gravel. Woven wire or three-strand fence should be sufficient for this purpose. Locked gates should be used to allow restricted access.

Grass on the berms surrounding the filter cell should be mowed regularly, and clipping should be collected or blown away from the filter surface

11. How to Conduct a Routine Maintenance Visit

Routine maintenance may include checking septic tanks, but details of septic tank inspection are not given here. We will concentrate on the maintenance activities needed around the RSF treatment system. The following outline is intended to provide a ready reference to follow for each aspect of a maintenance visit. A field check sheet for keeping notes in the field is also provided.

RECIRCULATING MEDIA FILTER CHECK SHEET

RSF CONTROLS AND PUMPS

1. Start at the panel
2. The panel should be equipped with a pump run event counter and a total pump run time meter. Identify each. The run time meter will usually show hours, tenths, and hundredths of hours. The pump run event counter is just a counter. You may want to label each for future reference if they are not labeled.
3. Record meter readings and determine total run time and the number of pump cycles counted since the meters were last read.
4. Does the system have a timer override float function?
   a. If yes, determine the average run time per cycle [(total run time)/(no. of cycles)] and compare with timer setting. If the run time per cycle is much longer than the timer setting implies, the system is running on float (demand) basis a significant amount. It may be necessary to shorten the off time to compensate for the fact that the timer setting is not providing enough total run time per day to keep up with the flow.
5. Determine net pump run time each cycle.
   a. Best done by observation – with a helper.
   b. Uncover pipe network near input end.
   c. Have helper start pump.
   d. With stop watch, determine time to fill and pressurize.
   e. Subtract this from run time per cycle (check actual run time being delivered by timer) to determine **effective run time** per cycle.
      i. To check actual run time, set timer to short off time.
      ii. Stand at panel and listen for pump to kick on.
      iii. With stop watch, determine actual run time.
      iv. Compare with timer setting as read off timer dial.
      v. Repeat the above for 3 cycles to check repeatability and accuracy of time measurement.
      vi. Use actual measured run time in calculations.
   f. **Effective run time** per cycle is actual run time – time to fill and pressurize:
   $\mathbf{t_{eff} = t_{act} - t_{fill}}$

RECIRCULATING MEDIA FILTER CHECK SHEET
(Continued)

RECIRCULATION TANK

1. Recirculation tank water level
    a. Normal level should be between the splitter valve closed level or just above and the splitter valve open level.
    b. If significantly above or below this zone, some problems are:
        i. Low level
2. Splitter valve not allowing desired flow to return to the tank
3. Blockage in return line or filter drain
4. Filter drain blinded off
5. Pumps have just run and are set to run for too long a time
6. Tank leaks
7. Filter is partially frozen
    a. High level
8. Recent heavy rain
9. Groundwater infiltration
10. Float valve not closing or other flow splitter not working correctly
11. High raw wastewater inflow rate, short term
12. Pumps not set to run enough for the incoming flow
13. Flow to the filter is severely restricted
14. Recirculation splitter valve – float type without pipe overflow returns
    a. Float ball in place and free, not stuck between vertical rails, etc.
    b. Float ball properly inflated
        i. Check by using an L-shaped paddle to raise and feel ball
        ii. It should not be possible to push ball out between vertical rail guides
15. Recirculation splitter valve – float type with overflow returns
    a. Check ball condition as above
    b. Run pumps on manual for a longer than normal dose
        i. Allow return flow to build up (3-5 minutes after pumps turned on)
        ii. Check to be sure all return lines are flowing after return flow has built up
        iii. Float valve should close
        iv. Check flow rate into final dose tank to be sure float by-pass is working correctly
16. Check scum and sludge in recirculation tank
    a. Normal conditions may vary by type of wastewater input, time of inspection including seasonal effects, and location along the tank.
    b. Scum on the top of the tank may be only floating clumps or may be a continuous mat, which is unusual.

RECIRCULATING MEDIA FILTER CHECK SHEET
(Continued)

RECIRCULATION TANK

c. Scum thickness should not exceed a few inches. If scum is consistently more than this in 2-3 observations, it is time to have the contents of the recirculation tank removed to an approved septage disposal site.
d. Sludge is usually light and fluffy. Be very slow, deliberate and careful in making a measurement to avoid stirring up the sludge.
e. Sludge thickness should not exceed about 15 inches in depth anywhere in the tank.

17. Recirculation tank contents (i.e. blend of wastewater and return water)
    a. pH throughout the tank should be near neutral (pH of 7)
    b. Dissolved Oxygen content of the tank will vary. It should be higher, 4-5mg/L or more, near where water is returning from the sand filter. The incoming sewage should be less than 1 mg/L. The blended mix in the tank that is pumped to the sand filter should be less than 2 mg/L.
    c. Temperature of the tank near the pumps feeding the RSF should be greater than 40 F.
    d. Odor of the tank should be faint septic near the incoming end to musty hear the filtered water return end.

SURFACE OBSERVATIONS AT THE RSF BED

1. Weed growth
    a. All growth should be kept off the surface of the filter. Where effluent is surface applied, this will require regular, frequent weed removal in the summer. If done frequently, the maintenance provider will deal only with small weeds having little root depth. Removal can be accomplished by raking the surface stones around to dislodge the developing weed roots. If weeds are allowed to get well started with significant roots into the stone, removal will require hand pulling, probably with follow-up work to prevent plants from getting reestablished from roots that do not come out with the initial attempt.
    b. Where effluent is applied within a few inches of stone, take care to keep stones arranged over the distribution lines to prevent any surface wetness. This will prevent most weeds from getting a start.
2. Check monitoring tubes, if present
    a. Monitoring tubes to the treatment media surface
        i. Monitoring tubes penetrating the RSF to the surface of the treatment media should not show ponded water, except toward the end of a dose application and possibly for a few seconds thereafter, if the monitoring tube is near an orifice in the distribution pipe.

RECIRCULATING MEDIA FILTER CHECK SHEET
(Continued)

ii. If ponding remains visible on the media minutes after a dose, either the dose volume is much too large (pumps running too long) or the surface is becoming clogged and is in need of renovation.

b. Monitoring tubes to the bottom of the filter

i. A few inches of ponding is normal at the bottom of the filter. Where the drain system consists of chambers, each with an outlet, the ponding should not exceed 2-3 inches, and that will be due to irregularities in the surface under the liner. Where the drain system is slotted drain pipe embedded in stone, up to 4-8 inches of ponding may be present, especially right after a dose application. The ponding depth should be consistent, varying only due to dose timing and possibly precipitation.

3. Check appearance of several orifices under the orifice shields.
   a. Removal of some stone around distribution pipes may be necessary.
   b. Remove orifice caps
   c. Look for any clogging in the orifices. If orifices are pointed down, it may be necessary to use a mirror to get a good look them.
   d. Look for any tan to light gray gelatinous deposits around the orifices, orifice caps and stone immediately around these zones. If present, this is an aerobic floc starting to build and is an indication that the applied effluent is too aerobic. Reduce pump run time to reduce recirculation ratio. See Calculations section.
   e. Look for black deposits. If present, this is an indication of anaerobic overload conditions. It may mean that the organic loading rate is too high or that the recirculation ratio is too low. Sometimes some black deposits may build during cold weather and dissipate when it warms up, even if the organic loading and the recirculation ratio are both within the proper range. As long as the blackness goes away seasonally, it is not a major problem.
   f. Replace orifice shields and stone over distribution pipes.
4. Flush the distribution laterals
   a. With the pump running to a zone, remove the end cap or open the valve on each lateral sequentially, one at a time, to flush each line clean. This takes only a few seconds for each line. Wear rubber gloves and take care not to get effluent on you. If end caps are used instead of a valve on each line, loosen all caps before starting the procedure.
   b. Surge the flow in each line by rapidly closing and reopening the valve or hold and remove the end cap over the end to stop and start the flow. This can help dislodge solids in the line or in slightly clogged orifices. Take care to be sure any squirt does not come toward you.
5. Check pressure in each zone after flushing

RECIRCULATING MEDIA FILTER CHECK SHEET
(Continued)

a. Use a clear stand pipe or piece of tubing on a support to check the pressure at the end of a line in each application pipe zone. The height to which the water rises in a tube is the head or pressure in the pipe, measured in feet of water. (One psi pressure is equivalent to 2.31 feet of water head).

b. Compare head measured with what is supposed to be in the system and with the last measurement.

c. If the head increases more than a few inches, it is an indication that orifices are becoming plugged. If the head is approaching 20% more than it should be, the lines must be cleaned to clean the orifices.

d. If head has decreased since the last check, it is an indication of a leak in the system, a partial blockage in the line feeding the system or a problem with a pump, which is unlikely.

e. Fluctuating pressure would be an indication that the flow to the suction side of the pump is limited. If the pump is in a pump vault, the screen ahead of the pump is in need of cleaning or the screen around the pump intake is clogged, if so equipped.

6. Orifice cleaning procedures you may use:
   a. Bottle brush on a snake
      i. Obtain a stiff bristle bottle brush that is just larger in outside diameter than the inside diameter of the distribution laterals
      ii. Securely fasten the brush to an electric wire pulling snake longer than the length of the laterals
      iii. With the pump turned off, push the bottle brush through each lateral, moving it back and forth as you go.
      iv. Clogged orifices are most likely to be at dead-end of the pipe where flow is lowest and where any solids in the pipe get pushed each time the pump turns on, so be most vigorous when the brush is near that end.
   b. High pressure jetting
      i. Obtain a high pressure jetter with a small hose and jetting nozzle that will fit inside the laterals.
      ii. With the pump off, run the jetter down each lateral 2-3 times.
   c. Apply suction to the laterals
      i. Make an attachment so that you can fasten a vacuum pump to one or more laterals at a time. A septic tank pumper truck works well for this as it has a powerful vacuum pump.
      ii. Close the valve at the pump leading to a distribution zone pipe network.

RECIRCULATING MEDIA FILTER CHECK SHEET
(Continued)

iii. Build up a vacuum and suddenly open the vacuum to the line(s). This will suck out anything that has entered an orifice. It may be necessary to cycle the vacuum on and off several times to for each set of pipe(s) to which it is attached.

7. Recheck pressures as described above to be sure that orifices have been successfully cleaned. System pressure should be restored to proper level.
8. Use the actual head on the system to determine the proper pump run time each cycle. The water application through each orifice should be 1 – 2 gallons per dose.
9. Flow per orifice
   a. The flow through an orifice depends on the orifice size and the head or pressure in the pipe at the location of an orifice. Flow in gallons per minute can be calculated using the following:
      i. $q = 12.38\ d^2\ h^{0.5}$, where d is in inches and h is in feet
      ii. For example, if the orifice size is 1/8 inch (0.125") and the head is 4 feet, the flow is $q = 12.38\ (.125 \times .125)\ (4^{0.5)})$ or $q = 12.38 \times 0.0156 \times 2 = 0.3869$ gpm.
10. Total daily flow through a zone of the RSF, V_{zone}
   a. Multiply the effective total daily run time by the flow per orifice times the number of orifices:
      i. $V_{zone} = n \times t_{eff} \times q \times N_{orif}$ where n is the number of pump cycles per day, t_{eff} is the effective run time each cycle, q is the flow per orifice and N_{orif} is the number of orifices in a zone.
11. Recirculation ratio
   a. Determine the total daily flow to the sand filter
      i. Determine the flow to each zone using the method above
      ii. Add up the flow to all zones
      iii. That is the total flow to the sand filter, V_{total}
   b. Recirculation Ratio = (total daily flow to RSF) / (daily average forward flow)

VII. COST ESTIMATES

A. Sources of Information and Reliability

Due to the extreme variably of local markets for labor and materials, it is not possible to estimate universally the cost of construction and operation of recirculating media filters. Cost differentials are significant across local geographies and economies. Therefore the reader of this manual is advised to consult local markets for specific data.

B. Capital Costs

A major determinant in the overall cost of a project is its size. The larger the project, the greater the benefit from economies of scale. Therefore the reader of this manual is advised to consult with knowledgeable individuals for specifics relating to costs of construction for a particular project.

1. Capital Cost estimating Spreadsheet

The next page details a typical cost estimating spreadsheet for estimating overall capital costs for a recirculating media filter treatment system. The spreadsheet identifies major components of the proposed construction and allocates units for each component. Upon completion of a standard design, actual units of installation may be inputted into the spreadsheet. Costs per unit must be obtained from local sources due to the aforementioned extreme variability in local markets.

A spreadsheet showing the major capital cost line items and unit costs that could be anticipated is shown in Table 7-1.

Table 7-1
Recirculating Sand Capital Costs

Capital Costs

Item	Quantity	Units	Unit Cost	Total Cost
Land		Acres		
Site Work		cy		
Site Electrical (3 Phase)				
Flow Meters		each		
Samplers		each		
Septic Tanks		each		
Recirculation Tank		each		
Splitter/Valve Vault		each		
RSF System				
Earthwork		cy		
Filter Liner		sy		
Underdrain Piping		lf		
Coarse Filter Media		cy		
Fine Filter Media		cy		
Distribution Piping and Valves		lot		
Pumps and Controls		lot		
Control Building (incl. Elec and HVAC)		sf		
Fencing		lf		
Yard Piping		lf		
Electrical (10%)				
Contractor OH&P (20%)				
Subtotal				
Capital Contingencies (25%)				
Subtotal				
Engineering (20%)				
Legal and Administative (5%)				
Total Estimated Capital Cost				

Table 7-2
Recirculating Sand Filter O&M Costs

Operation and Maintenance Costs	Qty	Units	Unit Cost	Annual Cost
Labor		hours/yr		
Electric Power		kWh		
Supplies		lot		
Maintenance and Repair				
Laboratory Testing				
Sludge Disposal		gallons		
Annual O & M Cost				

C. Annualized Costs

1. Operations and Maintenance Cost Estimating Spreadsheet

A spreadsheet showing the major operations and maintenance cost line items and unit costs that could be anticipated is shown in Table 7-2.

2. Significant Assumptions

a) Sludge Removal

Bi-annual sludge removal should be assumed, with an annual amount built into the budget equal to one-half the cost. Accumulation of sludge to one-day's average forward flow would be a conservative assumption.

b) Power

Power costs will vary across the state and in time. A current estimate of the cost of power per kWh should be obtained to estimate annual power costs for the dosing pumps. Power cost for the dosing pumps can be done by multiplying the total number of pumps times the average running time, and converting horsepower into kilowatts as per the following formula:

$$\text{Annual Power Cost} = (N_p)(T_{\%})(24 \text{ hours})(HP)(0.75)(\$/kWh)(365)$$

Where:
- Np = Number of pumps
- T% = Percent daily run time
- HP = Horsepower of each pump
- $/kWh = Cost of power per kWh

c) Maintenance

(1) Equipment Maintenance and Replacement

An annual set-aside for equipment replacement should be built into the budget. The amount set aside should be based on the original cost of the equipment, and prorated out over the expected design life of the equipment.

(2) Site Maintenance

The annual cost should account for site maintenance such as grass mowing and snow removal.

d) Labor

The estimated cost for labor should be based on the total compensation for the operating staff, including any benefits, plus any administrative salaries for meetings, billing, etc. The estimated hours needed should consider the monitoring and sampling requirements of the particular facility, and include provision for periodic maintenance such as vegetation removal, flushing of laterals and regular pump maintenance.

e) Sampling and Analysis

The cost for a facility's sampling and analysis program will vary from one facility to another based on the permit. Larger facilities with surface water discharges will require more frequent and comprehensive sampling than a small facility with a subsurface discharge. The cost should be based on the total number of samples expected in a year, and include the cost of analysis by a certified laboratory, plus the costs of sample delivery.

APPENDIX A

PRIMARY TREATMENT UNITS

APPENDIX B

PERFORMANCE DATA

Table B-1 Effluent BOD Performance from Wisconsin RSFs

	Influent	Effluent					
	Mean	Mean	90% C.I. - Low	90% C.I. - High	Mean, May-Oct	Mean, Nov-Apr	% Removal
Montfort	104	3.5	3.4	3.7	2.4	4.5	97
Fairwater	184	6.2	5.3	7.2	3.8	8.8	97
Packwaukee	86	8.3	7.4	9.1	10.8	6.2	90
Roxbury	189	11.0	10.5	11.6	8.0	13.8	94
Footville	177	8.1	7.6	8.6	5.5	10.1	95
Merrimac	99	3.3	2.8	3.7	3.4	2.7	97
Avoca	243	6.8	6.6	7.1	6.3	7.4	97
Gratiot	133	10.3	9.2	11.5	6.1	13.0	92
Oakdale	184	9.7	8.2	11.3	6.0	13.5	95
Highland	130	5.5	5.3	5.7	4.3	6.7	96
Barneveld	226	9.1	8.6	9.7	6.0	12.2	96
Dons Mobile	173	4.2	3.4	5.0	2.8	5.9	98
Selwood Farms	259	7.0	6.0	7.9	7.4	6.5	97
Arlington	246	5.7	5.4	6.0	5.4	6.1	98
Peninsula	253	11.4	10.3	12.5	11.5	7.6	95
Yuba	138	3.0	1.7	4.3	2.0	3.3	98
Comfort Suites	212	2.0	1.9	2.1	2.0	2.0	99
Knapp (pre-9/03)	323	25.6	21.8	29.4	19.5	32.6	92
Knapp (post-9/03)	403	17.3	15.8	18.7	11.0	23.4	96
Ixonia	86	7					92
Average	**192**	**8.3**	**7.1**	**8.7**	**6.2**	**9.3**	**96**

Table B-2 Effluent TSS Performance from Wisconsin RSF

	Influent	Effluent					
	Mean	Mean	90% C.I. - Low	90% C.I. - High	Mean, May-Oct	Mean, Nov-Apr	% Removal
Montfort	97	3.0	2.9	3.1	2.8	3.2	97
Fairwater	198	5.2	4.6	5.7	4.5	5.9	97
Packwaukee	96	7.1	6.1	8.0	10.2	4.4	93
Roxbury	159	6.0	5.6	6.3	5.0	6.9	96
Footville	161	4.8	4.5	5.1	4.5	5.1	97
Merrimac	68	4.3	3.0	5.7	4.7	3.0	94
Avoca	251	7.4	6.9	7.9	7.8	7.0	97
Gratiot	131	7.1	6.3	7.9	5.4	8.1	95
Oakdale	256	6.2	5.3	7.1	4.2	8.1	98
Highland	104	4.2	4.0	4.5	3.9	4.6	96
Barneveld	132	6.6	6.2	6.9	5.7	7.4	95
Dons Mobile	261	2.3	2.2	2.4	2.1	2.5	99
Selwood Farms	202	5.1	4.5	5.7	5.5	4.8	97
Arlington	253	2.7	2.6	2.8	2.6	2.8	99
Peninsula	181	7.4	6.6	8.1	7.5	4.3	96
Yuba	54	3.2	2.5	4.0	4.0	3.1	94
Comfort Suites	86	2.7	0.5	4.9	4.0	0.0	97
Knapp 1	191	10.1	8.4	11.8	10.0	10.3	95
Knapp 2	415	7.8	7.1	8.5	7.4	8.2	98
Ixonia	107	6	0	0	0	0	94
Average	**170**	**5.5**	**4.5**	**5.8**	**5.1**	**5.0**	**96**

Table B-3 Effluent NH3 Performance from Wisconsin RSFs							
		Effluent					
	Influent Mean	**Mean**	**90% C.I. - Low**	**90% C.I. - High**	**Mean, May-Oct**	**Mean, Nov-Apr**	**% Removal**
Montfort [1]	21	1.8	1.6	1.9	0.3	3.0	92
Fairwater [1]	37	4.2	0.6	7.8	4.3	5.9	89
Packwaukee [1]	17	7.4	7.4	9.1	0.0	7.4	57
Roxbury [1]	38	6.9	6.3	7.4	2.0	11.3	82
Footville [1]	35	4.1	3.6	4.7	0.9	8.2	88
Merrimac	25	0.5	0.3	0.7	0.4	0.5	98
Avoca [1]	49	7.6	6.8	8.2	1.8	12.9	84
Highland [1]	26	5.3	4.9	5.6	1.5	9.0	80
Barneveld [1]	45	1.5	1.3	1.8	0.5	2.9	97
Dons Mobile	12	1.7	0.3	3.1	0.3	3.6	86
Comfort Suites	53	0.8	0.3	1.3	0.7	1.0	99
Knapp (post 9/03) [1]	81	5.7	4.7	6.6	5.0	17.3	93
Merrimac [2]	34	1.0	0.7	1.3	1.0	1.1	97
Dons Mobile [2]	34	2.9	1.1	4.7	1.4	5.0	91
Selwood Farms [2]	45	7.3	5.4	9.3	5.3	8.8	84
Yuba [2]	57	1.5	N/A	N/A	1.5	N/A	97
Average	**38**	**3.8**	**3.0**	**4.9**	**1.7**	**6.5**	**88**

1 – Influent ammonia estimated based on influent BOD at ratio of 1.0:0.2 BOD:NH_3

2 – Nitrogen reported as TKN

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