GREENBOOK

Division of Surface Water

Design Standards:

Wastewater Treatment Plants
& Collection Systems
≤ 100,000 gpd

2013 Version

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Acknowledgement

"Sewage: Collection, Treatment, & Disposal Where Public Sewers Are Not Available", commonly known as the Greenbook, is a guidance document used for the design of wastewater treatment plants with an average daily design flow up to 100,000 gpd. The Ohio Department of Health originally developed the guidance document in 1957 with Ohio EPA making revisions to the Greenbook in: 1974, 1980, 1983, 1988, and 1993. This latest revision was prepared under the direction and coordination of the Ohio EPA Division of Surface Water's PTI Oversight Committee, an internal Agency work group.

> *The PTI Oversight Committee is a group of permit staff and management from the Division of Surface Water and the Division of Environmental & Financial Assistance. The committee was tasked with addressing permitting issues and providing direction for the permitting program, which included providing guidance on various programmatic areas.*

The Greenbook addresses provisions to be considered for the treatment of domestic wastewater, or wastewaters similar in strength and composition to domestic wastewater. Its emphasis is to outline Ohio EPA's recommendations for the extended aeration activated sludge treatment process. The Greenbook will help:

A) Ensure that the design of wastewater treatment systems comply with public health, water quality, and regulatory/rule objectives of the State of Ohio,

B) Establish a basis for the design and minimum requirements set forth by the Division of Surface Water for the review of wastewater treatment system detail plans and specifications,

C) Assist design engineers in the preparation of plans, specifications, reports, and other data; and

D) Guide Division of Surface Water district office staff in their determination of whether a PTI and/or NPDES permit for a wastewater treatment system should be approved.

The design of a wastewater treatment plant should incorporate treatment processes that will perform effectively with minimal attention. It should be noted that processes requiring sophisticated controls beyond conventional treatment will require a higher level of operation and maintenance skill level and may not be able to consistently meet NPDES permit limits when applied to small scale wastewater treatment. This does, however, not preclude the design and operation of other wastewater treatment processes, nor is the intent of this guidance document to stifle new technology.

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Guidance Document Disclaimer

The Greenbook provides guidance for the benefit of Ohio EPA permit staff, design engineers, owners, and operators of wastewater treatment plants. The intent of the Greenbook is to address requirements that will lead to approvable plans and specifications for wastewater treatment plants. Although this guidance document is not regulation, the technical expertise and regulatory requirements infused into the guidance document will provide consistency with the design and review across the State of Ohio.

The Ohio EPA Division of Surface Water is willing to discuss alternative methods of complying with applicable statutes, regulations, or requirements. Ohio EPA may approve other designs that deviate from the Greenbook requirements as long as they are adequately justified. It is requested that these deviations are discussed and agreed upon with Ohio EPA prior to the submittal of the permit application to avoid unnecessary or wasteful expenditures of resources.

Helpful Tips:

Throughout the guidance document, these helpful tips are provided to give unique, experience-based perspectives with respect to wastewater treatment plant design. The design should meet the minimum guidelines but also consider the end users to create an efficient and effective treatment system. These tips are intended to provide an operational perspective to the engineer for incorporation into the design of the treatment system.

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Chapter 1. Administration

1.1 Introduction

The Division of Surface Water's mission is to ensure compliance with the federal Clean Water Acts' goal of maintaining and protecting the chemical, physical, and biological integrity of all Ohio waterways. One way the Division of Surface Water tries to ensure compliance is through the permitting of wastewater treatment plants.

Permits are issued to ensure wastewater treatment plants are properly designed to treat the collected wastewater before discharging it back into Ohio waterways without negatively impacting human health, aquatic life, and the environment. When designing a wastewater treatment plant, the following rules and regulations are applicable and must be accounted for/incorporated into the design to obtain approval:

1.11 Ohio Revised Code: Chapter 6111

Section 6111.44 of the Ohio Revised Code (ORC) requires that all plans for sanitary sewers and sewage treatment devices for all buildings and places other than a private residence or dwelling (one, two, or three family) be submitted to and approved by the Ohio Environmental Protection Agency (Ohio EPA) *before* construction is begun.

1.12 Ohio Administrative Code: Chapter 3745

The following table outlines the rules of the Ohio Administrative Code (OAC) that contain wastewater treatment requirements and have been adopted by the Ohio EPA in accordance with state administrative procedures:

Table 1.1: Ohio EPA Rules

Chapter 3745-01	*Water Quality Standards*
Chapter 3745-07	*Operator Certification*
Chapter 3745-33	*Ohio NPDES Permits*
Chapter 3745-42	*Permits to Install*

Copies of all Ohio EPA Division of Surface Water (DSW) rules may be obtained at http://www.epa.ohio.gov/dsw .

The following table outlines rules of the OAC that contain wastewater treatment requirements administered by the Ohio EPA in conjunction with the Ohio Department of Health (ODH):

Table 1.2: Ohio Department of Health Rules	
Chapter 3701-25	*Recreation Vehicle Parks, Recreational Camps, & Combined Park-Camps*
Chapter 3701-27	*Mobile Home Parks*
Chapter 3701-31	*Swimming Pools*
Chapter 3701-35	*Marinas*

Copies of ODH individual rules, OAC 3701, are available at http://www.odh.ohio.gov .

Other state statutes and rules of interest to users of the Greenbook include the following:

Table 1.3: Other Rules & Regulations	
ORC 3703.01-09 OAC 4101-2-51	*Submittal of plans for plumbing to the ODH or proper local authorities*
ORC 3732.01-08 OAC 3701-21	*Submittal of plans for food service operations & equipment to the local health departments*
ORC 3745.11	*Covers fees which must be paid for PTI & NPDES permits*

1.2 Antidegradation

Request for new discharge sources to waters of the state will require an antidegradation review. This review determines whether a discharge is feasible and/or what level of treatment may be necessary. The review may include consideration of the following:

A) Location of nearest sanitary sewer system,

B) Suitability of soils for soil based treatment[1],

C) Area for land application[2],

D) Adequate receiving stream available (e.g. stream quality, size, etc.),

E) Site topography; and

F) Economics and costs associated with various treatment/disposal alternatives.

[1] *See Ohio EPA Interim Onsite Sewage Treatment System Guidance Document for requirements*
[2] *See OAC 3745-42-13 for land application requirements*

After all the no-discharge alternatives have been analyzed and it is determined that a discharge is the most feasible option, it is acceptable to proceed with the design of the wastewater treatment plant. However, if the results of this no-discharge analysis indicate that the best option is not to discharge to the stream, additional review[3] and analysis should be conducted for land application, onsite sewage treatment systems (OSTS), or connection to centralized sanitary sewer options.

1.3 State or Regional (Areawide) 208 Plans

A 208 plan refers to Section 208 of the Clean Water Act wherein federal Construction Grant funding for wastewater infrastructure improvements was contingent upon doing facility planning with a regional perspective. An effective 208 plan helped ensure that water quality problems and limitations are known and that politically separate communities in urban metropolitan areas built cost effective wastewater infrastructure with a minimum of excess or overlapping service capacity. The underlying requirements regarding 208 plans remain in the law despite the complete phase out of the Construction Grant program in the 1980s.

Currently, the submission of plans for wastewater infrastructure does not require any updated 208 planning effort. However, all wastewater infrastructures for Publically Owned Treatment Works (POTW) must be reviewed in light of the existing approved 208 plan. All permit applications are required to conform to any pre-existing 208/201 plans for the area *(see PTI Form A, question 12)*. The Director may not approve a permit if it conflicts with an approved 208 plan [ORC 6111.03(J)(2)(b)].

For additional information regarding Ohio EPA Division of Surface Water's Water Quality Management plan section, please visit the following website:

http://epa.ohio.gov/dsw/mgmtplans/208index.aspx

1.4 Permitting Procedure

Before procuring the site, if central sanitary sewer services are not available, it is strongly encouraged that the appropriate district office of the Ohio EPA be contacted to discuss the proposed new discharge. The district office can provide:

A) Guidance on the suitability of a wastewater treatment plant discharge,

B) Potential National Pollutant Discharge Elimination System (NPDES) permit effluent limits; and

C) Permitting requirements.

[3] ***Contact the appropriate district office and/or central office for details prior to submitting plans.***

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Before developing and submitting detailed plans to the Ohio EPA for a new discharge, an adequate investigation shall have been conducted to determine if:

A) The antidegradation review deems the discharge feasible and acceptable,

B) All applicable rules/regulations are able to be met; and

C) The conceptual design will be able to meet applicable NPDES permit effluent limits.

It is preferred that an NPDES permit application is submitted prior to the submittal of the PTI. However, it is acceptable to submit the NPDES permit with the PTI application.

1.41 NPDES Permit

The following NPDES permit application criteria shall be followed for any new, expansion, or major modification project:

A) NPDES permit limits must meet both treatment technology and water quality-based requirements of the Clean Water Act (CWA) and its regulations,

B) Facilities shall be designed to meet Best Available Demonstrated Control Technology (BADCT) requirements spelled out in OAC 3745-1-05. Otherwise, basic treatment technology requirements for publicly-owned treatment works (POTW) are those in 40 CFR Part 133[4],

C) New facilities may need to do modeling to determine whether BADCT or other treatment technology limits meet water quality standards (WQS),

D) Required permit applications:

1) For new or existing POTWs – Forms 1, 2A and the Antidegradation Addendum (AA),

2) For new or existing WWTPs not POTWs – Forms 1, 2E and the AA,

3) For new discharges from plants not POTWs that combine sewage and process water treatment – Forms 1, 2D and the AA,

4) For existing discharges from plants not POTWs that combine sewage and process water treatment – Forms 1, 2C and the AA.

[4] ***These rules are often used as Best Professional Judgment (BPJ) in setting treatment technology limits for existing sewage plant discharges at non-POTW facilities***

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E) Application testing requirements *(new dischargers must provide estimated values):*

1) Form 2A – None,

2) Form 2C/2D – Varies by industry category – See application instructions,

3) Form 2E – BOD_5, TSS, oil & grease, ammonia-nitrogen, pH, temperature and residual chlorine (if used as a disinfectant); and

F) For additional requirements, refer to OAC 3745-33.

1.42 PTI Submittal

The following shall be completed and submitted with the plans before a PTI can be approved:

A) A complete application for a permit to install in accordance with OAC 3745-42. Copies of all Ohio EPA DSW application forms may be obtained at http://www.epa.ohio.gov/dsw,

B) Appropriate fees in accordance with ORC 3745.11,

C) Four (4) sets of detailed plans with at least two sets including the original signature of the professional engineer on the title page,

D) Letter of transmittal or owner's signature on the plans to indicate evidence of the owner's approval,

E) If the finished project will be owned by anyone other than the applicant for the permit to install (i.e., city, county, sewer districts, etc.), a signed contract or similar indication of approval of plans and acceptance of the project by the final owner shall be submitted with the plans,

G) Engineering report including design data, specifications, and other pertinent information *(design data may be all included on the detailed plans forgoing the submittal of a separate engineering report)*; and

H) For additional requirements, refer to OAC 3745-42.

1.43 Supporting Data

Some projects may require additional information to be submitted with the normal permit application. Supporting data shall be submitted with the plans on only those sections of the specifications pertaining to wastewater treatment & disposal. Specifications, diagrams, etc. of equipment/processes not related to wastewater treatment should not be included with the plans. Where data sheets are prescribed, they should be completed in full detail. Materials, particularly, should be listed by their standards designation as they would in a specification.

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Chapter 2. Collection System

2.1 Introduction

Collection system design requires particular attention to several factors, including average flow rates, peak flow rates, industrial and/or commercial flow rates, potential inflow and infiltration (I/I), site specific issues (i.e. topography, bedrock depths), and the potential for future development.

Roof drainage, foundation drainage, or other clean water connection to the sanitary sewer shall be prohibited by enforcement of legally adopted rules by the authority regulating the use of the sanitary sewers or Ohio EPA. This statement should be prominently placed on all detailed plans

2.2 Size & Slope

Sanitary sewers shall be designed on a peak hourly design flow basis using a peak flow factor of three and one-third (3.33) times the total calculated average design flow. Pumps and force mains should be designed for the peak flow that discharges into the pump station.

Figure 2-1: Peak Hourly Design Flow Calculation

$$\text{Peak Factor} = \frac{3.33 \times 24 \text{ (hours)}}{\text{Run-off period (hours)}}$$

Peak hourly design flow (gpd) = peak factor x average daily flow

Table 2-1: Typical Run-Off Periods and Peak Factors

Facility Type	Run-Off Period[5]	Peak Factor(s)
Factories	Length of work day	Variable
Subdivisions or Municipalities > 250 homes	24 hours	3.33
Subdivisions or Municipalities < 250 homes	16 hours	5.0
Hospitals, Nursing and Rest Homes	16 hours	5.0
Camps	16 hours	5.0
Public schools	8 hours	10
Restaurants	8 to 12 hours	10-6.66
Boarding schools	16 hours	5.0
Mobile home parks	16 hours	5.0
Apartments, Motels	16 hours	5.0

[5] *Flow equalization facilities are required for plants greater than or equal to 2,500 gpd with runoff time period of 16 hours or less. (Other run-off periods must be documented.)*

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All gravity sanitary sewers should be designed to give a mean velocity of at least two feet per second at peak design flow based on Manning's formula with n = 0.013, or in accordance with Table 2-2.

Figure 2-2: Manning's Formula

$$Q = (k_n / n) * A * R^{2/3} * S^{1/2}$$

Q	=	cross-sectional average flow (ft/s, m/s)	R	=	hydraulic radius (ft, m)
k_n	=	1.486 for English units and k_n = 1.0 for SI units	S	=	slope of pipe (ft/ft, m/m)
n	=	*Manning coefficient of roughness*	A	=	cross section area of pipe (ft^2)

Table 2-2: Minimum Slope for Sewer Design

Sewer Size	Minimum Slope (feet/100 feet)	Approximate Capacity at Minimum Slope (gpd)	Approximate Capacity at Minimum Slope (cfs)
6"	0.60	271,000	0.42
8"	0.40	520,000	0.80
10"	0.28	750,000	1.16
12"	0.22	1,100,000	1.70

Sewers on 20 percent slope or greater and at all changes in alignment locations shall be anchored with concrete anchors spaced as follows:

Table 2-3: Severely Sloped Sewers[6]

Center to Center	*Grade (percent)*
Not over 36 ft	20-35
Not over 24 ft	35-50
Not over 16 ft	50+

In general, the minimum size of sanitary sewers shall be eight (8) inches to accommodate future growth. However, six (6) inch sanitary sewers may be used on a case by case basis, provided their hydraulic capacity is not exceeded because of short run-off periods *(high peak flows).*

When a smaller sewer discharges into a larger one through a manhole, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing this result is to place the 0.8 depth point of both sewers at the same elevation.

[6] ***When velocities greater than 15 feet per second are expected, provisions should be made to protect against displacement and erosion of the pipe.***

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2.3 Water Line Separation

Sanitary sewers should be laid with at least a 10 foot horizontal separation from any water line. Sewers may be laid closer than 10 feet to a water line if it is laid in a separate trench and elevation to the crown of the sewer is at least 18 inches below the bottom of the water line. If it is impossible to maintain the 18-inch vertical separation when the sewer is laid closer than 10 feet to the water line, the sanitary sewer shall be encased in concrete or constructed of water line type materials which will withstand a 150 psi water pressure test *(must be thrust blocked at the ends to prevent separation of the pipe at the push on gaskets).*

Whenever a gravity sanitary sewer and water line must cross, the sewer shall be laid at such an elevation that the crown of the sewer is at least 18 inches below the bottom of the water line. If it is absolutely impossible to maintain the 18-inch vertical separation, the sanitary sewer shall be encased in concrete or constructed of water line type materials which will withstand a 150 psi pressure test. These requirements will extend for a distance of 10 feet, measured perpendicular, on both sides of the water line.

Whenever a sewage force main and water line must cross, the sewage force main shall be laid at such an elevation that the crown of the sewage force main is at least 18 inches below the bottom of the water line.

2.4 Manholes

A) Manholes shall be installed at the end of each line that is 150 feet or greater in length, at all changes in grade, size, alignment, and at all pipe intersections. Manholes shall also be installed at distances not greater than 400 feet. For manholes with proposed spacing greater than 400 feet, the collection system owner must demonstrate that adequate sewer cleaning equipment is available. Cleanouts may be installed at the ends of lines which are less than 150 feet in length and in lieu of manhole(s) on wastewater treatment plant effluent discharge lines.

B) Drop manholes shall be used when the sewer invert entering the manhole is two (2) feet or greater above the manhole invert. When the difference in elevation between the incoming sewer and the manhole invert is less than two (2) feet, the manhole invert should be filleted to prevent solids deposition.

C) The minimum internal diameter of manholes shall be 48 inches. The flow channel through manholes should be made to conform in shape, slope, and smoothness to that of the sewers. A bench shall be provided on each side of the manhole channel. The bench should be located one (1) pipe diameter above the invert and slope no less than ½" per foot to the channel.[7] "Dog house" manholes are prohibited.

[7] ***Manholes shall conform to applicable ASTM standards or equivalent for all sanitary sewer projects.***

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D) Manhole covers can be adjusted to grade by the use of no more than 12 inches of precast concrete adjusting collars. Watertight and bolted manhole covers should be used in street locations. In other areas, the manhole casting should be adjusted so that the top is slightly above grade to prevent the entrance of surface water. Watertight manhole covers shall be used in all locations which may be flooded by runoff or flooding. The use of brick chimney shims is prohibited.

2.5 Fats, Oils, & Grease (FOG)

Facilities that have the potential to generate significant volumes of Fats, Oils, & Greases (FOG), (i.e. food service operations), shall install an external FOG interceptor. At a minimum, a 1,000 gallon FOG interceptor shall be designed using US EPA Method #1 or #2 for interceptors.[8] These interceptors shall be installed on dedicated kitchen lines that service food prep and dishwashing activities only. The location of the FOG interceptor shall be located in such a manner as to allow the influent to cool sufficiently to allow the FOG interceptor to function as designed.

2.6 Pipe Material & Installation

Type, depth, and existing soil conditions shall be considered in selecting the type, strength, and stiffness of pipe. Installation details shall be in compliance with the selected specification, except bedding and backfill for rigid pipe shall comply with class A, B, or C in ASTM C-12 and bedding and backfill for flexible pipe shall comply with class IA or IB, II, or III in ASTM D-2321. All bedding material shall have 100 percent passing a 3/4" sieve.[9]

Additionally, the Ohio Department of Transportation (ODOT) specifications 603.06 & 603.10 may be used in lieu of the ASTM standards for bedding and backfill for rigid and flexible pipe. Approvable[10] gravity sewer pipe, force main materials, and other materials may be found at:

http://www.epa.ohio.gov/dsw/pti.aspx

The lateral connections should comply with standards shown in this policy and should be made of the same material as the street sewer whenever possible to minimize infiltration from the connection between the street main and house lateral. When joint material and/or dimensions are not compatible, a commercial adaptor shall be provided.

Construction of sewers in stream beds should be avoided when feasible. If necessary, stream crossings shall be perpendicular to the crossing location flow line. Collection systems cannot be placed under streams except at crossings. The preferred sewer installation method when crossing a stream is directional drilling or jack & boring. All stream crossing projects shall meet all applicable stream antidegradation rules and Clean Water Act section 401/404 requirements. The applicant shall discuss any proposed stream crossing with the appropriate Ohio EPA District Office and Army Corps of Engineers Office.

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[8] ***See A.9 in the appendix for Method #1 & #2 formulas***
[9] ***Deviation from the ASTM standards for bedding and backfill should have proper justification***
[10] ***Alternative specifications may be approved. Contact appropriate district office to discuss.***

Chapter 3. Pump Stations

3.1 Introduction

Pump stations provide a means for transporting sewage when topography and geology is not favorable for gravity sewer construction. Pump stations can be sited in the collection system or at the wastewater treatment plant. When located at the wastewater treatment plant, influent pump stations are incorporated into flow equalization basins *(see Chapter 5: Flow Equalization)*.

However, the use of a pump station can be avoided by placing the sewers at a shallow depth, or by locating the treatment plant at a lower elevation. When shallow sewers are used, the wastewater generated on the floors of buildings, which are located below grade, may need to be pumped.

Generic pump station schematics are located in Appendix A-2 through A-4.

3.2 General Requirements

All pump stations should include: [11]

A) At least two pumps,

B) Each pump capable of passing three (3) inch solids each,

C) Each pump capable of pumping the design peak flow (*supported by a pump curve for the selected pumps)*,

> ***Helpful Tip #1*:**
> For pump stations that do not discharge to flow equalization basins, care must be given to the effect of high pumping rates on the clarifier design. A pump station that pumps at a greater rate than the acceptable clarifier surface overflow rate could wash out the clarifier every time the pump runs. Unless the pump is controlled by a variable frequency drive, the clarifier would need to be sized so that the peak pumping rate and the acceptable clarifier surface overflow rate is maintained.

D) Independent controls for each pump,

[11] ***No overflows or bypasses will be permitted.***

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Helpful Tip #2:
The influent sewer tributary to the wet well should enter the wet well above the "pump on" float setting to avoid surcharging the influent sewer during normal operation.

E) Automatic alternators; and

F) Security fencing and access hatches with locks, when necessary.

3.3 Alarms

A visual and/or audible alarm system shall be provided. The alarm will be activated when:

A) High water levels are detected in the wet well,

B) The standby pump is activated; or

C) Loss of power to the pump station or pump failure occurs.

Once activated, the alarm will remain on until manually deactivated by a qualified operator. The alarm shall have a light atop the control box and have a battery back-up in case of power failure. An audible alarm or automatic dialer system may also be included along with a visual alarm system. If the pump station is remotely located, an automatic dialer system would be appropriate. [12]

3.4 Alternate Power Sources

Alternate sources of power, standby pumping, or generating equipment shall be provided as follows for the appropriate average daily design flow:

Table 3-1: Alternate Source of Power Options

≤ 20,000 gpd	Hook-up available
20,001 – 40,000 gpd	Portable generator available
> 40,000 gpd	Permanent power or second feed

If an onsite generator is not an option being utilized, an emergency plan to secure a portable generator/gasoline pump should be developed. This emergency plan should be kept onsite in case of emergency. Pump stations should also have emergency bypass pumping capabilities.

[12] *See A.1, in the Appendix for appropriate isolation distances*

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3.5 Pumping Rates

Pump stations on small wastewater treatment plants produce hydraulic surges which may affect the treatment effectiveness at the wastewater treatment plant. To reduce this surging effect, the pumped influent to the wastewater treatment plant should produce a peak surface overflow rate not to exceed 500-800 gpd/ft^2 on the clarifier.

Pumping rates that will exceed the peak surface overflow rates on the clarifier shall be returned to the wet well or a flow equalization basin.

Helpful Tip #3:

Acceptable peak surface overflow rates for clarifiers will depend on the type of clarifier. For hopper type clarifiers in package plants, the peak overflow rate would be on the low end of the range (500 – 600 gpd/ft^2). For clarifiers with mechanical sludge collectors, acceptable peak overflow rates would be on the higher end of the range (600 – 800 gpd/ft^2).

Example 3.1: Pump Station Influence Calculation

Assumptions:

ADF	=	25,000 gpd
Run-off Time	=	16 hours
Pump Station	=	2 pumps @ 100 gpm each
Clarifier Size	=	2 hopper clarifiers @ 8’ x 14’ each

Step 1: Determine peak flow

Peaking Flow = Peaking Factor * ADF
= (5.0)*(25,000 gpd)
= 125,000 gpd peak flow

Step 2: Determine surface overflow rate

SOR = peak overflow rate / clarifier surface area
= 125,000 gpd / 2*(8 ft.)*(14 ft.)
= 558 gpd/ft^2 *(acceptable SOR)*

Step 3: Determine pump station pumping rate

Q_{pump} = 100 gpm * 1,440 min/day
= 144,000 gpd

Step 4: Determine pumping impact on clarifier

SOR_{pump} = Q_{pump} / clarifier surface area
= 144,000 gpd / 224 ft^2
= 643 gpd/ft^2

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∴ *the pump flow rate exceeds the acceptable clarifier SOR. Therefore either a flow equalization basin should be considered, pump station pumps downsized, or the clarifier size (surface area) be increased.*

Explanation: Upstream processes like pump stations can have unanticipated consequences on downstream processes like clarifiers. By checking how pump station pumping rates can influence clarifiers, potential design based problems can be avoided.

3.6 Acceptable Types of Pump Stations

All sewage pump stations shall be equipped with rails, hoists, and other means for lifting the pump, motor, or pump motor units above ground without entering the wet well. [13]

3.61 Wet Well/Dry Well

Wet Well/Dry Well pumps stations are normally located in the collection system or serve as the influent pump station. General requirements are:

A) A sump pump equipped with dual check valves shall be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well,

B) Where the dry well is below the ground surface, mechanical ventilation is required,

C) Covered wet wells shall have provisions for air displacement to the atmosphere, such as an inverted "j" tube; and

D) Suitable shutoff valves shall be placed on the suction line. Shutoff and check valves shall be placed on the discharge line of dry well pumps.

3.62 Submersible Pumps

Submersible pump stations are the most common type of pump station used in smaller installations. Grinder pumps are considered acceptable for submersible pump station applications. These pump stations shall have:

A) A pump and motor unit that can be removed and installed from above ground without dewatering or having an operator enter the wet well,

B) Valves installed in an enclosure outside the wet well. Valves shall be located in a separate valve pit. Valve pits may be dewatered to a wet well through a check valve without using a manually operated valve or liquid seal. Check valves that are integral to the pump need not be located in a separate valve pit provided that the valve can be removed from the wet well without entering the wet well,

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[13] ***See A.2-A.4 in the appendix for diagrams of acceptable pump stations***

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C) Electrical controls installed in an NEMA 4X enclosure outside the wet well. All electrical equipment and installation shall meet all current NEC regulations. It is strongly recommended that the control panel is sealed to prevent gas intrusion. Wet wells should be vented such as with an inverted "J" tube.

D) Access opening, hand crane, or crane base should be provided, sized, and located to allow easy removal of pumps and equipment,

E) Explosion-proof submersible pumps used for ~~raw~~ sewage,

F) Stainless steel slide rails and connection (linkage) components be installed; and

G) Pumps attached with stainless steel chains or cables for maintenance and/or replacement.

Helpful Tip #4:
Over time, the blades within grinder pumps become dull and lose their effectiveness to properly grind solids. Special consideration should be made for annual/quarterly maintenance of grinder pump and checking with the manufacturer regarding required blade sharpening or replacement of cutters.

3.63 Suction Lift Pump

Suction lift pumps can be used as an alternative to submersible pump stations. General requirements are:

A) Suction-lift pumps must be automatically self-priming,

B) Suction piping should not exceed the size of the pump suction and shall not exceed 25 feet in total length,

C) The combined total of dynamic suction-lift at the "pump off" elevation and required net positive suction head at design operating conditions shall not exceed 22 feet; and

D) The pump equipment compartment shall be above grade or offset and shall be effectively isolated from the wet well to prevent a hazardous and corrosive sewer atmosphere from entering the equipment compartment.

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Chapter 4. General WWTP Design

4.1 Introduction

All domestic wastewater or wastewaters similar to domestic wastewater in strength and composition must be treated prior to discharge into waters of the state. In general, the degree of treatment will be determined by the location, volume of flow, existing water quality, and use designation of the receiving stream. The site specific conditions and the existing and future service area must be considered in the selection of the treatment system configuration. This guidance document is intended for domestic wastewater treatment. Treatment of a waste that is not characteristic of domestic wastewater or is inherently toxic to microorganisms may not be properly treated utilizing this design guidance.

Chapter 4 introduces basic general design information that would be applicable to any extended aeration wastewater treatment plant. Detailed design information for specific treatment processes is presented in Chapters 5 – 9.

4.11 Alternative Treatment Processes

The intent of this document is to provide guidance for the design of the extended aeration wastewater treatment process. The extended aeration process is the most commonly designed system for wastewater treatment plants. However, it is not the only treatment process. Various other treatment processes are proposed as alternative, innovative, nonconventional, or experimental designs. For these non-extended aeration proposals, the following issues should be resolved with the Ohio EPA prior to PTI application submittal:

A) Actual operating data sufficiently demonstrating that the proposed technology is capable of meeting all applicable permit limits,

B) Engineering report submitted providing justification for the usage and deviation from extended aeration process. Included in the Engineering report shall be a contingency plan outlining steps that will be taken if the alternative, innovative, nonconventional, or experiment designs fail to reliably meet all applicable permitted effluent limits,

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4.2 Siting

Proper siting of new wastewater treatment plants shall conform to the isolation distances in rule OAC 3745-42-08[14]. For existing wastewater treatment plants that are being modified, smaller isolation distances may be allowed if:

A) Isolation distances in the rule would impede the function of the existing wastewater treatment plant; or

B) The facility would incur more cost at the required distances.

If smaller isolation distances are allowed, the design must include mitigative measures such as:

A) Additional freeboard,

B) Landscape mounds,

C) Fencing,

D) Trees; and/or

E) Other means to reduce impacts

4.21 Noise Nuisance

Positive displacement blowers are generally used to provide aeration for wastewater treatment plants. These blowers can generate excessive noise when operating. Appropriate measures should be taken to reduce the noise generated in locations where excessive noise may be objectionable.

4.22 Odor Control

A well maintained and operated wastewater treatment plant should have minimal nuisance odors. Odors can be released in processes that store or convey raw wastewater, such as discharges from long forcemains, influent pump stations, and flow equalization basins. Other odors can be generated by treatment processes that are not operating properly. The best remedy for odor control is adequate process control by a conscientious operator who can identify and troubleshoot the problem.

4.23 Topography

The topography of the proposed site will influence the horizontal and vertical control of the design. Flood plain, erosion/stormwater, and the hydraulic profile of the influent sewer

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[14] ***See A.1, in the Appendix for appropriate isolation distances***

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relative to the rest of the treatment plant are all issues that need to be addressed in the design.

A) *Flood Plain* - The treatment works structures, electrical and mechanical equipment shall be protected from physical damage by the one hundred (100) year flood. Treatment works should remain fully operational and accessible during the twenty-five (25) year flood. This requirement applies to new construction and to existing facilities undergoing major modification. Flood plain regulations of local, state, and federal agencies shall be considered.

B) *Erosion Protection and Stormwater Drainage* – Stormwater management is essential to avoid erosion damage and to keep clean water from entering the wastewater treatment plant. Effective drainage design must be provided around the area surrounding the wastewater treatment plant. In order to prevent surface water from flowing into the plant, the walls should extend at least six inches above the surrounding ground and surface drainage shall be diverted away from the plant.

C) *Influent Sewer Elevation* - When the invert of the influent sewer is deeper than four (4) feet below grade, either:

   1) A lift station should be provided and the plant set at grade, or

   2) A retaining wall or excavation with four (4) feet clear horizontal distance around the plant be installed. Steps shall be provided for safe access.

   When the invert of the influent sewer line is shallower than four (4) feet below grade, riser wall sections may be used to extend the height of tank walls to at least six inches above the surrounding ground:

   1) Blower housing and electrical controls must be placed above or beside the riser section; and

   2) All valve handles and cleanouts must be brought up to a minimum of one (1) foot from the top of the riser for easy maintenance.

4.3 Asset Protection

A wastewater treatment plant is a major infrastructure asset. The treatment plant operators and staff represent an investment in expertise. These assets combine to protect human health and the environment downstream. The following measures are designed to enhance the security of the treatment plant and the safety of operators and staff.

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4.31 Security

A) A rugged fence of chain-link, wood, or other suitable material at least six (6) feet high should surround the wastewater treatment plant (i.e. trash traps, disinfection). Alternatively, a wastewater treatment plant may be constructed inside a building. Isolated treatment units may be fenced individually.

B) Entry gates/doors must be equipped with locking mechanisms to keep unauthorized individuals from entering the facility.

C) A minimum of four (4) feet working space must be provided between the treatment units and the security fence or building walls. Additional space may be required to accommodate trucks, loaders, equipment, etc. as necessary.

4.32 Protection of Equipment

A) Wastewater treatment plants, located in areas where thrown objects (vandalism) or falling leaves might be a problem, should be equipped with lightweight open grating over all tanks in addition to the fencing described above.

B) Blowers and motors shall be located in an enclosure or placed within a building for protection from moisture and debris.

C) All electrical controls shall be placed in waterproof enclosures and comply with NEMA codes.

D) Equipment located over treatment tanks, or in close proximity to them shall be designed to resist corrosion. Otherwise, equipment should be located on an adjacent pad or in an adjacent enclosure.

E) Hosing facilities for routine flushing of walls and walkways should be provided. Where a water supply is not available, a pump equipped with a hose connection may be used to pump clarified water from the chlorine contact tank for this purpose. If a potable water supply is used, refer to OAC 3745-95 for applicable requirements.

4.33 Protection of Workers

A) If a building is provided, the building shall be of sufficient size to provide adequate space (including head room) for operation and maintenance activities. The building should be constructed in accordance with all local building department codes.

B) Adequate lighting should be provided if the treatment plant is inside a building. Proper lighting levels should be provided within all buildings to allow for the safety

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of the operators and to allow the operator to comfortably observe all the parts of the wastewater treatment plant.

C) Sturdy grating shall be provided on all open tanks and along walkways. For larger treatment plants, handrails with kickplates may be substituted around open tanks.

D) All equipment, control valves, etc., shall be safely accessible from a position where firm footing is available.

E) Motor shafts, pulleys, belts, etc., shall be guarded.

F) Above or below ground treatment units shall be accessible by stairways.

G) Occupational Safety & Health Administration (OSHA), American National Standards Institute (ASNI) and/or Public Employment Risk Reduction Program (PERRP) rules shall be met.

4.4 Hydraulic & Organic Loadings

In the absence of site specific influent data, a wastewater treatment plant's hydraulic and organic loadings should be designed in accordance with OAC 3745-42-05. This rule states that a wastewater treatment plant's peak hourly design flow shall be calculated using the formula in Figure 2.1.

Helpful Tip #5:

A properly designed wastewater treatment plant is based on peak hydraulic loadings (i.e. what the collection system is designed to transport) and the average organic loadings that are expected to be treated from domestic sanitary wastewater. Because hydraulic and organic loadings can vary considerably depending on the population that is being served and even on the length of the collection system, relying upon textbook "rules of thumb" can lead to both overdesign and under design of wastewater treatment plants.

For wastewater treatment plants that receive flows from schools, institutions, or roadside rest areas with low flow water conservation fixtures in place, hydraulic loads tend to be very low but the ammonia concentrations tend to be very high. Some waste streams have influent ammonia concentrations greater than 60 mg/L and as high as 150 mg/L. These high ammonia concentrations can impact the biological treatment process, especially when large flow equalization basins pump large volumes of high ammonia loads and/or low alkaline water into the aeration tanks. Often, nitrification will proceed until all of the background alkalinity in the raw wastewater is consumed. Once the alkalinity is consumed, nitrification ceases and pH drops resulting in ammonia and pH violations. To accommodate high ammonia concentrations:

1) Adjust the floats in the flow equalization basin to pump less flow each cycle,

2) Provide for aeration tank blowers to cycle ON/OFF to encourage denitrification in the aeration tank,

3) Design an anoxic tank into the system for denitrification, or

4) Provide a method to feed alkalinity to the aeration tank.

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Figure 2.1 applies to treatment plant designs that do not include flow equalization basins. For wastewater treatment plants that include flow equalization, Figure 2.1 should be considered in the flow equalization basin design. The downstream wastewater treatment units will be designed based on the equalized flow and loadings of the flow equalization pumps and distribution box.

4.5 Design Influence

The goal of the wastewater treatment plant is to treat the collected wastewater sufficiently to prevent negative impacts to human health, aquatic life, and the environment. After being properly treated, the wastewater can then be discharged back into waters of the state. In order to maintain compliance with its NPDES permit limits, a wastewater treatment plant must be designed to provide reliable treatment despite fluctuating characteristics of the influent waste stream. These fluctuations include diurnal flow, diurnal organic loading, and seasonal variations brought on by temperature and wet weather events.

A well designed treatment plant will not only provide adequate treatment capacity but will also provide operators with process control options to respond to fluctuating conditions. This will enhance the wastewater treatment plant's reliability and overall efficiency to consistently meet permit limits.

The design of a wastewater treatment plant needs to consider the relationship from unit to unit and how each unit "influences" the downstream design of the treatment process. <u>For the usage of this document</u>, the wastewater treatment plant can be broken up into (4) four distinct zones. Each zone has unique design features to consider that will "influence" the design of pump rates, tank sizing, overflow rates, etc., in the downstream zones. These zones and specific design considerations are as follows[15]:

4.51 Zone 1: Influent and Preliminary Processes

Smaller systems will have trash traps to capture floatables while larger systems will have screening devices. Flow equalization basins provide flow control and organic loading control which will allow for more efficiently designed treatment units based on the controlled equalized flows rather than peak influent flows. The treatment units in Table 4.1 affect the WWTP design in the following manner:

[15] ***See A.12 in the appendix for a diagram showing stages and locations of the treatment units within those stages.***

Table 4.1: Influent & Preliminary Process

Unit	Design Consideration	Design Influence
Influent Sewer	Must convey all flow to treatment plant	Peak hydraulic & peak organic loading to WWTP
Trash Trap	Must remove floatables and grit	Keeps inorganic trash out of WWTP
Screening	Must remove inorganic material	Keeps inorganic material out of WWTP
Flow Equalization	Must store peak hydraulic and organic loadings	Aeration tank and clarifier sizing

4.52 Zone 2: Secondary Treatment Processes

The secondary treatment process is the heart of the biological wastewater treatment plant. In the aeration tank, the organic material is consumed by bacteria. In the clarifier, bacteria are separated from the clean water. The treatment units in Table 4.2 affect the WWTP design in the following manner:

Table 4.2: Secondary Treatment Processes

Unit	Design Consideration	Design Influence
Splitter Box	Must control forward flow	Aeration tank and clarifier sizing
Aeration Tank	Must match treatment capacity to organic load	Effluent quality and solids handling capacity
Clarifier	Must handle hydraulic load capacity	Effluent quality, RAS pumping and downstream tertiary filters

4.53 Zone 3: Tertiary Treatment Processes

These processes remove the remaining solids that escape clarifiers/up-flow fixed media filters (UFFM), provide a safety net for upstream treatment unit upsets, and kill/inactivate pathogens. The treatment units in Table 4.3 affect the WWTP design in the following manner:

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Table 4.3: Tertiary Treatment Processes

Unit	Design Consideration	Design Influence
UFFM (if applicable)	Must handle hydraulic load capacity	Effluent quality and downstream tertiary filters
Dosing Tank	Must handle hydraulic loading capacity	Sand filters and disinfection
Sand Filters	Must handle hydraulic loading capacity	Effluent water quality, disinfection
Disinfection	Must handle hydraulic loading and dosing	Effluent water quality
Post Aeration	Must provide air supply, hydraulic loading	Effluent water quality

4.54 Zone 4: Solids Handling

Solids handling provides the operator the ability to remove excess biomass that is generated during biological treatment. This process control capability is critical to a properly functioning wastewater treatment plant. The treatment units in Table 4.4 affect the WWTP design in the following manner:

Table 4.4: Solids Handling

Unit	Design Consideration	Design Influence
Sludge Holding Tank	Must provide adequate storage capacity	Aeration tank, clarifier, tertiary filters, effluent water quality.
Decant Pump	Must provide level adjustment	Sludge holding tank capacity
Dewatering Bag	Must provide adequate storage capacity, polymer feed	Aeration tank, clarifier, tertiary filters, effluent water quality.
Sludge Drying Beds	Not recommended	Labor intensive, capacity

The design of the wastewater treatment plant needs to provide operators with the controls to respond to fluctuating conditions that will help enhance the wastewater treatment plant's reliability and overall efficiency to consistently meet permit limits. This concept of flexibility is incorporated throughout the proceeding chapters through design recommendations and helpful tips. The goal is to help design a wastewater treatment plant that will consistently meet NPDES permit limits and run efficiently.

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Chapter 5. Flow Equalization

5.1 Introduction

Flow equalization serves to control the hydraulic and organic loadings to the treatment tanks. Diurnal peak flows and loadings can be reduced and stored in the flow equalization basin and then dosed to the rest of the treatment system in a controlled manner so as not to overload the treatment units. The pumps in the flow equalization basin direct raw wastewater into a flow distribution box. The flow distribution box is designed to discharge a controlled flow rate to the downstream treatment units while excess flow is returned to the flow equalization basin.

5.2 Requirements

Flow equalization is required for all wastewater treatment plants that receive diurnal peak flows and loadings that would be detrimental to the operation of the facility and may result in NPDES permit violations. Generally, the desired equalized flow rate should be based on organic loading rates for the aeration tank design and surface overflow rates for the clarifier design.

Flow equalization basins must be located downstream of the preliminary treatment units, such as trash traps, screening devices, and pump stations, except for a pump station that is incorporated into the flow equalization basin.

Helpful Tip #6:

There are two high flow scenarios that need to be considered when determining the design of flow equalization: diurnal flows and wet weather flows. Flow equalization basins for wastewater treatment plants covered in this document are primarily designed to even out the daily hydraulic peaks and organic peaks along with a "normal" amount of I/I. If an existing collection system is subject to excessive wet weather flows, then the trade-offs between constructing a large equalization basin, collection system rehabilitation, and clarifier surface overflow rate design must be considered.

A variety of methods may be employed to achieve flow equalization. Consideration may be given to in-line units, where all the flow passes through the equalization basin(s); and side-line units, where only that amount of flow above the desired flow rate *(usually design average flow)* is diverted through the equalization basin(s).

5.3 Volume Sizing

Flow equalization basin sizing will depend on the diurnal influent peak flow and loading patterns, the flow distribution box design, and the clarifier peak surface overflow rate design. The flow equalization pumps working with the flow distribution box must be able to draw down the water level in the basin under off-peak flow conditions. However, the forward flow through the flow distribution box cannot violate the peak surface overflow rate of the clarifier. The flow

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equalization basin volume must be sized to store the influent flow rates that exceed the forward flow rate through the distribution box.

Table 5.1: Flow Equalization Basin Design Criteria

Flows (gpd)	Minimum Tank Basin Volume (% ADF)
0 - 40,000	50
40,001 - 100,000	33

Example 5.1: Flow Equalization Volume Calculation

Assumptions:

Type of Facility = Mobile Home Park
ADF = 0.05 MGD

Step 1: Determine Q_{avg}

Q_{avg} = 50,000 gpd

Step 2: Determine % of ADF

= 33% *(See Table 5.1)*

Step 3: Determine Flow Equalization Volume

= 50,000 gpd * 0.33
= 16,500 gpd

Explanation: *This example illustrates the design calculation to be used when actual flow data is not available.*

A) When pumps are utilized, duplicate pumps shall be provided that can pass two (2) inch solids, if preceded by a trash trap and/or screening device. Otherwise the pumps should be capable of passing three (3) inch solids.

B) Aeration or mechanical mixing must be provided to prevent deposition of solids in the tank(s) and to maintain aerobic conditions. Minimum air requirement is four (4) cfm per 1,000 gallons of storage. Independent blowers and timers for the flow equalization basin are encouraged.

Helpful Tip #7:

Very small wastewater treatment plants may use process aeration for flow equalization mixing, treatment aeration and return sludge pumping. If process aeration is used for the flow equalization basin, valving should be installed to control air flow to the basin. This will prevent loss of aeration to the aeration tanks when the water level in the flow equalization is at a low level, and the back pressure over the flow equalization diffusers is reduced. Under this scenario, the airflow will follow the path of least resistance and be diverted to the flow equalization basin away from the aeration basin diffusers.

When a dedicated blower is provided for flow equalization, the blower should be equipped with a timer to control its operation. Excessive flow equalization capacity can be a detriment to treatment in the winter months by lowering the temperature of the influent wastewater. Cold water temperatures (less than 6^{o} C) can impact nitrification.

C) Corner fillets shall be provided to facilitate the periodic removal of any accumulated sludge or grit.

D) Equalization basins shall be suitably equipped with an effective and reliable device to permit flow control and ensure proper flow equalization such as a flow distribution box to limit the forward flow of the raw wastewater. This prevents organically overloading the aeration basins and hydraulically overloading the clarifiers, and will ensure an equal flow split between parallel treatment trains. Adjustable overflow weirs in the flow distribution box (forward flow and recycle flow) shall be required.

Helpful Tip #8:

Adjustable overflow weirs provide the most reliable method of flow control in a flow distribution box. Flow equalization pumps normally pump at much higher rates than the peak design flow of the wastewater treatment plant. By adjusting the forward flow weirs and the recycle flow weir, the forward flow will not violate the peak design flow rate of the wastewater treatment plant. Under flow weirs should be prohibited.

E) The equalization basin should be equipped with an overflow structure to insure that all wastewater flow will pass through the secondary and tertiary treatment units before being discharged. The overflow shall:

1) Be installed at the high water alarm level in the flow equalization basin

2) Be above the normal maximum liquid level in the aeration tank; and

3) Interconnect the flow equalization basins with the aeration tanks

F) Stand-alone pump stations shall not be utilized for flow equalization for all wastewater treatment plants.

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G) Audible and visual high water alarms shall be required for pumps in equalization basins.[16]

H) Equalization basin capacity should be sufficient to effectively reduce expected flow and load variations. With a diurnal flow pattern, the volume required to achieve the desired degree of equalization can be determined using Table 5.1 or from a cumulative flow plot over a representative 24-hour period.

Helpful Tip #9:
Sizing a flow equalization basin requires sufficient data of diurnal flow patterns. This data is rarely available for smaller wastewater treatment plants that use pump run-time meters to measure daily flows rather than dedicated flow metering devices. Similarly, totalizer data also does not have sufficient resolution to determine diurnal flows. Since this data is not available to construct a 24 hour cumulative flow plot, other methods will need to be used to estimate flow equalization basin volume. Table 5.1 can be used for minimum required volumes, but careful consideration must be given to peaking factors. For instance, as peaking factors increase, more flow would need to be equalized for a given average daily design flow.

Chapter 6. Preliminary Treatment

6.1 Introduction

The goal of preliminary treatment is to remove non-biodegradable nuisance solids to prevent damage to pumps and diffusers in the downstream treatment processes and to prepare the raw wastewater for biological treatment.

6.2 Trash Traps

The function of a trash trap is two-fold:

A) Heavy non-biodegradable solids (grit, rocks, etc.) settle to the bottom of the tank, as well as particulate organic material. The particulate organic material can solubilize in the anaerobic environment at the bottom of the tank.

B) Floatable and other light weight material (plastics, oil, grease, etc.) will rise to the top of the liquid surface. These materials are either non-biodegradable (plastics) or can promote the growth of undesirable bacteria in the treatment plant (oil and grease).

[16] *See Chapter 3: Pump Stations for additional audible and visual alarm requirements*

When sludge blankets and scum layers increase sufficiently to reduce the trash trap's effective volume, the trash trap should be pumped out.

6.21 Design

A) Trash trap capacity shall be based upon design criteria in A.5 of the Appendix,

B) Risers shall be provided at both inlet and outlet portions of the tank,

C) Scum baffles shall be provided across the tank influent to prevent short circuiting,

D) Outlet T's shall extend down to between 30 and 50 percent of the liquid depth; and

E) For fats, oils, and grease (FOG), see Section 2.5.

6.3 Screens

Screens come in two types: coarse screens and fine screens. Coarse screens, also known as bar racks, are designed to keep large objects such as rocks, wood, etc. from damaging pumps. Fine screens will reduce plastics and fibrous material from entering the treatment plant. This material can clog pumps and can restrict aeration in extreme cases. If biosolids are land applied, screens with maximum aperture of five-eighths inch (1.59 centimeters)[17] are required.

6.4 Flow Equalization

Refer to Chapter 5 for design criteria.

6.5 Flow Distribution Box

The proper design and setup of flow distribution boxes allow downstream treatment units to be sized smaller than if no flow equalization basin was installed. Inside the flow distribution box:

A) There shall be a forward flow overflow weir for each treatment train, plus an overflow weir returning flow back to the flow equalization basin.

B) All weirs shall be adjustable to give the operator control over the forward flow rate.

C) Underflow weirs on the flow back to the equalization basin are prohibited (Underflow weirs are less controllable than overflow weirs and are highly susceptible to clogging with debris, which then affects flow distribution),

D) A weep hole should drain back to the equalization basin so that wastewater won't freeze in the piping or distribution box during cold weather.

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[17] ***See OAC 3745-40 for additional requirements***

Chapter 7. Secondary Treatment

7.1 Introduction

The secondary treatment process is the heart of the wastewater treatment plant. In the aeration tank, the organic material is consumed and converted into bacteria mass. In the clarifier, bacteria are separated from the clean water.

7.2 Extended Aeration

The extended aeration process is the most commonly designed system for wastewater treatment plants. Extended aeration is characterized by forcing compressed air through submerged diffusers in order to dissolve oxygen in the mixture of bacteria and raw wastewater, commonly referred to as mixed liquor. It is a time tested method of treating wastewater and can reliably meet effluent limitations under widely varying conditions, providing that is it properly operated and maintained.

The design goal of aeration tanks is to match the organic loading to the aeration tank capacity. Under design will result in insufficient treatment and effluent violations. Overdesign will result in an unnecessarily expensive wastewater treatment plant to construct, operate, and maintain. Overdesign can also lead to potential operational difficulties which generate effluent violations, therefore wastewater treatment plants should be designed with flexibility in both hydraulic and organic loading treatment capabilities.

Helpful Tip #10:

The ability to direct influent flow to each of the aeration tanks can provide additional operational flexibility. For example, a three aeration tank configuration could have the pipe from the flow distribution box extended and valved so that flow can be directed to any of the aeration tanks. If the transfer lines between the tanks were at the water surface instead of subsurface and set at progressively slightly lower elevations, this would prevent back flow into the upstream tank. Likewise, the RAS pipe can be valved to discharge RAS into any aeration tank that the operator chooses. This gives the operator the flexibility to match the organic treatment (number of aeration tanks in service) to the influent organic loading. It also provides the capability to use contact/stabilization for high flow conditions due to rain events.

The design engineer must be cautious in determining design hydraulic and organic loadings in order to design a cost effective and efficient wastewater treatment system. There are numerous valid design calculations that are commonly used to size aeration tanks. However, each has particular criteria assumptions that need to be considered. Using textbook averages can lead to both over and under design if the actual criteria vary greatly from the textbook criteria. Using actual data, if possible, is always better, if the data is truly representative of the expected wastewater characteristics. Some of the most commonly used design calculations are as followed:

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7.21 Aeration Tank Design Options

A) *Organic Loading Method* – Design shall be based on influent sampling data. If no data exists, careful consideration of the wastewater source must be taken into account. The following influent concentrations may be used for standard domestic wastewater: $cBOD_5$ of 200-250 mg/L & NH_3-N of 20-30 mg/L. For facilities with low flow fixtures, the NH_3-N concentration values will be higher and for food operation facilities a higher $cBOD_5$ concentration may be appropriate. The above standard design concentrations do not apply to small diameter gravity collection systems *(less than 8" pipe)*.

For extended aeration treatment with nitrification, acceptable organic loading rates should not exceed 11-15 lbs. $cBOD_5$/day/1,000 ft^3 of aeration tank capacity.

Example 7.1: Organic Loading Calculation

Assumptions:

ADF	=	50,000 gpd
$cBOD_5$ concentration	=	225 mg/L
conversion factor	=	8.34 lbs. /gal
conversion factor	=	7.48 gal/ ft^3
organic loading factor	=	15 lbs. $cBOD_5$ / 1,000 ft^3

Step 1: Determine organic load

lbs. $cBOD_5$	=	ADF * $cBOD_5$ conc. * 8.34 lbs./gal
	=	0.05 MGD * 225 mg/L * 8.34 lbs./gal
	=	93.8 lbs. $cBOD_5$

Step 2: Determine Minimum Volume Required

Volume $_{AT}$	=	lbs. $cBOD_5$ / Max Organic Load Rate
	=	93.8 lbs. / 15 lbs. $cBOD_5$/1,000 ft^3
	=	6.255 1,000 ft^3
	=	6,255 ft^3 * 7.48 gal/ft^3
	=	46,800 gallons

Explanation: The total volume of aeration tanks shall not be less than 46,800 gallons. This volume would be divided up between the treatment trains and aeration tanks.

B) *24 Hour Hydraulic Detention Time Method* – Aeration tank size is based on the average daily design flow of the wastewater treatment plant. This is a very straight forward method, but does not take into consideration the organic loadings. For smaller wastewater treatment plant with a typical waste stream, this may be sufficient.

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Example 7.2: 24 Hour Detention Time Calculation

Assumptions:

ADF = 30,000 gpd
conversion factor = 7.48 gal/ft^3
tank dimensions = 6 ft. wide x 16 ft. long x 10 ft. deep = 960ft^3

Step 1: Determine total aeration tank volume

$$Volume_{total} = 30{,}000 \text{ gpd} / 7.48 \text{ gal/ft}^3$$
$$= 4{,}011 \text{ ft}^3$$

Step 2: Determine number of aeration tanks required

$$\# \text{ of tanks} = Volume_{total} / Volume_{AT}$$
$$= 4{,}011 \text{ ft}^3 / 960 \text{ ft}^3$$
$$= 4.2$$

Therefore, 4 tanks required

Step 3: Determine Detention Time

$$Volume_{total} = \# \text{ of tanks} * \text{tank width} * \text{tank length} * \text{average water depth}$$
$$= 4 \text{ tanks} * (960 \text{ ft}^3/\text{tank}) * 7.48 \text{ gal/ft}^3$$
$$= 28{,}700 \text{ gallons}$$

$$\text{Detention Time} = \text{Volume (gal)} / \text{Flow (gpd)} * 24 \text{ hrs.}$$
$$= 28{,}700 \text{ gal} / (30{,}000 \text{ gpd} * 24 \text{ hrs.})$$
$$= 23 \text{ hours}$$

Explanation: Detention time is approximately based on a 24 hour time frame. 20-24 hours detention time is generally acceptable.

C) *Food to Microorganism Ratio (F/M) Method* – Aeration tank size is based on a design organic loading and a design mixed liquor suspended solids concentration. To use the calculation, the design engineer must choose the design average daily flow, a design F/M, a design $cBOD_5$ concentration, and a design mixed liquor suspended solids concentration. This method assumes that the mixed liquor suspended solids concentration does not change and that the design F/M on average will not vary. Small changes in the design F/M choice will have large implications in the tank sizing. Also, inflating the mixed liquor concentration will reduce the size of the aeration tank. However, mixed liquor suspended solid greater than 3,500-4,000 mg/L will typically result in poor settling and potential clarifier failure. Since this method requires several assumptions on many variables it is the least desirable method for designing aeration tank volumes.

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Example 7.3: F/M Ratio Method Calculation

Assumptions:

ADF	=	50,000 gpd
F/M ratio	=	0.05 – 0.1
$cBOD_5$ concentration	=	225 mg/L
MLSS concentration	=	3,000 mg/L
F/M ratio equation	=	($cBOD_5$ loading * ADF) / ($Volume_{AT}$ * MLSS)

Step 1: Choose F/M Ratio

= 0.05

Step 2: Determine $cBOD_5$ & MLSS loadings

$cBOD_5$ = 225 mg/L * 8.34 lbs./gal * 0.05 MGD
= 94 lbs. $cBOD_5$

MLSS = 3,000 mg/L * 8.34 lbs. / gal * 0.05 MGD
= 1,250 lbs. MLSS

Step 3: Determine aeration tank total volume

$Volume_{AT}$ = (lbs. $cBOD_5$ * ADF) / (lbs. MLSS * F/M ratio)
= 94 lbs. $cBOD_5$ * 0.05 MGD / (1,250 lbs. MLSS * 0.05)
= 75,100 gallons *(this is probably excessive, see explanation)*

Explanation: If an F/M ratio of 0.1 is used, the aeration tank volume is 37,600 gallons which is considerably lower than the volumes calculated in the "Organic Loading" method and the "24-hour detention time" method. Textbook ranges for F/M ratios are anywhere from 0.05-0.1. This illustrates the high variability with selecting parameters. The other variable, MLSS, can also affect the aeration tank volume. Choosing a higher MLSS concentration will shrink the size of the aeration tank, however, high MLSS concentrations can have a negative effect on clarifier performance, (i.e. poor settling, loss of solids, etc.). Using a 0.05 F/M ratio, a MLSS concentration of 5,000 mg/L would be required to achieve an aeration tank volume of close to 45,000 gallons. Thus, the use of F/M is strongly discouraged.

D) *Mean Cell Residence Time (MCRT) Method* – Aeration tank size is based on a design mean cell residence time, a design average daily flow, a design mixed liquor suspended solids concentration, a design waste sludge concentration, and a design effluent suspended solids concentration. This method is particularly susceptible to errors in the criteria assumptions. Small variations in either the design mixed liquor suspended solids concentration or either the waste sludge flow or waste sludge concentration will have large effects on the size of the aeration tanks. In addition, this method does not take into consideration any characterization of the influent wastewater. The MCRT method, while useful in operations to determine wasting rates, it is not the most effective method in determining aeration tank volume.

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Example 7.4: Mean Cell Residence Time Calculation

Assumptions:

ADF	=	50,000 gpd
F/M Ratio	=	0.05
MLSS concentration	=	3,000 mg/L
MCRT Equation	=	$(Volume_{AT} * MLSS) / (Q_{WAS} * MLSS_{WAS} + Q_{eff} * X_{eff})$

Step 1: Choose design parameters

Design MCRT	=	12 days
Design MLSS	=	3,000 mg/L
Design ADF (Q_{eff})	=	50,000 gpd
Design Effluent TSS (X_{eff})	=	6 mg/L
Design Wasting Rate (Q_{WAS})	=	1,000 gpd
Design Wasting MLSS ($MLSS_{WAS}$)	=	8,000 mg/L

Step 2: Determine aeration tank volume

$Volume_{AT}$ = $MCRT * (Q_{WAS} * MLSS_{WAS} + Q_{eff} * X_{eff}) / MLSS$
= 12 days * (1,000 gpd * 8,000 mg/L + 50,000 gpd * 6 mg/L) / 3,000 mg/L
= 33,200 gallons *(This is probably insufficient)*

Explanation: Because this method does not take into account the influent organic concentration, the selected parameters must be chosen carefully. The MCRT value is more of an operational parameter than a design parameter. An operator will use the MCRT to determine how much to waste in order to maintain conversion and separation. Many environmental factors will determine sludge yield, wasting rates and concentrations, and may vary considerably when maintaining wasting rates and the mixed liquor concentrations. Care must be taken that the design MCRT, MLSS, and the wasting parameters will result in realistic operational conditions.

All four of these methods are acceptable design methods for sizing aeration tanks. However, for Greenbook sized systems (100,000 gpd or less), methods A) and B) are the most appropriate design choice. Methods C) and D) require numerous assumptions that could lead to over/under design of the wastewater treatment plant.

7.22 Additional Requirements

A) Parallel aeration tank treatment trains are required for wastewater treatment plants designed for flows over 50,000 gallons per day.

B) Ports between tanks must be large enough to prevent liquid level buildup within any tank. Ideally, transfer ports between tanks should be at the water surface to prevent foam trapping.

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Helpful Tip #11:

If the surface port inverts are installed at progressively lower elevations through the treatment train, a hydraulic gradient occurs and will help to prevent back mixing. This is particularly important if the influent and RAS piping are extended and valved to allow isolation of individual tanks. Together, these features give the operator the ability to set up contact stabilization for wet weather flows or to reduce treatment capacity to match the organic loading by taking tanks offline, if necessary.

C) To accommodate seasonal and/or variable loadings, aeration tank design should incorporate flexibility to place appropriate number of tanks online/offline.

D) Sufficient freeboard should be added to the design volume of aeration tanks to maintain the mixed liquor within the tank. Eighteen inches (18) of freeboard is generally considered sufficient.

Helpful Tip #12:

For treatment plants that have been designed for future flows that may take years to develop, aeration tank design should provide the operator the ability to place aeration tanks online or take aeration tanks offline in order to match the treatment capacity to the influent organic load. This would require that aeration tanks in series have the influent and RAS piping extended and valved to isolate one or more of the tanks while still allowing the rest of the system to function. Together, these features give the operator the ability to reduce treatment capacity to match the organic loading by taking tanks offline, if necessary, or set up a contact stabilization mode of operation for wet weather flows.

For instance, a low loaded treatment plant that was only receiving 50% of its design load would only need 50-60% of its organic treatment capacity. If the treatment plant consists of three aeration tanks in series, then the ability to remove one of the tanks would allow the operator to optimize treatment. This design feature would require that influent flows can be directed to any aeration tank. Also the return activated sludge would also need to be directed to any aeration tank.

For systems experiencing high hydraulic loadings from inflow and infiltration (I/I), the design should incorporate a contact stabilization (storm mode) mode of operation. This design feature directs raw wastewater into an aeration tank downstream of the aeration tank where return sludge is discharged. This helps prevent the washout of mixed liquor by reducing the solids loading rate on the clarifier and protects the return sludge from the hydraulic pressures associated with I/I.

7.23 Air Diffusion Design

A) Each aeration tank header and diffuser drop pipe must be adjustable to balance the air supply.

> ***Helpful Tip #13:***
>
> Having a dedicated blower for a conventional airlift return sludge pump allows a clarifier to be operated independently of aeration tank operation. For instance, for a low organically loaded treatment plant, the operator may choose to run the aeration tank in an ON/OFF mode with timers. If all aeration comes from one blower, then the clarifier return sludge pumping stops during an air off cycle. By having a small dedicated blower for the RAS airlift pump, additional flexibility is given to the operator to optimize the wastewater treatment plant.

B) 2,050 cubic feet of air (1.4 cfm) per pound of applied BOD_5 (carbonaceous and nitrogenous oxygen demands) is required for extended aeration plants unless adequate justification has been provided. Additional capacity should be provided to operate air lifts, skimmers, sludge wasting facilities, and tertiary treatment facilities.

Example 7.5: Air Requirement Calculation

Assumption:

lbs. $cBOD_5$	=	1.1 lbs. O_2
lbs. NH_3-N	=	4.6 lbs. O_2
ADF	=	50,000 gpd
Organic concentration: $cBOD_5$	=	137 mg/L
Inorganic concentration: NH_3-N	=	52 mg/L
ft^3 air	=	0.0173 lbs. O_2

Step 1: Determine the average daily flow & concentrations

ADF	=	50,000 gpd
$cBOD_5$	=	137 mg/L
NH_3-N	=	52 mg/L

Step 2: Convert concentration to mass

= 137 mg/L $cBOD_5$ * 0.05 MGD * 8.34 lbs./gal
= 57 lbs. $cBOD_5$
= 52 mg/L NH_3-N * 0.05 MGD * 8.34 lbs./gal
= 22 lbs. NH_3-N

Step 3: Determine oxygen demand

= 57 lbs. $cBOD_5$ * 1.1 lbs. O_2 / lb. $cBOD_5$
= 63 lbs. O_2
= 22 lbs. NH_3-N * 4.6 lbs. O_2 / lb. NH_3-N
= 101 lbs. O_2
= 63 lbs. O_2 + 101 lbs. O_2
= 164 lbs. O_2

Step 4: Determine Volume Air Required & Choose Diffuser Type

Volume$_{AIR}$	=	164 lbs. O_2 / 0.0173 lbs. O_2 / ft^3 air
	=	9,480 ft^3 / day air
Coarse bubble	=	9,480 ft^3 / day air / 0.05 O_2 transfer efficiency
	=	**189,600 ft^3 / day**
Medium bubble	=	9,480 ft^3 / day air / 0.10 O_2 transfer efficiency
	=	**95,000 ft^3 / day**
Fine bubble	=	9,480 ft^3 / day air / 0.25 O_2 transfer efficiency
	=	**37,920 ft^3 / day**

Step 5: Determine Standard Cubic Feet per Minute

Coarse bubble	=	189,600 ft^3 / day / 1,440 min / day
	=	**132 scfm** for organic treatment
Medium bubble	=	95,000 ft^3 / day / 1,440 min / day
	=	**66 scfm** for organic treatment
Fine bubble	=	37,920 ft^3 / day / 1,440 min / day
	=	**26 scfm** for organic treatment

Explanation: This calculation demonstrates the air requirements for treating the influent organic loading. Additional air demands for other processes such as air lift RAS pumps, skimmers, post aeration (*if applicable*), sludge digesters, etc. will require additional blower capacity. This additional blower capacity can be in the form of individual dedicated blowers or upsizing process air units. The decision to use coarse, medium, or fine bubble diffusers needs to consider the appropriate number of diffusers and impacts on blower selection. One cannot simply replace coarse bubble diffusers with fine bubble diffusers. The aeration system as a whole requires close evaluation.

C) Stand-by blower unit must be installed and operable.

Helpful Tip #14:

Standby blower(s) should be manifolded and valved so that independent operation of aeration tank air and air lift pumps can be configured. For instance, if the blower for an aeration tank is being operated with a timer in order to match aeration to the organic loading, the blower cycle might not be what would be required by the airlift sludge return pump. Being able to operate the airlift return pump independently from the aeration would allow the operator to provide the necessary air for both processes independently.

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7.3 Clarifiers

Final clarifiers following the aeration units shall be designed to give effective settling and continuous return of sludge. Design of the clarifier should be based on the following requirements:

A) *Duplicity* - Duplicate clarifiers are required for plants treating over 50,000 gallons per day average daily flow. Smaller plants may be required to have duplicate clarifiers. The installation of multiple clarifiers in series will not be accepted.

B) *Clarifier Types* - Extended aeration package plants usually have hopper type clarifiers with airlift return sludge pumps. These clarifiers will have one or two inverted "pyramid-shaped" hoppers where sludge collects and compacts. The compacted sludge is pumped back to the aeration tank with an airlift sludge pump. Larger extended aeration treatment systems may have circular or rectangular clarifiers with mechanical sludge scrapers that direct the settled sludge to either a sludge pump or to a telescoping valve connected to a sludge wet well. The design of larger clarifiers should refer to Ten State Standards for guidance.

C) *Peak Surface Overflow Rate (SOR)* - Hopper type clarifiers without active sludge scraping should be designed for a peak surface overflow rate of 500-600 gpd/ft^2. For treatment systems with flow equalization, the peak flow rate is the forward flow pumping rate of the flow equalization pumps. The peak flow rate for treatment systems without flow equalization is the peak hourly influent flow rate.

Helpful Tip #15:

Acceptable peak surface overflow rates for clarifiers will depend on the type of clarifier. For hopper type clarifiers in package plants, the peak overflow rate would be on the low end of the range (500 – 600 gpd/ft^2). For clarifiers with mechanical sludge collectors, acceptable peak overflow rates would be on the higher end of the range (600 – 800 gpd/ft^2).

Example 7.6: Surface Overflow Rate Calculation

Assumptions:

ADF	=	50,000 gpd
Peak ADF	=	165,000 gpd
Forward Flow from Splitter Box	=	75,000 gpd
2 clarifiers	=	14 ft. long x 6 ft. wide

Step 1: Determine peak flow rate to clarifiers

= 75,000 gpd

For this example, the assumption is that the flow splitter box is set up for a forward flow rate of 52 gallons per minute, or approximately 75,000 gallon per day.

Step 2: Determine the surface area for the clarifiers

$$\text{Surface Area} = \text{\# of clarifiers} * \text{clarifier length} * \text{clarifier width}$$
$$= (2) * (14 \text{ ft.}) * (6 \text{ ft.})$$
$$= 168 \text{ ft}^2$$

Step 3: Determine surface overflow rate

$$\text{SOR} = \text{flow / surface area}$$
$$= 75{,}000 \text{ gpd} / 168 \text{ ft}^2$$
$$= 446 \text{ gpd/ft}^2$$

***Explanation**:* Surface overflow rates reflects the interaction between the upward flows of the clear water vs. the downward settling rates of the sludge particles. High overflow rates counteract the settling rates resulting in the loss of solids. By maintaining a satisfactory overflow rate, this will prevent solids from leaving the clarifier and settling out on the tertiary treatment filters.

D) *Solids Loading Rate (SLR)*

Helpful Tip #16:

Clarifiers can fail due to excessive solids loading applied to them. A common calculation for this is the Solids Loading Rate. For larger clarifiers, this is commonly calculated and compared to a design specification of 35 lbs. per day per ft^2. Although this calculation is not usually performed in the design of package plant (hopper type) clarifiers, an excessive solids loading rate can impact the performance of the clarifier. In order to perform the calculation, several numbers are required:

1) Influent flow (for equalized flows, this would be the forward flow from the splitter box),
2) The RAS flow (this would include all of the return flows from all clarifiers, i.e., first hopper RAS, second hopper RAS and all skimmers),
3) The mixed liquor concentration (2,000 mg/L – 3,500 mg/L); and
4) The surface area of the clarifiers.

The SLR will vary mostly with the MLSS and the airlift RAS and skimmer rates.

Example 7.7: Solids Loading Rate Calculation

Assumptions:

ADF	=	50,000 gpd
Peak ADF	=	165,000 gpd
Forward Flow from Splitter Box (Q_i)	=	75,000 gpd
Dimensions of clarifier	=	14 ft. long x 6 ft. wide
Number of clarifiers	=	2
# of Return Airlifts / clarifier	=	2
Return Flow Rate / airlift	=	20,000 gpd
Mixed Liquor Suspended Solids (MLSS)	=	3,000 mg/L

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Step 1: Determine Q_i and Q_{rtotal}

$$Q_i = 0.075 \text{ MGD}$$
$$Q_{rtotal} = (2 \text{ clarifiers}) * (2 \text{ returns / clarifier}) * (0.020 \text{ MGD / return}) = 0.080 \text{ MGD}$$

Step 2: Determine the surface area for the clarifiers

$$\text{Surface Area} = \text{Number of clarifiers} * \text{clarifier length} * \text{clarifier width} = (2) * (14 \text{ ft.}) * (6 \text{ ft.}) = 168 \text{ ft}^2$$

Step 3: Determine solids loading rate

$$SLR = \frac{(Q_i + Q_{rtotal}) * 8.34 \text{ lbs. / gal} * MLSS}{\text{Surface Area}} = \frac{(0.075 \text{ MGD} + 0.08 \text{ MGD}) * 8.34 \text{ lbs. / gal} * 3{,}000 \text{ mg/L}}{168 \text{ ft}^2} = 23.1 \text{ lbs. / day / ft}^2$$

Explanation: Although this is less than the 35 lbs./day/ft^2 that 10 State Standards specifies, for hopper type clarifiers, 25 lbs./day/ft^2 should not be exceeded. Keeping the mixed liquor concentrations low and optimizing the RAS rate are the best ways to reduce the solids loading rate on the clarifiers. An excessively high RAS rate associated with conventional airlift RAS pumps will add an additional hydraulic stress to the system and potentially lead to clarifier failure, both from a solids loading perspective and from a hydraulic *(surface overflow rate)* perspective.

E) Sludge Pumping (RAS)

1) The sludge return rate should be adjustable in order to match the return pumping rate to the settling characteristics of the mixed liquor. A fast settling sludge needs a fast return rate. A slow settling sludge requires a slow return rate. For a conventional air lift return sludge pump, valving should be provided for the air supply and the return sludge piping in order to control the return sludge rate.

2) Air supply for airlift RAS pumps should have the ability to operate independently from the aeration tanks.

3) Discharge from the sludge pumps shall be visible in order to verify that: 1) the pump is operational, 2) the sludge can be measured, and 3) the sludge is being returned. Each hopper shall have separate sludge return pumping and return sludge piping. Piping and valving shall not be less than two (2) inch diameter.

4) A cleanout shall be installed on the “tee” where the vertical RAS pipe transitions to horizontal. This enables the operator to rod the RAS pump inlet in the event of a clog.

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Helpful Tip #17:

Conventional airlift pumps are difficult to adjust. They operate in a very narrow range based on the air supply to them. With multiple airlift pumps returning sludge to an aeration tank, the resulting high return flow rate can impact the sludge settling in the clarifier. The ability to slow down sludge return pumping to give enough time for the sludge to settle provides the operator flexibility to optimize clarifier performance. Modifications to the traditional airlift pump are available that can provide a wider operational range for sludge return pumping and may be considered.

An independent source of air supply should be provided to operate air lift pumps. If the blower for an aeration tank is being operated with a timer in order to match aeration to the organic loading, the air lift return pump would not operate during an air off cycle. Sludge may accumulate in the clarifier to unacceptable levels and/or may denitrify causing the sludge to "pop". This may cause loss of solids from the clarifier which would result in solids accumulation on the tertiary sand filters or in the receiving stream. Being able to operate the airlift return pump independently from the aeration would allow the operator to provide the necessary air for both processes.

F) Sludge Hopper

1) The base of the sludge hopper should be no greater than one foot square or one foot diameter. The inlet of the airlift RAS pump should extend down to within six (6) inches of the sludge hopper base. Inlet pipes should be cut at an angle to reduce clogging.

2) Multi-hoppered tanks should provide a minimum water depth of two (2) feet above the sloped side wall.

3) The installation of more than four (4) hoppers per settling tank or more than three (3) hoppers in a row per settling tank will not be accepted.[18]

G) Inlet Structures

1) The transfer line from the aeration tank to the clarifier shall end in an upturned elbow to help dissipate velocity and momentum of the mixed liquor.

2) An influent baffle shall extend at least six (6) inches above the water line down to within a foot of the hopper slope line. This provides for a flocculation zone and prevents density currents through the clarifier. Also, this creates an effective collection area for floatable material, such as foam and scum, preventing the migration of material across the surface of the clarifier.

[18] *See A.10 in the appendix for additional information on clarifier design*

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3) The location of the RAS/skimmer return pipes should not impede/interfere with the influent scum baffle cleaning and maintenance.

H) Outlet Structures

1) Outlet structures consist of weir baffles, adjustable weirs, and effluent troughs.

2) The weir baffle should be located close to the effluent trough and extend sufficiently above and below the liquid level to keep scum and floatables from entering the effluent trough.

3) Weirs will be attached to both sides of the effluent trough and will extend across the width of the clarifier.

4) For unequalized flows, the weir overflow rate shall not exceed 2,500 gpd/lf at 24-hour design flow, nor 7,500 gpd/lf at peak flow. For equalized flows, the weir overflow rate should not exceed 7,500 gpd/lf.

5) The effluent trough should be located a sufficient distance from the end wall of the clarifier to offset the effect of density currents.

Example 7.8: Weir Overflow Rate Calculation

Assumptions:

Peak forward flow rate	=	75,000 gpd
Number of clarifiers	=	2
Width of each clarifier	=	6 feet

Step 1: Determine the peak forward flow rate

= 75,000 gpd

Step 2: Determine the effective length of the weirs. *(For Hopper Clarifiers, the weir trough has v-notch weirs on each side running the width of the clarifier)*

= # of clarifiers * clarifier width * weir lengths/clarifier
= (2 clarifiers) * (6 feet width) * (2 weir lengths)
= 24 feet

Step 3: Determine the Weir Overflow Rate (WOR)

WOR = peak forward flow rate / total weir length
= 75,000 gpd / 24 feet
= 3,125 gpd/ft

Explanation: Since the effluent weirs are the only "escape" route for the clear water, long broad weirs inhibit the development of density currents that can carry solids away from the clarifier and into the tertiary treatment units.

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I) Scum skimmers

Scum skimmers shall be provided. Airlift type scum skimmers shall have a valve on the air supply so adjustments can be made.

Helpful Tip #18:

Passive skimmers are rarely effective at removing floatables from clarifier water surfaces. Due to the varying water levels in clarifiers, fixed skimmers tend to be either completely submerged or completely above the water level. When submerged, the skimmer flow rate can be excessive, adding an additional hydraulic stress on the system. When the skimmer is out of the water, no water is skimmed and process air is vented to the atmosphere and unavailable for other uses (aeration, RAS airlifts, digester air, etc.). Even when the water level in the clarifier allows the skimmer to function properly, the skimmer usually draws from a small fraction of the clarifier's surface area. Skimmers work best when an operator is present and can physically direct floatables to the skimmer.

Chapter 8. Tertiary Treatment

8.1 Introduction

Tertiary treatment removes suspended solids from clarifier effluent, disinfects the water, and boosts the dissolved oxygen of the effluent prior to discharge to the receiving stream. These processes polish the treated wastewater and provide an added degree of reliability in meeting permitted limits. Tertiary treatment devices usually include up-flow fixed media clarifiers, dosing chambers, flow distribution boxes, surface sand filters, disinfection, and post aeration.

8.2 Up-Flow Fixed Media Clarifiers

Up-flow fixed media (UFFM) clarifiers are treatment units following final clarifiers. UFFM clarifiers capture solids that would otherwise end up in the final effluent with the added benefit of being easy to clean and maintain. Design recommendations to consider are:

A) The total area of fixed media decking shall be divided equally between two or more compartments.[19]

B) The fixed media shall be mounted parallel with the surface of the water and not less than six (6) inches below the weir elevation.

[19] *See A.11 in Appendix for method of construction and configuration*

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C) An inlet zone shall include a baffle to direct all flow beneath the filter media preventing short circuiting the UFFM clarifier.

D) UFFM clarifiers shall be no less than three (3) feet deep at the shallowest point and the base shall slope downward toward the sludge collection zone of the tank to facilitate sludge removal.

E) Adequate hose facilities using potable or clarified water shall be provided to routinely clean the fixed media.

F) Weir lengths in UFFM clarifiers are usually double sided to minimize weir overflow rates for a given geometry. The weir overflow rate shall not exceed 3,000 gallons per day/linear foot based on the average daily design flow.

G) The design of the UFFM clarifier is based upon the surface area of the filter media. The UFFM clarifier sizing shall not exceed 600 gpd / sq. ft. of media at peak flow rate.

Helpful Tip #19:

Proper sizing of UFFM clarifiers are dependent on peak flow rates. For treatment systems with flow equalization basins, the peak flow rates would depend on the number of flow equalization pumps and the weir settings on the influent flow distribution box. If the weir in the distribution box is set for the average daily flow, then this would be the normal peak flow rate. If the water level in the flow equalization basin reaches the high water alarm and calls for the second pump, then the peak flow rate would be 2 times the average daily flow rate or the maximum peak flow rate.

For treatment systems without flow equalization, the peak design flow rate should be in accordance with the peak flow calculation in Section 2.1. Careful consideration should be given to the unequalized peak flow rates because significantly undersized UFFM clarifiers are less effective and could result in noncompliance.

Example 8.1: UFFM Clarifier Sizing

Assumptions:

Forward Flow Rate = 130,000 gpd
Number of Units = 2
UFFM Design Criteria = 600 gpd/ft^2 *(not to exceed)*

Step 1: Determine Total Filter Area

Total Area = forward flow rate / UFFM design criteria
= 130,000 gpd / 600 gpd/ft^2
= 217 ft^2

Step 2: Determine Total Number of Filters

Total = Total filter area / individual UFFM media area
= 217 ft^2 / (8 ft. x 11 ft.)
= 2.4

Explanation: A total of 3 UFFM units would be required.

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8.3 Dosing Chamber

A dosing chamber is used to accumulate a sufficient volume of clarifier effluent to properly dose the tertiary sand filters. Intermittent dosing prolongs the life of filters by allowing "rest" periods between doses. A dosing chamber is required preceding all tertiary sand filters.

The effective volume of the dosing chamber should be such that the online bed of a tertiary sand filter will be theoretically flooded to a depth of three inches in 10 minutes or less.

Example 8.2: Dosing Chamber Pump Sizing

Assumptions:

ADF	=	20,000 gpd
Filter Area	=	800 sq. ft. (2 @ 400 sq. ft. each)
Dose Volume	=	1 sand filter flooded to theoretical depth of 0.25 ft
UFFM	=	Unit present

Step 1: Determine volume to be dosed

Volume	=	Filter area online * flooding depth
	=	400 sq. ft. * 0.25 ft. * 7.48 gal/ft^3
	=	748 gallons

Step 2: Determine pumping rate

Pumping rate	=	Volume / pump cycle time
	=	748 gallons / 10 minutes
	=	75 gpm

Explanation: The pumping rate is based on a 25 gpd/sq. ft. loading rate to the tertiary sand filters (*see Appendix A.6*), an 800 sq. ft. total filter area, and a 10 minute pump criteria. This is for determining a pumping rate not determining an effective volume for the dosing tank wet well.

Dosing equipment should be comprised of the following:

A) A dosing chamber, required for dosing all tertiary sand filters. Gravity dosing of tertiary sand filters is prohibited.

B) Two automatically alternating pumps,

C) A dosing chamber with access hatches sized and located to allow easy removal of pumps and equipment. Work platforms should be provided below pipe connections; and

D) Visual and/or high water alarms.

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Helpful Tip #20:

Sand filters work best when they are dosed intermittently. In order to effectively achieve this, two design considerations need to be examined: 1) The forward flow from the influent flow distribution box and 2) the dosing chamber pump. The dosing chamber will fill at the equalized forward flow rate. The dosing pumps typically pump at a much higher flow rate than the equalized forward flow rate. The difference between these two flow rates allows the intermittent dosing and subsequent "rest" time for the sand filters. Under high influent flow conditions when both influent pumps may be operating, the dosing pump must still be able to cycle without overflowing or over dosing the sand filters.

8.4 Flow Distribution Box

The purpose of the flow distribution box is to direct the flow leaving the dosing chamber to the online tertiary sand filters. The flow distribution box shall be:

A) Equipped with a means to isolate each sand filter distribution line. Threaded caps or watertight plugs are preferred,

B) Designed hydraulically to not overflow during operation of the dosing pump,

C) Water tight,

D) Equipped with a cover; and

E) Constructed of concrete. Aluminum flow distribution boxes may be considered to prevent degradation of flow distribution box.

8.5 Surface Sand Filters

Sand filters are a physical treatment process that captures suspended solids that escape the final clarifier and/or the UFFM. Not only does this improve efficiencies of the downstream disinfection processes such as chlorination and ultraviolet radiation, it also prevents suspended solids from entering the receiving stream. This treatment process provides added protection and assurance to meet final effluent limits. Design requirements include:

A) A minimum of two (2) beds, each capable of independent operation. Generally, for a two sand filter bed system, one-half the total filter area is to be intermittently dosed while the other half "rests".[20]

B) The sand filter beds shall be hydraulically loaded at 1.2 – 2.0 gal/ft^2.

C) Maximum size of a sand filter bed with a single distribution point is 625 square feet.

[20] ***See A.8 in the appendix for method of construction and configuration***

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Helpful Tip #21:

Sand filter designs must take into account the influent flows, flow equalization pumping rates, dosing tank pumping rates, number of sand filter beds, freeboard, and surface area. Under normal flows, sand filters can be very forgiving and provide adequate filtration as long as sufficient operator control is in place. However, during peak influent flow events, proper consideration of the above design factors can maintain treatment reliability under difficult circumstances.

For instance, if a flow equalization splitter box is set to deliver a forward flow rate equal to the average daily design flow all tertiary components should function normally. However, if influent flows are higher than the ADF, the flow equalization basin will begin to fill up to the point that the second pump is called. Once the influent flow rate reaches the peaking factor and the flow equalization basin overflow activates, the dosing pump will receive this unequalized flow and must be able to convey it to the sand filters.

D) Multiple points of discharge on the sand may be necessary for larger sand filter areas. Dosing pumps should be sized in accordance with Section 8.3.

E) Sand filter beds should be installed above-grade to prevent run-off from entering the filter beds.

Example 8.3: Sand Filter Sizing

Assumptions:

ADF	=	5,000 gpd
Number of Beds	=	2
Forward Flow Rate	=	7,500 gpd
Sand Filter Area Criteria	=	12 gpd/ft^2

Step 1: Determine Total Sand Filter Area

Total Area = Forward Flow Rate / 12 gpd/ft^2
= 7,500 gpd / 12 gpd/ft^2
= 625 ft^2 *(25' x 25')*

A minimum of two beds are required. Each sand filter bed will be 25' x 12.5'

Step 2: Determine Dosing Pump Size

Pump = Filter loading rate * surface area of 1 bed
= 2.0 gal/ft^2 * 312 ft^2
= ~ 625 gallons
= 625 gallons / 10 min pump cycle
= 62.5 gpm

Minimum pumping rate = Approximately 60 gpm

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F) The bottom of the beds shall be provided with an impervious liner with a maximum permeability rate of 1.0 x 10^{-7} cm/s; acceptable materials include concrete, synthetic liner, or clay.

G) Filter walls shall be constructed of reinforced precast or reinforced poured-in-place concrete. Particular attention should be made to ensure water tightness at corners and joints. An 18-inch freeboard is required.

H) To prevent short circuiting by the erosion of filter bed sand, concrete splash pads (typically 3' x 3') are required. Rip rap (rock protection) should be placed on and around the splash pad to break up velocity currents. Up-turned elbows with a weep hole at the discharge outlet from the distribution box may be considered.

I) An emergency overflow notch or transfer pipe should be installed on the common wall between adjacent filters. This would prevent a clogged filter from overflowing to the ground and instead overflow to the standby filter.

8.51 Filter Sand Specifications

A) Filter sand shall be clean and washed. The sand should have an effective size between 0.4 and 1.0 mm and a uniformity coefficient not greater than 4.0. In addition, less than 4% should pass a #100 sieve.[21]

Helpful Tip #22:

This sand specification is recommended for sand filters. However it may not be readily available in the State of Ohio. The selected sand should be as close to the recommended specification as feasibly possible. When selecting sand for sand filters, sand that is too coarse does not provide adequate filtering. Sand that is too fine can clog easily. Sand with excessive fines can pass through into the underdrain and diminish capacity. Unwashed sand can have particulates pass through into the effluent and show up as a total suspended solids violation.

8.6 High Rate Effluent Filtration

High rate sand filters are not common at small wastewater treatment plants. However many manufacturers can provide proprietary high rate sand filters for smaller treatment plants. Approval of these proprietary sand filters will be on a case by case basis. Important considerations for approval would be:

A) Operational and maintenance complexity,

B) Reliability,

C) Hydraulic capacity; and

D) Documentation that the sand filter can meet the applicable effluent limitations.

---

[21] *See Washington Filter Sand Specification Study in the Reference Section*

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A major concern would be that the backwash flow rates do not cause hydraulic issues in the wastewater treatment plant, especially on clarifiers.

Design criteria for high rate effluent filters shall be in accordance with Ten State Standards.

The filter must be located in a building of sufficient size to provide adequate space (including head room) for operation and maintenance. The building should be constructed in accordance with local building codes.

8.7 Disinfection

In Ohio, all wastewater treatment plants with NPDES permits must be able to comply with pathogen removal requirements. This is done by a disinfection process which either destroys or prevents the growth of disease-carrying microorganisms in the treated effluent prior to discharge to waters of the state.

Disinfection equipment should be located following tertiary filters, readily accessible, and designed for proper operation during all months that are required by the NPDES permit. Equipment may be located in the building being served or in a suitable structure or enclosure at the plant site.

8.71 Ultraviolet Disinfection

Ultraviolet (radiation) disinfection occurs by ultraviolet (UV) rays deactivating the pathogenic *(i.e. virus, bacteria, & fungi)* organisms' DNA. The UV radiation interferes with the pathogen's ability to reproduce, leaving the organisms sterile.

A) UV disinfection is the preferred disinfection option due to:

1) Health and safety concerns of other options;

2) Consistent compliance results; and

3) Long term cost savings.

B) UV disinfection shall immediately follow the tertiary filters. High concentrations of suspended solids *(~ greater than 20- 30 mg/l TSS)* can shield pathogenic organisms from the UV radiation and allow viable pathogens to enter the receiving stream. Effluent should have at least a 65% UV radiation transmittance[22].

C) UV disinfection shall be sized to be effective at the peak flow rate *(dosing pump rate.)*

[22] ***UV transmittance represents the percentage of UV energy in the water that reaches the microorganisms***

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D) UV electronics susceptible to flooding should be located above ground in a suitable electrical enclosure or protected.

E) An alarm should be called if a bulb burns out.

F) Design must allow operators to determine whether UV is operating properly without having to disassemble the unit *(such as a runtime meter)*.

8.72 Chlorination

A) *Contact chamber* - a minimum detention period of 15 minutes shall be provided at peak flow rate *(i.e. dosing pump rate)* to the contact chamber. The chamber shall be baffled so that the flow path approaches plug flow (no short circuiting). The contact chamber should have an open top or an easily-removed cover.

B) *Mixing* - initial mixing of the chlorine solution and the wastewater should be as rapid and complete as possible. The ideal design calls for complete mixing before the effluent enters the contact chamber.

C) The chlorine dose usually ranges from 5 to 20 milligrams per liter (mg/L).

Example 8.4: Chlorine Contact Tank Design

Assumptions:

Detention Time = 15 minutes @ Peak Forward Flow
Peak Forward Flow = 15,000 gpd *(Both EQ pumps pumping)*

Step 1: Determine Chlorine Tank Volume

$$\text{Volume}_{\text{TANK}} = \text{Peak Forward Flow} * 1\text{ day}/1{,}440\text{ minutes} * 15\text{ min DT}$$
$$= 15{,}000\text{ gpd} * (1\text{day}/1{,}400\text{ min}) * 15\text{ min}$$
$$= 156\text{ gallon minimum or } 20\text{ ft}^3$$

Step 2: Determine Size of Tank

$$\text{Volume}_{\text{TANK}}\text{ Required} = 20\text{ ft}^3$$
$$24\text{ft}^3 = 3' \times 4' \times 2'$$

Minimum Size of Chlorine Contact Tank = 3 ft. x 4ft. x 2ft.

D) Hypochlorite is the only acceptable type of chlorine for disinfection. Chlorinated isocyanurates *(swimming pool disinfection products)* are not to be used in disinfection of wastewater treatment.

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E) The use of chlorine gas for disinfection is not acceptable due to health and safety concerns.

8.73 Dechlorination

A) If chlorination is being utilized as the disinfectant, dechlorination shall be required,

B) Dechlorination shall immediately follow chlorination; and

C) It shall be located as the effluent exits the chlorination tank.

Table 8-1: Forms of Dechlorination Chemicals

Dechlorination Chemical[23]	Theoretical Dosage Required to Neutralize 1 mg/L Cl_2 (mg/L)
Sodium bisulphite (solution)	1.46
Sodium meta bisulphite (solution)	1.34
Sodium sulphite (tablet)	1.78
Sodium thiosulphate (solution)	0.56

Helpful Tip #23:

Tablet feeders for chlorination and dechlorination must be installed above the normal water level in the chlorine contact tank. If the bottom of the tablet feed is sitting in water, the tablets will readily dissolve, causing an unnecessary and expensive overdose of chemical. The tablets should be out of the water until the flow from the sand filter underdrain enters the tablet feeder and the channel should drain down completely.

8.8 Post Aeration

Re-aeration may be necessary to ensure appropriate dissolved oxygen limits are maintained to meet water quality standards. Re-aeration can occur through turbulence over weirs and travel through pipes. However, to ensure reliability and compliance with NPDES permit limits for dissolved oxygen, cascade or diffused aeration may be necessary in warm months. Post aeration shall be located separately and downstream from the disinfection and dechlorination unit.

[23] *Sodium dioxide (gas) is not recommended for usage due to health and safety concerns.*

Chapter 9. Solids Handling

9.1 Introduction

Sludge production is an essential byproduct of wastewater treatment. Bacteria consume the organic material in wastewater and convert it into bacterial mass *(sludge)*. This process repeats until an excessive amount of bacteria populates the treatment system. The excess bacteria must be removed in order to maintain the proper environment in the treatment system so it will produce water that will meet effluent limitations. To facilitate the removal and storage of the removed bacterial mass, wastewater treatment plant designs must include sludge handling facilities onsite. All sludge handling facilities shall conform to requirements of OAC 3745-40.

9.2 Storage

Sludge storage tank design should be based on the expected sludge production of the treatment plant. This production should be based on the design influent organic concentration, the design influent hydraulic loading, and the anticipated concentration of thickened sludge. Sludge handling facilities shall consist of sludge storage, dewatering, and a means of wasting accumulated sludge. General design considerations are as follows:

A) All wastewater treatment plants shall provide for:

1) One hundred twenty days of biosolids storage[24] for the design capacity of the wastewater treatment plant, or

2) Contracted alternatives to facility storage of sewage sludge or biosolids. Wastewater treatment plants that do not adhere to this requirement must provide alternate means for sludge removal from the wastewater treatment plant *(i.e. periodically pumping sludge from tanks).*

[24] The one hundred twenty days of facility storage should be over and above the treatment capacity of the sewage sludge or biosolids treatment train. Units provided for storage should be dedicated for storage and not sewage sludge or biosolids treatment.

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Example 9.1: Liquid Sludge Holding Calculation

Assumptions:

ADF	=	20,000 gpd
Influent $cBOD_5$	=	175 mg/L
Yield	=	0.67 lbs. biomass / lb. $cBOD_5$
Thickened sludge concentration	=	20,000 mg/L (~ 2% solids)

Step 1: Calculate the influent $cBOD_5$ loading

Organic Loading = Q_{mgd} * $C_{mg/L}$ * 8.34 lbs./gal
= 0.020 mgd * 175 mg/L $cBOD_5$ * 8.34 lbs./gal
= 29 lbs. /day $cBOD_5$

Step 2: Determine the observed sludge yield *(Typical Y = 0.67 lbs. Biomass / lb. $cBOD_5$)*

Y_o = (0.67 lbs. biomass / lb. $cBOD_5$) *(29 lbs. / day $cBOD_5$)
= 19.5 lbs. / day biomass

Step 3: Determine thickened sludge volume per day

Sludge Volume = (lbs. biomass / day) / ($C_{mg/L}$ * 8.34 lbs./gal)
= (19.5 lbs. biomass / day) / (20,000 mg/L * 8.34 lbs./gal)
= 117 gal / day

Step 4: Determine sludge storage requirement for 120 days

Storage Volume = 117 gal/day * 120 days
= 14,000 gallons

Explanation: This is an example of a 120 day sludge storage facility. Alternately, a smaller sludge storage tank may be acceptable if a sludge hauling contract is provided. The contract hauler must be available to remove sludge on a more frequent basis.

B) The return sludge piping shall be valved so that waste activated sludge (WAS) can be diverted to the sludge storage tank.

C) If the sludge is to be further processed onsite, a sludge pump is required to transfer the sludge from the storage tank.

D) The sludge storage tank will include decant equipment so that waste sludge can be thickened and storage capacity can be increased. An overflow port is not acceptable.

E) Sufficient aeration for the sludge storage facility shall be designed to provide adequate mixing and maintain aerobic conditions when there is not a dedicated blower to the sludge storage facility.

Helpful Tip #24:

When a single blower is to provide aeration for process aeration as well as storage tank aeration, consideration must be given to adverse effects on treatment when the sludge storage tank water level is lower. With positive displacement blowers, air flow will tend to follow in the direction of lower head.

9.3 Disposal

A) Provision for ultimate disposal of sludge must be clearly indicated for all wastewater treatment plants.

B) Wasted sludge shall be hauled away for proper disposal, dewatered onsite using “dewatering bags”, sludge drying beds, or other means of dewatering. Filtrate shall be returned to the flow equalization basin or to the aeration tank if there is no flow equalization basin. An all-weather access road shall be provided to allow trucks access to the sludge holding tanks and drying beds.

C) The design and usage of sludge drying beds is not recommended because:

1) Sludge drying beds are cost prohibitive to provide 120 days storage capacity for small systems,

2) They are labor intensive to maintain; and

3) Difficult to operate during cold temperatures and wet weather.

However, if sludge drying beds are being proposed, multiple beds shall be provided. The design and construction shall provide at least nine (9) inches of sand over the gravel above the under drains. The underdrains shall direct the filtrate back to the flow equalization basin or to the aeration tank if there is no flow equalization basin.

D) All attempts shall be made to acquire sand with an effective size of 0.25 mm to 0.5 mm with a uniformity coefficient less than 4.0. [25]

E) The bottoms of the beds shall be provided with an impervious liner with a maximum permeability rate of 1.0 x 10^{-7} cm/s; acceptable materials include concrete, synthetic liner, or clay.

[25] *See A.8 in the appendix for a diagram of surface sand filter design*

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9.4 Septage

Domestic septage is defined as liquid or solid material removed from a septic tank, cesspool, portable toilet, type III marine sanitation device, or similar treatment works that receives only domestic sewage. Domestic septage does not include liquid or solid material removed from a septic tank, cesspool, or similar treatment works that receives either commercial wastewater or industrial wastewater and does not include grease removed from a grease trap at a restaurant. Commercial and industrial septage is not considered domestic septage.[26]

Table 9.1: Septage Rules & Regulations		
40 CFR, Part 503	US EPA	*Standards for Use or Disposal of Sewage Sludge*
OAC 3745-40	Ohio EPA	*Sewage Sludge*
ORC 6111	Ohio EPA	*Water Pollution Control*

Septage receiving at wastewater treatment plants with design flows of less than 100,000 gpd is not recommended and is discouraged due to the potential for exceeding the wastewater treatment plant's organic treatment capacity design and/or toxicity issues. However, if septage receiving is being considered, the following should be contemplated prior to accepting the waste:

A) Does the WWTP have hydraulic and organic treatment capacity to accept this additional loading?

B) Has the septage been screened prior to discharge into the WWTP? All septage being received shall be screened prior to discharge.

C) Where will the septage be introduced into the wastewater treatment plant? Due to the anaerobic nature of septage, it should be input at the raw sewage influent pump station; and

D) Has the septage been analyzed prior to acceptance *(i.e., pH, ammonia, for example)* to determine potential impacts of the septage to the wastewater treatment system?

[26] Restaurant grease traps; dump stations at campgrounds, RV parks, and marinas; or wastes from industrial facilities should not be mixed with domestic septage.

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Chapter 10. Flow Measurement

10.1 Introduction

Quantifying the volume of treated wastewater being discharged to the receiving stream is a requirement of all NPDES permits. Plants over 25,000 and up to 100,000 gpd capacity should have a recording or totalizing flow measuring device. Flow measurement devices are recommended on wastewater treatment plants under 25,000 gpd capacities.

10.2 Flow Measurement Devices

Flow measurement devices should be located so as to avoid recycle flows and loss of water in surface sand filters. Acceptable flow metering devices for wastewater treatment plants 25,000 gpd and above are:

A) Level Sensors with recording/totalizing capabilities
   1) Ultra-sonic
   2) Pressure Transducer

Acceptable flow metering devices for wastewater treatment plants less than 25,000 gpd are:

A) Level Sensors with recording/totalizing capabilities
   1) Ultra-sonic
   2) Pressure Transducer
B) Pump Run Time Meters *(WWTPs with dosing pumps)*
C) Bucket & Stop Watch *(For very small WWTPs only)*[27]
D) Water Use Records

10.3 Calibration

For wastewater treatment plants utilizing either ultra-sonic or pressure transducer flow measuring devices, an annual third party vendor/service provider should calibrate and certify operations of these flow measuring devices.

[27] ***Recording must be representative of flow that the WWTP sees.***

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For wastewater treatment plants that utilize pump run time meters, an annual draw down test shall be performed to determine the pumping rate of the dosing/effluent pump(s). If a dosing/effluent pump(s) is replaced, the same draw down test applies to determine the actual pumping rate.

Chapter 11. Treatment Plant Design Example

11.1 Introduction

This section will provide two examples of the step by step process of the design of a WWTP. The example problems will incorporate many of the recommendations and helpful tips that have been included throughout this document. One design will be for a smaller WWTP and one will be designed for a larger WWTP relative to this document.

11.2 Design of 5,000 gpd WWTP

The design example is for a rural elementary school:

Assumptions		
300 Students	=	4,500 gpd *(OAC 3745-42-05)*
18 Staff	=	360 gpd *(OAC 3745-42-05)*
ADF	=	5,000 gpd
cBOD5 Influent Concentration	=	210 mg/L
NH3-N Influent Concentration	=	42 mg/L
Alkalinity Influent Concentration	=	360 mg/L
Run Off Period	=	8 hours

Step 1: Determine Peak Hourly Design Flow

$$\text{Peak Factor} = \frac{3.33 \times 24\ (\text{hours})}{\text{Run-off period (hours)}}$$

$$= \frac{(3.33) \times (24\ \text{hours})}{8\ \text{hours}}$$

$$= 10$$

Peak Hourly Design Flow (gpd) = peak factor x average design flow

= 10 x 5,000 gpd

Peak Hourly Design Flow (gpd) = 50,000 gpd

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Step 2: Influent Pump Station Design

Determine Pump Station Pumping Rate *(gpm)*

Q_{pump} = Peak hourly design flow / 1 day
= 50,000 gpd / 1,440 min/day
= 34.7 gpm

Check Pumping Rate Impact on Clarifier

This example uses a flow equalization basin, so the influent pumping station has no effect on the clarifier.

Influent Pump Station = Two (2) 35 gpm pumps @ the required TDH.

Step 3: Flow Equalization Basin Sizing

Determine % of ADF *(See Table 5.1)*

ADF ≤ 40,000 gpd = 50%

Determine Flow Equalization Volume

= 5,000 gal * 0.50
= 2,500 gal

Min Flow Equalization Tank Volume = 2,500 gallons

Step 4: Aeration Tankage Sizing – "Method : 24 Hour Hydraulic Detention Time"

Step 1: Determine Total Aeration Tank Volume

$Volume_{total}$ = ADF (gpd) * 1 day / 7.48 gal/ft^3
= 5,000 gpd * 1day / 7.48 gal/ ft^3
= 668 ft^3

Step 2: Determine Number of Aeration Tanks Required

Number of Tanks = $Volume_{total}$ /$Volume_{AT}$
= 668 ft^3 / 334 ft^3 *(2,500 gallon tanks)*
= 2.0

Step 3: Determine Detention Time

$Volume_{total}$ = # of tanks * tank width * tank length * average water depth
= 2 Tanks* (334 ft^3/tank) * 7.48 gal/ft^3
= 4,996 gallons

Detention Time = Volume (gal) / flow (gpd) * 24 hrs.
= 4,996 gal / (5,000 gpd * 24 hrs.)

Detention Time = 24 hours

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Step 5: Organic Loading Design

Determine Influent Characteristics

ADF = 5,000 gpd
Organic Concentration: $cBOD_5$ = 210 mg/L or 8.8 lbs.

Determine Required Tank Volume in Thousand Cubic Feet (tcf)

Volume = (ADF / gal/ft^3 conversion factor) * (1tcf/1,000 ft^3)
= (5,000 gpd / 7.48 gal/ft^3)* (1tcf / 1,000 ft^3)
= 0.668 tcf

Determine Organic Loading Rate

Organic Loading Rate = ($cBOD_{5\ conc.}$) / 0.668 tcf
= (8.8 lbs) / 0.668 tcf

Organic Loading Rate = 13.2 lbs / 1,000 ft^3 ≤ 15 lbs. $cBOD_5$/day/tcf (Chapter 7, Section 7.3 D)

Step 6: Air Requirement Calculation

Assumption:

1.1 lbs. O_2 = lb. $cBOD_5$
4.6 lbs. O_2 = lb. NH_3-N
ADF = 5,000 gpd
Organic Concentration: $cBOD_5$ = 210 mg/L
Inorganic Concentration: NH_3-N = 42 mg/L
0.0173 O_2 = ft^3 air
Coarse Bubble Diffuser Efficiency = 4-6% @ 10 ft. submergence

Convert Concentration to Mass

$cBOD_5$ = 210 mg/L $cBOD_5$ * 0.005 MGD * 8.34 lbs./gal
= 8.8 lbs.

NH_3-N = 42 mg/L NH_3-N * 0.005 MGD* 8.34 lbs./gal
= 1.7 lbs. NH_3-N

Determine Oxygen Demand

$cBOD_5$ = 8.8 lbs. $cBOD_5$ * 1.1 lbs. O_2 / lb. $cBOD_5$
= 9.7 lbs. O_2

NH_3-N = 1.7 lbs. NH_3-N * 4.6 lbs. O_2 / lb. NH_3-N
= 7.8 lbs. O_2

Total O_2 = 9.7 lbs. O_2 + 7.8 lbs. O_2
= 17.5 lbs. O_2

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Determine Volume Air Required

$= 17.5 \text{ lbs. } O_2 / 0.0173\ O_2 / ft^3 \text{ air}$
$= 1{,}013\ ft^3\text{/day air}$
$= 1{,}013\ ft^3\text{/day air} / 0.05 \text{ efficiency}$
$= 20{,}200\ ft^3\text{/day}$

Determine Standard cfm

$= (25{,}300\ ft^3 \text{ / day}) / (1{,}440 \text{ min / day})$
$= 14 \text{ scfm}$

Required Air for Organic Treatment = 14 scfm (check air required for mixing)

Step 7: Clarifiers: Determine Surface Overflow Rate (SOR)

Assumptions:

ADF	=	5,000 gpd
Peak ADF	=	50,000 gpd
Forward Flow from Splitter Box	=	7,500 gpd *(assume 1.5 * ADF)*
Clarifier Dimension	=	14 ft. long x 6 ft. wide

Determine Surface Area for the Clarifiers

Surface Area = # of clarifiers * clarifier length * clarifier width
= (1) * (14 ft.) * (6 ft.)
= $84\ ft^2$

Determine Surface Overflow Rate @ Design Forward Flow Rate

Surface Overflow Rate = forward flow / surface area
= $7{,}500 \text{ gpd} / 84\ ft^2$
= $89 \text{ gpd}/ft^2$

Determine Flow at Maximum Acceptable Surface Overflow Rate

$SOR_{max} = 600 \text{ gpd}/ft^2$
$\text{Forward Flow}_{max} = 600 \text{ gpd}/ft^2 * 84\ ft^2$
$= 50{,}400 \text{ gpd or } 35 \text{ gpm}$

This clarifier would operate over a wide range of splitter box weir elevations regulating the forward flow.

Surface Overflow Rate = 89 gpd/ft² @ design forward flow

Step 8: Determine Solids Loading Rate (SLR)

Assumptions:

ADF	=	5,000 gpd
Peak ADF	=	50,400 gpd
Forward Flow from Splitter Box (Q_i)	=	7,500 gpd
Clarifier Dimension	=	14 ft. long x 6 ft. wide
# of Return Airlifts per Clarifier	=	2
Return Flow Rate per Airlift	=	3,600 gpd
Mixed Liquor Suspended Solids (MLSS)	=	2,500 mg/L

Determine $Q_i + Q_{rtotal}$

$$Q_i = 7{,}500 \text{ gpd}$$
$$Q_{rtotal} = (\text{\# clarifiers}) * (\text{\# of returns/clarifier}) * (\text{return flow rate})$$
$$= (1 \text{ clarifiers}) * (2 \text{ returns/clarifier}) * 3{,}600 \text{ gpd}$$
$$= 7{,}200 \text{ gpd}$$
$$Q_i + Q_{rtotal} = 7{,}500 \text{ gpd} + 7{,}200 \text{ gpd}$$
$$= 14{,}700 \text{ gpd}$$

Determine Surface Area for Clarifiers

$$\text{Surface Area} = (\text{\# clarifiers}) * (\text{clarifier length}) * (\text{clarifier width})$$
$$= (1)*(14\text{ft})*(6\text{ft})$$
$$= 84 \text{ ft}^2$$

Determine Solids Loading Rate

$$\text{Solids Loading Rate} = \frac{(Q_i + Q_{rtotal})*8.34 \text{ lbs./gal}*\text{MLSS}}{\text{surface area}}$$
$$= \frac{(0.0147 \text{ MGD})*8.34\text{lbs/gal}*2{,}500 \text{ mg/l}}{84 \text{ ft}^2}$$

Solids Loading Rate = 3.6 lbs./day/ft²

Step 9: ***Weir Overflow Rate***		
Assumptions:		
Forward Flow Rate	=	7,500 gpd
Number of Clarifiers	=	2
Width of each Clarifier	=	6 feet

Determine the effective length of the weirs. *(For Hopper Clarifiers, the weir trough has v-notch weirs on each side running the width of the clarifier)*

= # of clarifiers * clarifier width * 2 weir lengths
= (2 clarifiers) * (6 feet width) * (2 weir lengths)
= 24 feet

Determine the Weir Overflow Rate

Weir Overflow Rate = forward flow / total weir length
= 7,500 gpd / 24 feet

Weir Overflow Rate = 321.5 gpd/ft

Step 10: ***Sand Filter Design***		
Assumptions:		
Number of Beds	=	2
Forward Flow Rate	=	7,500 gpd
Sand Filter Area Criteria	=	12 gpd/ft^2

Determine Total Sand Filter Area

Total Area = Forward flow rate / Sand Filter Area Criteria
= 7,500 gpd / 12 gpd/ft^2
= 625 ft^2 *(25' x 25')*

A minimum of two beds are required. Each sand filter bed shall be 25' x 12.5'

Determine Dosing Pump Size

Pump = Filter loading rate * surface area of 1 bed
= 2.0 gpd/ft^2 * 312 ft^2
= ~ 625 gallons
= 625 gallons / 10 min pump cycle
= 62.5 gpm

Minimum Pumping Rate = Approximately 60 gpm

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Step 11: Chlorine Contact Tank Design		
Assumptions:		
Detention Time	=	15 minutes @ Peak Forward Flow
Peak Forward Flow	=	15,000 gpd *(Both EQ pumps pumping)*
Determine Chlorine Tank Volume		
$Volume_{TANK}$	=	peak forward flow * 1 day/1,440 minutes * detention time
	=	15,000 gpd * (1day/1,400 min) * 15 min
	=	156 gallon minimum or 20 ft^3
Determine Size of Tank		
$Volume_{TANK}$ Required	=	20 ft^3
24ft^3	=	3' x 4' x 2'
Detention time check	=	24 ft^3 *(7.48 gal/ ft3 / 15,000 gpd) * 1,440 min/day
	=	17.2 minutes
Minimum Tank Volume Required = 3ft. x 4ft. x 2ft.		

Step 12: Solids Holding Volume		
Assumptions:		
Influent $cBOD_5$ (C)	=	210 mg/L
Yield (Y_o)	=	0.67 lbs. biomass / lb. $cBOD_5$
Thickened sludge concentration (C_s)	=	20,000 mg/L (~ 2% solids)
Calculate the influent $cBOD_5$ loading		
Organic Loading	=	Q_{mgd} * C * 8.34 lbs./gal
	=	0.005 mgd * 210 mg/L $cBOD_5$ * 8.34 lbs./gal
	=	8.7 lbs./day $cBOD_5$
Determine the observed sludge yield *(Typical Y = 0.67 lbs. Biomass / lb. $cBOD_5$)*		
Y_o	=	(0.67 lbs. biomass/lb. $cBOD_5$) *(8.7 lbs./day $cBOD_5$)
	=	5.9 lbs. / day biomass
Determine thickened sludge volume per day		
$Volume_{sludge}$	=	(lbs. biomass/day) / (C_s * 8.34 lbs./gal)
	=	(5.9 lbs. biomass/day) / (20,000 mg/L * 8.34 mg/L)
	=	35 gal / day
Determine sludge storage requirement for 120 days		
$Volume_{storage}$	=	35 gal/day * 120 days
	=	4,200 gallons
Required Minimum Sludge Storage Volume = 4,200 gallons		

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11.3 Design of 84,000 gpd WWTP

The design example is for a village:

Assumptions		
225 homes	=	81,000 gpd *(Per Design Flow Rule – See Appendix A.8)*
Church w/ 225 seats	=	1,600 gpd
School – (490 students/55 employees)	=	1,850 gpd *(low flow fixtures 3.4 gal/person/day)*
ADF	=	84,000 gpd
cBOD5 Influent Concentration	=	230 mg/L
NH3-N Influent Concentration	=	45 mg/L
Run Off Period	=	16 hours
Number of Aeration Tanks	=	8
Conversion Factor	=	7.48 gal/ft^3

Step 1: Determine Peak Hourly Design Flow

$$\text{Peak Factor} = \frac{3.33 \times 24 \text{ hours}}{\text{Run-off period (hours)}}$$

$$= \frac{3.33 \times 24 \text{ hours}}{16 \text{ hours}}$$

$$= 5.0$$

Peak Hourly Design Flow (gpd) = peak factor x average daily flow

= 5.0 x 84,000 gpd

Peak hourly design flow (gpd) = 420,000 gpd

Step 2: Influent Pump Station Design

Assumptions:

Peak Hourly Design Flow = 420,000 gpd

Determine Pump Station Pumping Rate

Q_{pump} = Peak hourly design flow / 1 day

= 420,000 gpd/1,440 min / day

= 292 gpm

Check Pumping Rate Impact on Clarifier

This example uses a flow equalization basin so the influent pumping station has no effect on the clarifier.

Pump Station Design = Two (2) pumps at 300 gpm each at the required TDH

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Step 3: Flow Equalization Basin Sizing

Determine Flow Equalization Volume

$Volume_{EQ}$ = 33% * ADF *(See Table 5.1)*
= 33% * 84,000 gpd
$Volume_{EQ}$ = 27,720 gpd

Minimum Flow Equalization Basin Size = 27,720 gpd

Step 4: Aeration Tank Detention Time

Determine Aeration Tank Volume

$Volume_{total}$ = ADF (gpd) * 1 day / conversion factor (gal/ft^3)
= 84,000 gpd * 1 day / 7.48 gal / ft^3
= 11,230 ft^3

Number of Tanks = $Volume_{total}$ / $Volume_{tank(1)}$
= 11,230 ft^3 / 1,339 ft^3 (*tank volume 9 ft. x 16 ft. x 9.3 ft. SWD*)
= 8.4 tanks (*use 8 tanks total – 2 trains w/ 4 tanks per train*)
= 80,100 gallons

Determine Detention Time

Detention Time = (Volume (gal) / flow (gpd))* 24 hr./day.
= (80,100 gal / 84,000 gpd) * 24 hr./day.

Aeration Tank Detention Time = 22.9 hours

Step 5: Organic Loading Design

Determine Influent Characteristics

ADF = 84,000 gpd
Organic Concentration: $cBOD_5$ = 230 mg/L or 161 lbs.

Determine Required Tank Volume

Volume = ADF/ conversion factor (gal/ft^3) * (1 tcf/1,000 ft^3)
= (84,000 gpd/7.48 gal/ft^3)* (1 tcf/1,000 ft^3)
= 11.2 tcf

Determine Organic Loading Rate

Organic Loading Rate = $cBOD_{5\ conc.}$ / 11.2 tcf
= 161 lbs / 11.2 tcf

Organic Loading Rate = 14.4 lbs / 1,000 ft^3

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Step 6: Air Requirement Calculation

Assumption:

1.1 lbs. O_2	=	1 lb. $cBOD_5$
4.6 lbs. O_2	=	1 lb. NH_3-N
ADF	=	84,000 gpd
Organic Concentration: $cBOD_5$	=	230 mg/L
Inorganic Concentration: NH_3-N	=	45 mg/L
0.0173 O_2	=	1 ft^3 air
Course Bubble Diffuser Efficiency	=	4-6% @ 10 ft. submergence

Convert Concentration to Mass

$cBOD_5$ (lb/day) = $cBOD_5$ (mg/L) * ADF (MGD) * Conversion factor
= 230 mg/L $cBOD_5$ * 0.084 MGD * 8.34 lbs./gal
= 161 lbs.

NH_3-N (lb/day) = NH_3-N (mg/L) * ADF (MGD) * Conversion factor
= 45 mg/L NH_3-N * 0.084 MGD* 8.34 lbs./gal
= 31.5 lbs. NH_3-N

Determine Oxygen Demand

$cBOD_5$ = $cBOD_5$ (lbs/day) * 1.1 lbs. O_2 / lb. $cBOD_5$
= 161 lbs. $cBOD_5$ * 1.1 lbs. O_2 / lb. $cBOD_5$
= 177 lbs. O_2

NH_3-N = NH_3-N (lbs/day) * 4.6 lbs. O_2 / lb. NH_3-N
= 31.5 lbs. NH_3-N * 4.6 lbs. O_2 / lb. NH_3-N
= 145 lbs. O_2

Total O_2 Required = 177 lbs. O_2 + 145 lbs. O_2
= 322 lbs. O_2

Determine Volume Air Required

Air required = Total O_2 (lbs) / conversión factor (lb O_2 / ft^3 air) / efficiency
= 322 lbs. O_2 / 0.0173 O_2 / ft^3 air / 0.05
= 18,600 ft^3 air / day / 0.05 efficiency
= 372,000 ft^3 / day

Determine cfm

= Air required / 1 day
= (372,000 ft^3 / day) / (1,440 min / day)
= 258 cfm

Required Air for Organic Treatment = 258 scfm

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Step 7: Determine Number of Clarifiers

Assumptions:

ADF	=	84,000 gpd
Forward Flow from Splitter Box	=	130,000 gpd *(1 pump)*
Forward Flow from Splitter Box	=	260,000 gpd *(2 pump)*
Maximum Surface Overflow Rate	=	600 gpd/ft^2
Clarifier Dimension	=	16 ft. long x 9 ft. wide *(each)*

Determine Capacity per Clarifiers

Capacity = Surface Area * Maximum Surface Overflow Rate
= (16 ft. x 9 ft.) * 600 gpd/ft^2
= 86,400 gpd/clarifier

Determine Number of Clarifiers

Number of Clarifiers = Forward Flow *(1 pump)*/ Clarifier Capacity
= 130,000 gpd / 86,400 gpd/clarifier
= 1.5 (2 clarifiers required)

Number of Clarifiers = Forward Flow *(2 pump)* / Clarifier Capacity
= 260,000 gpd / 86,400 gpd/clarifier
= 3.0 clarifiers required

Note: A splitter box between the aeration tanks and the clarifiers will be required if the design uses 3 clarifiers for two treatment trains. Also, additional piping will be required for RAS and skimmers in order to evenly distribute the return activated sludge to 1 or both treatment trains. Such a design will provide optimal flexibility for the operator to match online aeration tanks and clarifiers to the organic and hydraulic loadings.

Alternately, the design could require 2 clarifiers per treatment train (4 clarifiers total). This will reduce the flexibility the operator has to match tankage and loadings.

Number of Clarifiers = 4 clarifiers – 2 clarifiers per train

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Step 8: Determine Solids Loading Rate (SLR)

Assumptions:

ADF	=	84,000 gpd
Forward Flow from Splitter Box (Q_i)	=	130,000 gpd
4 Clarifiers	=	16 ft. long x 9 ft. wide
# of Return Airlifts per Clarifier	=	2
Return Flow Rate per Airlift	=	2,500 gpd
Mixed Liquor Suspended Solids (MLSS)	=	3,500 mg/L

Determine $Q_i + Q_{rtotal}$

Q_i = 130,000 gpd

Q_{rtotal} = (# clarifiers) * (# of returns/clarifier) * (return flow rate)
= (4 clarifiers) * (2 returns/clarifier) * 2,500 gpd
= 20,000 gpd

Q_{Total} = 130,000 gpd + 20,000 gpd
= 150,000 gpd

Determine Surface Area for Clarifiers

Surface Area = (# clarifiers) * (clarifier length) * (clarifier width)
= (4) * (16 ft.) * (9 ft.)
= 576 ft^2

Determine Solids Loading Rate

$$\text{Solids Loading Rate} = \frac{(Q_i + Q_{rtotal}) * 8.34 \text{ lbs./gal} * \text{MLSS}}{\text{Surface Area}}$$

$$= \frac{(0.130 \text{ MGD} + 0.02 \text{ MGD}) * 8.34\text{lbs/gal} * 3,500 \text{ mg/l}}{576 \text{ ft}^2}$$

Solids Loading Rate = 7.6 lbs./day/ft² (1 pump) or 14.2 lbs./day/ft² (2 pumps)

Step 9: Weir Overflow Rate

Assumptions:

Peak Forward Flow Rate	=	130,000 gpd
Number of Clarifiers	=	4
Width of each Clarifier	=	9 feet

Determine the effective length of the weirs. *(For Hopper Clarifiers, the weir trough has v-notch weirs on each side running the width of the clarifier)*

Effective Length = # of clarifiers * clarifier width * 2 weir lengths
= (4 clarifiers) * (9 feet width) * (2 weir lengths)
= 72 feet

Determine the Weir Overflow Rate

Weir Overflow Rate	=	peak forward flow / total weir length
	=	130,000 gpd / 72 feet

Weir Overflow Rate = 1,806 gpd/ft. (1 pump) or 3,612 gpd/ft. (2 pumps)

Step 10: UFFM Design

Assumptions:

Forward Flow Rate	=	130,000 gpd
UFFM Design Criteria	=	600 gpd/ft^2 *(not to exceed)*

Determine Total Filter Area

Total Area	=	Forward flow rate / UFFM design criteria
	=	130,000 gpd / 600 gpd/ft^2
	=	217 ft^2

Determine Total Number of Filters

Total	=	Total filter area / individual UFFM media area
	=	217 ft^2 / (8 ft. x 11 ft.)
	=	2.4 (3 UFFM would be required)

Number of UFFM = 3 units(1 pump) or 5 units (2 pumps)

Step 11: Sand Filter Design

Assumptions:

Forward Flow Rate	=	130,000 gpd
Hydraulic Loading Rate Criteria	=	28 gpd/ft^2 *(See Appendix 8.6)*
ADF	=	84,000 gpd

Determine Individual Filter Bed Size

Total Area	=	ADF / hydraulic loading rate criteria
	=	84,000 gpd / 28 gpd/ft^2
	=	3,000 ft^2
Total Number of Filters	=	Total Area / Required Area per Filter
	=	3,000 ft^2 / 625 ft^2
	=	4.8 beds

Note: Five sand filters would be required if the design called for one distribution point per bed. If multiple distribution points discharge to a single bed, larger beds can be designed. In this case, two filters, each 25 ft x 60 ft, with two distribution lines per filter would be acceptable.

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Five sand filters at 25 ft. x 25 ft. are required (one distribution point per bed) or two sand filters at 25 ft. x 60 ft. (two distribution points per bed)

Determine Dosing Pump Size

Pump = filter loading rate * surface area of 1 bed
= $2.0\ gal/ft^2 * 1500\ ft^2$
= 3,000 gallons
= 3,000 gallons / 10 min pump cycle
= 300 gpm

Minimum pumping rate = 300 gpm

Step 12: Ultraviolent Radiation Disinfection Design

Assumptions:

Per the manufacturer's specifications
Minimum of 2 banks to adequately disinfect 260,000 gpd (two pumps pumping at the forward flow pumping rate)

Step 13: Solids Holding Volume

Assumptions:

Influent $cBOD_5$ (C) = 230 mg/L
Yield (Y_o) = 0.67 lbs. biomass / lb. $cBOD_5$
Thickened Sludge Concentration (C_s) = 20,000 mg/L (~ 2% solids)

Calculate the Influent $cBOD_5$ Loading

Organic Loading = Q_{mgd} * $cBOD_5$ (mg/L) * 8.34 lbs./gal
= 0.084 MGD * 230 mg/L $cBOD_5$ * 8.34 lbs./gal
= 161 lbs. / day $cBOD_5$

Determine the Observed Sludge Yield

Y_o = (0.67 lbs. biomass / lb. $cBOD_5$) * (161 lbs. / day $cBOD_5$)
= 108 lbs. / day biomass

Determine Thickened Sludge Volume/Day

Volume = (lbs. biomass / day) / (sludge concentration) * 8.34 lbs./gal
= (108 lbs. biomass / day) / (20,000 mg/L * 8.34 lbs./gal)
= 647 gal / day

Determine sludge storage requirement for 120 days

Storage Volume = 647 gal/day * 120 days
= 77,000 gallons

Required Sludge Storage Volume = 77,000 gallons

Appendix

A.1 Isolation Distances

Component of disposal system	Minimum isolation distance required from an occupied building	Minimum isolation distance required from surface waters of the state
Earthen impoundment that contains sewage or treated sewage	300 feet	300 feet
Earthen impoundment that contains industrial waste, other than industrial waste generated from the recovery of any natural resource, such as a quarry mining operation	300 feet	300 feet
Sewage sludge drying bed	300 feet	300 feet
Covered sand filter	100 feet	100 feet
Housing or building enclosure for extended aeration treatment works	100 feet	100 feet
Pump station	50 feet	35 feet
Any other component of a treatment works, not including (1) a disposal field, (2) a land application area or (3) a wet weather management facility for treating combined sewer overflows or sanitary sewer overflows	200 feet	300 feet

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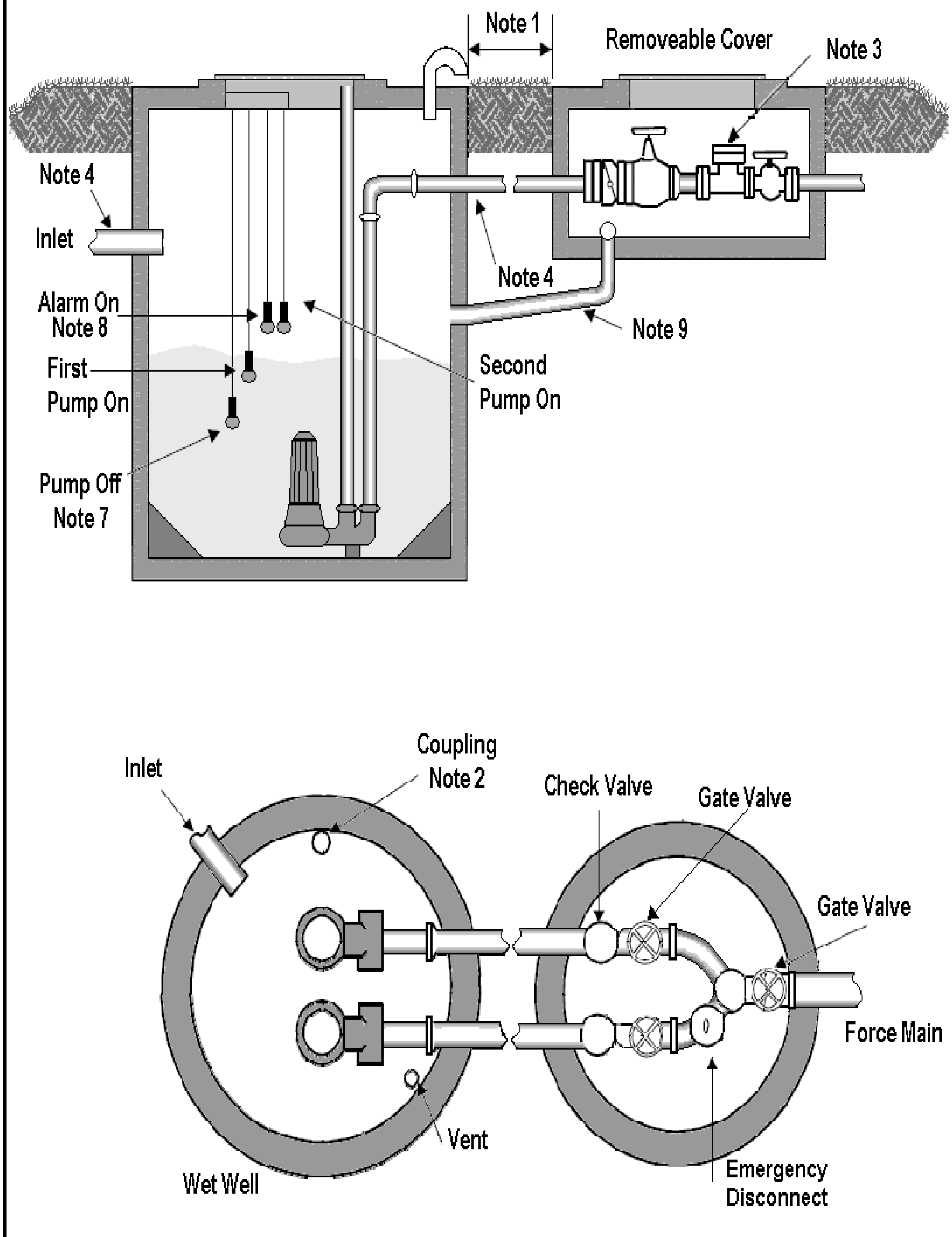
A.2 Submersible Pump Station



Notes:

Note 1.
Distance between wet well & vault to be determined in field

Note 2.
Control panel mounted outside of wet well (NEMA 3R if not in building)

Note 3.
Provide provisions for bypass pump connection

Note 4.
Provide gasketed watertight flexible connection.

Note 5.
Alarm-light and/or audio and/or telemetered case by case.

Note 6.
Provide hoist to remove pump

Note 7.
Maintain "pump off" switch above motor unless explosion proof motors are installed.

Note 8.
Do not set "alarm on" switch above "second pump on" switch.

Note 9.
Provide air seal with check valve or other automatic closing valve without using a manual valve or liquid seal.

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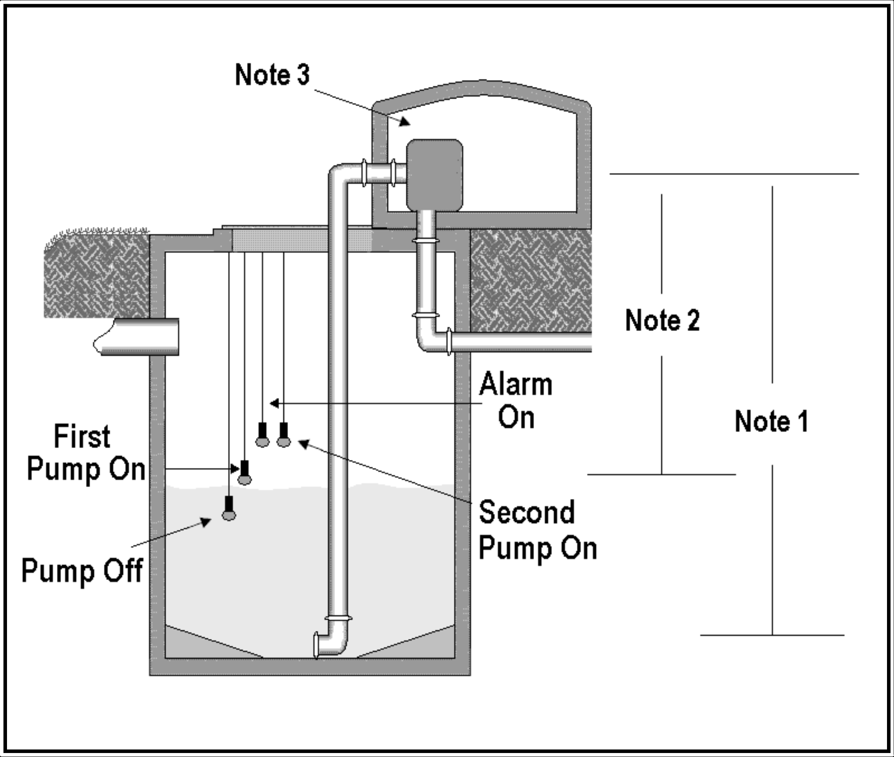
A.3 Suction Lift Pump Station



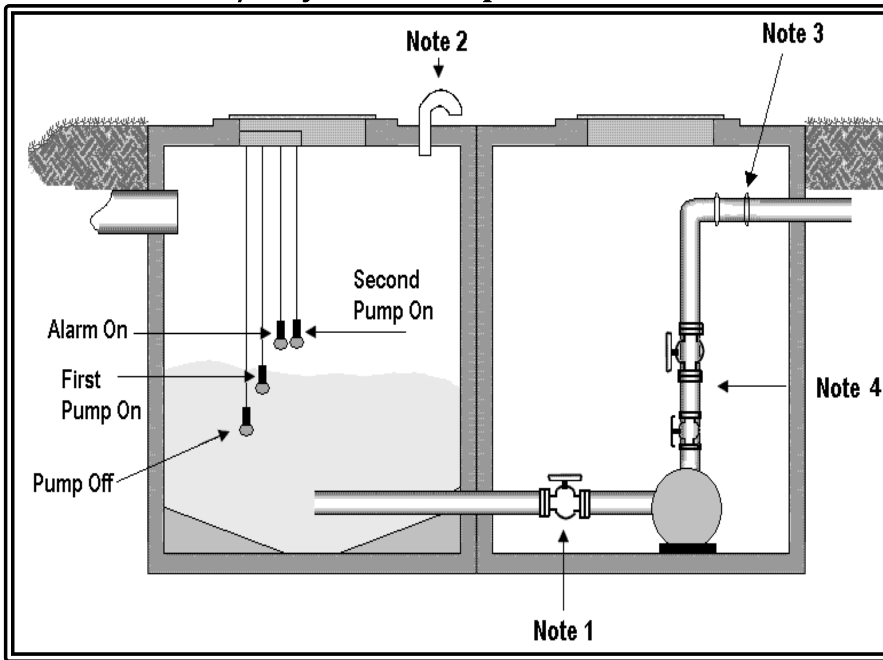
Notes:

Note 1: Piping should not exceed the size of the pump suction & not exceed 25 ft in total length.

Note 2: Combined total of dynamic suction-lift at pump off elevation & required net positive suction head at design operating conditions shall not exceed 22 ft.

Note 3: The pump equipment shall be above grade or offset & shall be effectively isolated from the wet well to prevent hazardous & corrosive sewer atmosphere from entering the compartment.

A.4 Wet Well / Dry Well Pump Station



Notes:

Note 1: Shut off valve shall be placed on the suction line of dry pit pump

Note 2: Covered wet wells shall have provisions for air displacement to the atmosphere, such as an inverted "J" tube

Note 3: Dry wells below grade shall be required to have mechanical ventilation

Note 4: Shutoff and check valves shall be placed on the discharge line of dry well.

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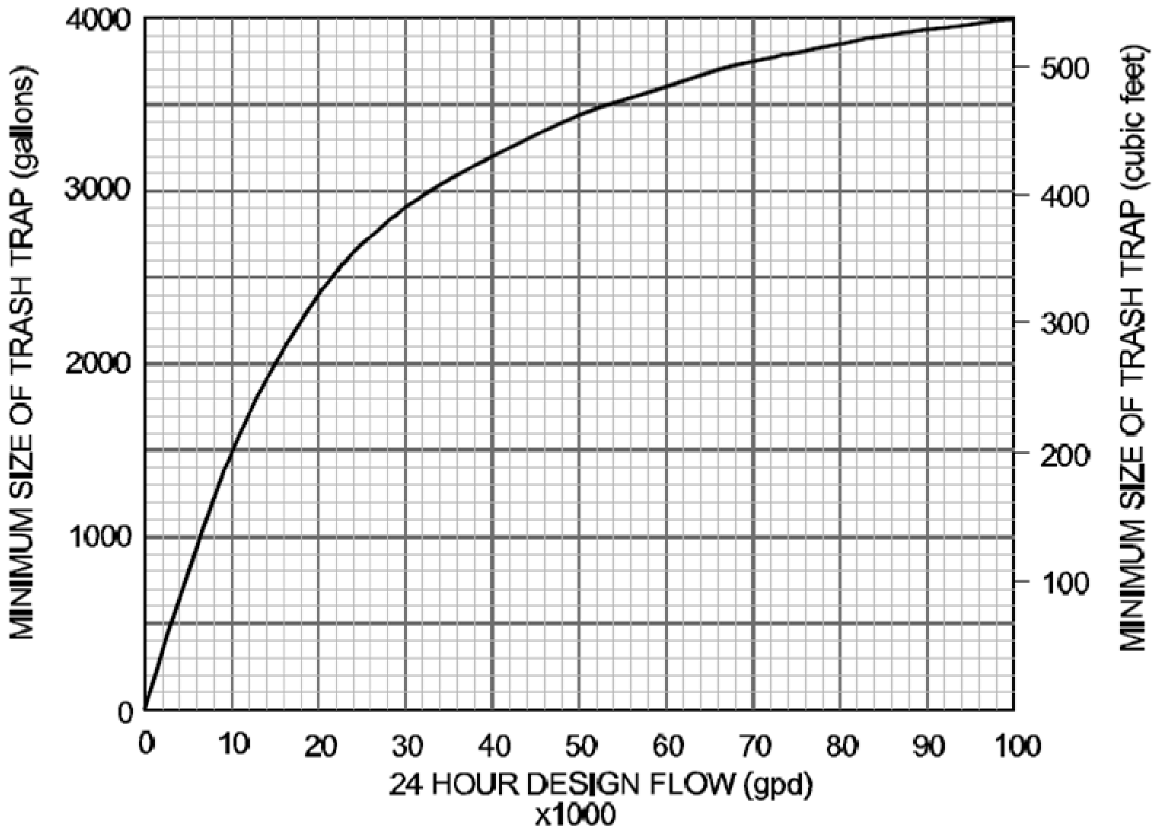
A.5 Trash Trap Sizing



24 HOUR DESIGN FLOW (gpd)	MINIMUM SIZE OF TRASH TRAP (gallons)	(cubic feet)	24 HOUR DESIGN FLOW (gpd)	MINIMUM SIZE OF TRASH TRAP (gallons)	(cubic feet)
1,000	150	20	10,000	1,500	200
2,000	300	40	20,000	2,400	321
3,000	450	60	30,000	2,900	388
4,000	600	80	40,000	3,200	428
5,000	750	100	50,000	3,430	459
6,000	900	120	60,000	3,600	481
7,000	1,050	140	70,000	3,740	500
8,000	1,200	160	80,000	3,840	513
9,000	1,350	180	90,000	3,920	524
10,000	1,500	200	100,000	4,000	535

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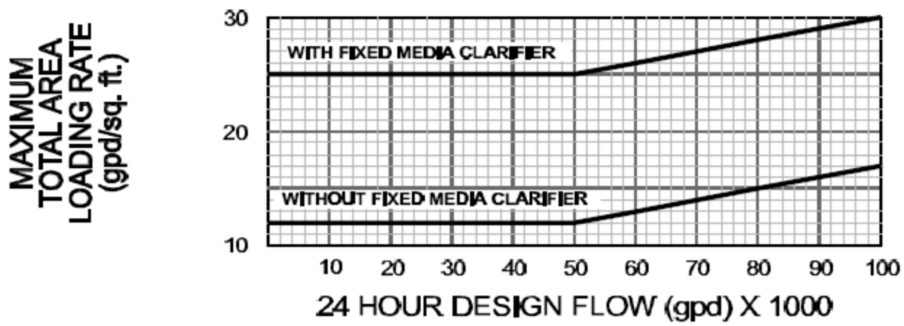
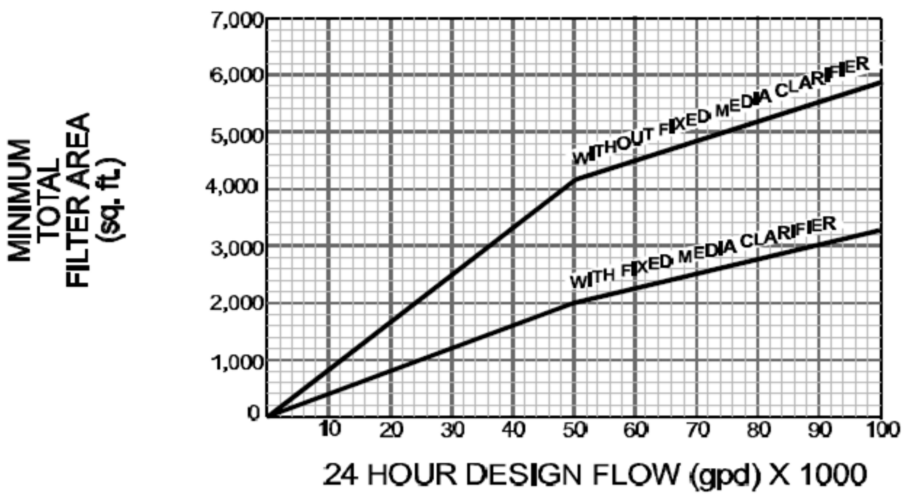
A.6 Slow Sand Filters – Loading Rate & Area





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24 HOUR DESIGN FLOW (gpd)	WITH FIXED MEDIA CLARIFIER		WITHOUT FIXED MEDIA CLARIFIER	
	LOADING RATE (gpd/sq. ft.)	MINIMUM SURFACE AREA (sq. ft.)	LOADING RATE (gpd/sq. ft.)	MINIMUM SURFACE AREA (sq. ft.)
10,000	25	400	12	833
20,000	25	800	12	1,667
30,000	25	1,200	12	2,500
40,000	25	1,600	12	3,333
50,000	25	2,000	12	4,167
60,000	26	2,308	13	4,615
70,000	27	2,593	14	5,000
80,000	28	2,857	15	5,333
90,000	29	3,103	16	5,625
100,000	30	3,333	17	5,882

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A.7 Design Flow Requirements

Place	Notes	Design flow (gpd)	Waste strength BOD_5 (mg/l)
Airport	b, i, j, p, r, t	15 / employee 4 / parking space	200 to 280 [r, s, t]
Apartment	b, l	120 / bedroom	200 to 280 [r, s, t]
Assembly hall	a, i, j	15 / employee 3 / seat w/o kitchen facilities 7 / seat w/ kitchen facilities	200 to 280 [r, s, t]
Banquet hall	b i, j	15 / employee 3 / seat w/o kitchen facilities 7 / seat w/ kitchen facilities	400
Barber shop	i, j	80 / basin	200 to 280 [s]
Beauty shop, styling salon	i, j	200 / basin	200 to 280 [s]
Bowling alley	a, i, j, p	75 / lane	200 to 280 [r, s, t]
Car wash	i, u	Sewer Connection Required	
Campground or recreational park	a, i, j, m, n, p	30 / primitive camp site w/o showers; 60 / primitive camp site w/ showers; 60 / camp site w/o water hook-up; 90 / camp site w/ water hook-up	200 to 280 [r, s, t]
Church (less than 200 sanctuary seats)	a, h, j, k, o, p	3 / sanctuary seat w/o kitchen; 5 / sanctuary seat w/ kitchen	200 to 280 [r, s, t]
Church (greater than 200 sanctuary seats)	b h, j, k, o, p	5 / sanctuary seat w/o kitchen; 7 / sanctuary seat w/ kitchen	200 to 280 [r, s, t]
Coffee shop	a i, j	15 / employee 5 / seat	200 to 280 [r, s, t]
Convenience store (facility with gas sales must be designed for a minimum of 500 gallons / day)	a, d, i, j, p, q	15 / employee 5 / parking space 500 / pump island	200 to 280 [r, s, t]
Country, Sportsman or Gun Club	b i, j, m, n, o, p	50 / member	200 to 280 [r, s, t]
Dance hall	a, i, j, p	15 / employee 3 / patron w/o kitchen facilities 7 / patron w/ kitchen facilities	200 to 280 [r, s, t]
Daycare facility	a, i, j, p	35 / employee 10 / student	200 to 280 [r, s, t]
Dentist/Doctor office	i	35 / employee 10 / patient 75 / dentist or doctor	200 to 280 [s]

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Dry cleaner	i	Contact Ohio EPA District Office	200 to 280 [s]
Factory	i, q	25 / employee w/o showers; 35 / employee w/ showers	200 to 280 [r, s, t]
Food service operation/restaurant categories (as noted below)			
Ordinary restaurant (not 24 hrs)	c, i, j, p	35 / seat	400 to 600
24 hour restaurant	c, i, j, p	60 / seat	400 to 600
Restaurant along freeway	c, i, j, p	100 / seat	400 to 600
Tavern (very little food service)	c, i, j, p	35 / seat	400 to 600
Bar (full food service)	c, i, j, p	35 / seat	400 to 600
Curb service (drive-in)	c, i, j, p	40 / car space	400 to 600
Vending machine	c, i, j, p	100 / machine	400 to 600
Homes in subdivision	b, l	120 / bedroom	200 to 280 [r, s]
Hospital	b, i, j, p	300 / bed 35 / employee	200 to 280 [r, s, t]
Hotel or motel	a, i, j, p	100 / room	200 to 280 [r, s, t]
Institution (psychiatric hospitals, prisons, etc.)	b, i, j, p	100 / bed 35 / employee	300
Laundromat	i, q	15 / employee 400 / machine	200 to 280 [s]
Marina (restrooms & showers only)	a, i	20 / boat mooring or slip	200 to 280 [r, s, t]
Migrant labor camp	e, i, j, p	50 / employee	200 to 280 [r, s, t]
Mobile home park	b, i, j, p	300 / mobile home space	200 to 280 [r, s, t]
Nursing and rest homes	b, i, j, p	200 / bed 100 / resident employee 50 / non-resident employee	300
Office building	a, i, j, k	20 / employee	200 to 280 [r, s, t]
Playground or day park	a, i, k, p	15 / employee 12 / parking space	200 to 280 [s]
Retail store	a, i, j, p	15 / employee 12 / parking space	200 to 280 [r, s, t]
School	b, i, j, k, p, t	15 / employee 15 / pupil for elementary schools; 20 / pupil for junior and high schools; 85 / pupil for boarding schools	200 to 280 [r, s, t]
Service station or gas station	a, i, q	500 / pump island; 500 / service bay; minimum of 750	200 to 280 [r, s, t]

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Shopping center	a, f, l, p, q	15 / employee 2 / parking space w/o food service 5 / parking space w/ food service	200 to 280 [r, s, t]
Swimming pool	a, i, m, n	5 / swimmer w/o hot showers 10 / swimmer w/ hot showers	200 to 280 [r, s, t]
Theater	a, i, j, p	5 / seat for indoor auditorium 10 / car for drive-in	200 to 280 [r, s, t]
Vacation cottage	b, i, j, p	50 / person w/o kitchen 75 / person w/ kitchen	200 to 280 [r, s, t]
Veterinarian office & animal hospital	f, i, j	15 / employee 100 / doctor 20 / run and cage	200 to 280 [r, s, t]
Youth and recreation camps	b, i, j, p	15 / employee for day camp 15 / camper for day camp w/ food 10 / camper for day camp w/o food 50 / employee for overnight camp 50 / camper for overnight camp	200 to 280 [r, s, t]

Notes:

Note a: Food service waste not included.

Note b: Food service waste included, but without garbage grinders.

Note c: Aeration tanks for these systems require forty-eight hour detention periods. Garbage grinders not permitted.

Note d: Truck parking areas will require consideration for treatment of runoff at large truck stops.

Note e: Twenty gallons per day of a vault latrine is used for toilet wastes.

Note f: Assume manual hosing of dog runs and solids (food droppings, etc.) removal prior to hosing.

Note g: Year round disinfection of all wastewater may be required before discharge to waters of the state or to any other surface or subsurface disposal systems.

Note h: Lower per seat estimate assumes a max of 1 church service per day, higher per seat estimate assumes a max of 3 church services per day. Weddings &funerals shall be counted as services.

Note i: Non-domestic or industrial wastes are prohibited from being discharged to soil based treatment systems.

Note j: Total capacity for number of persons should be confirmed by occupancy license or total occupancy capacity.

Note k: Higher flows shall be estimated when showers are available.

Note l: Deviating from this estimated design flow will require the director's approval, prior to applicant submitting the permit to install.

Note m: Pools cannot discharge pool filter backwash into soil based treatment systems.

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Note n:	Pool de-watering is prohibited from discharging to soil based treatment systems.
Note o:	Flow estimates do not consider daycare facilities. If a daycare is present, the flow requirements for a daycare facility must be included.
Note p:	An external grease trap is required for facilities with food service for OSTS.
Note q:	Assume 1 working shift of not more than eight hours. Assume higher flows for two or 3 shift operations.
Note r:	Assume no garbage grinder and normal domestic waste. If garbage grinders are present, the waste strength should be increased from twenty to sixty-five per cent.
Note s:	Data for regular strength waste range of 200 to 280 milligrams per liter was obtained from U.S. EPA's manual "Onsite Wastewater Treatment Systems Manual, February 2002 (EPA/625/R-00/008)."
Note t:	Waste strength should be twenty to 26 per cent higher for facilities that include food service operations, such as cafeterias, service stations & for facilities that may handle pet wastes.
Note u:	Sewer connection is required for a car wash. Please contact your district office.

Note: For additional information, refer to OAC 3745-42-05

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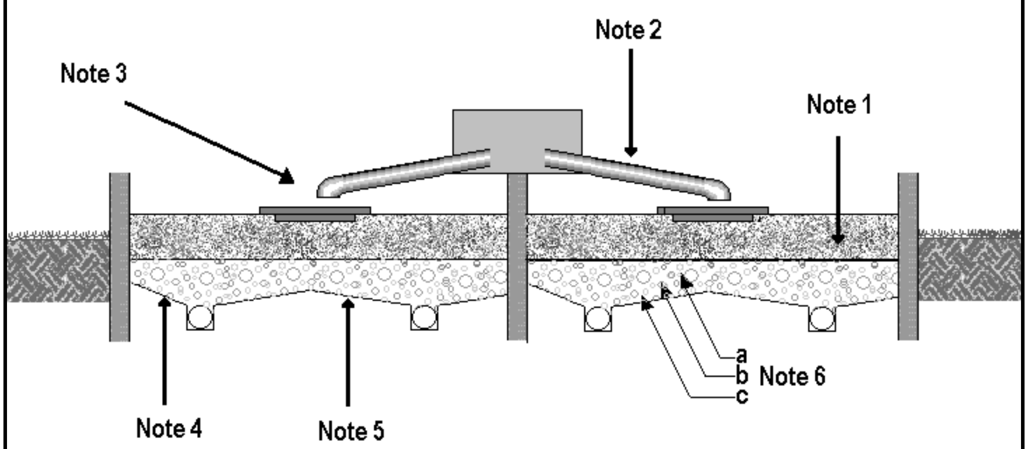
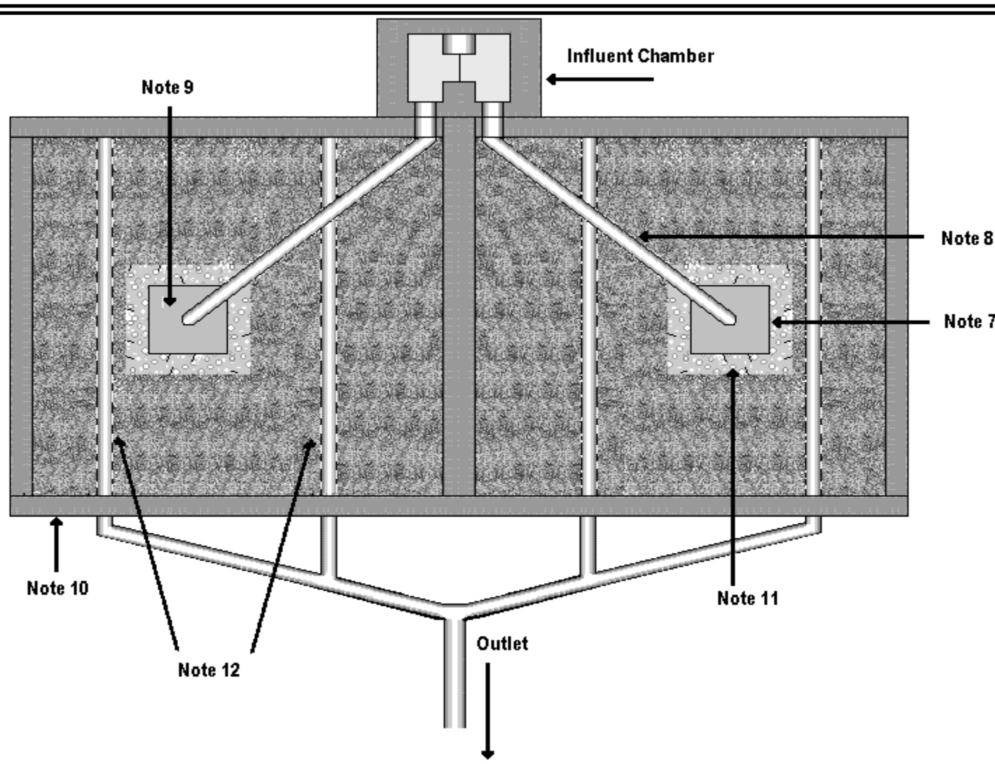
A.8 Surface Sand Filters





Notes:

Note 1:
18" filter sand

Note 2:
Distribution lines shall be adequately supported.

Note 3:
Down turned elbow suspended 1" above splash slab or serrated edge of down turn elbow.

Note 4:
Impervious Liner

Note 5:
Slope 1: 1

Note 6:
a) *3" of 1/8" to ¾" gravel*
b) *3" of 1/4" to ¾" gravel*
c) *3" min of ¾" gravel to 1.5" gravel*

Note 7:
Splash Pad - 3' x 3'
3' dimension may be reduced for smaller plants

Note 8:
6" rigid pipe minimum

Note 9:
Concrete splash pad

Note 10:
Both sides waterproof

Note 11:
Stone Rip rap (around splash pad only)

Note 12:
4" field tile "Under Drain"
Slope 1/8" to 1"

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A.9 Fats, Oils, & Grease Interceptor Tank Design

EPA Method #1

$$Size = D \times GL \times SF \times \frac{HR}{2} \times LF$$

D = *Number of seats in dining area*
GL = *5 gallons of wastewater per seat*
SF = *Storage capacity factor*
- *1.7 = single service kitchen*
- *2.5 = fully equipped commercial kitchen*

HR = *Number of hours open*
LF = *Loading factor:*
- *1.25 = locations adjacent interstate freeways*
- *1.00 = other freeways, recreational areas*
- *0.80 = main highways*
- *0.50 = other highways*

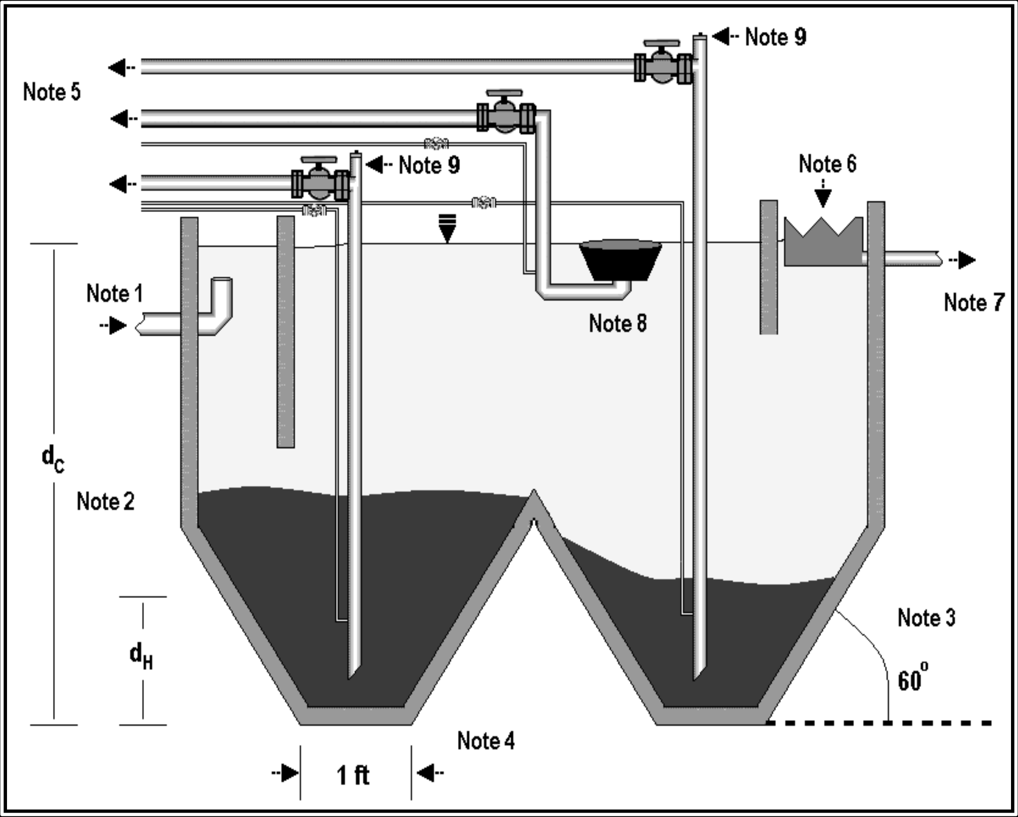
EPA Method #2

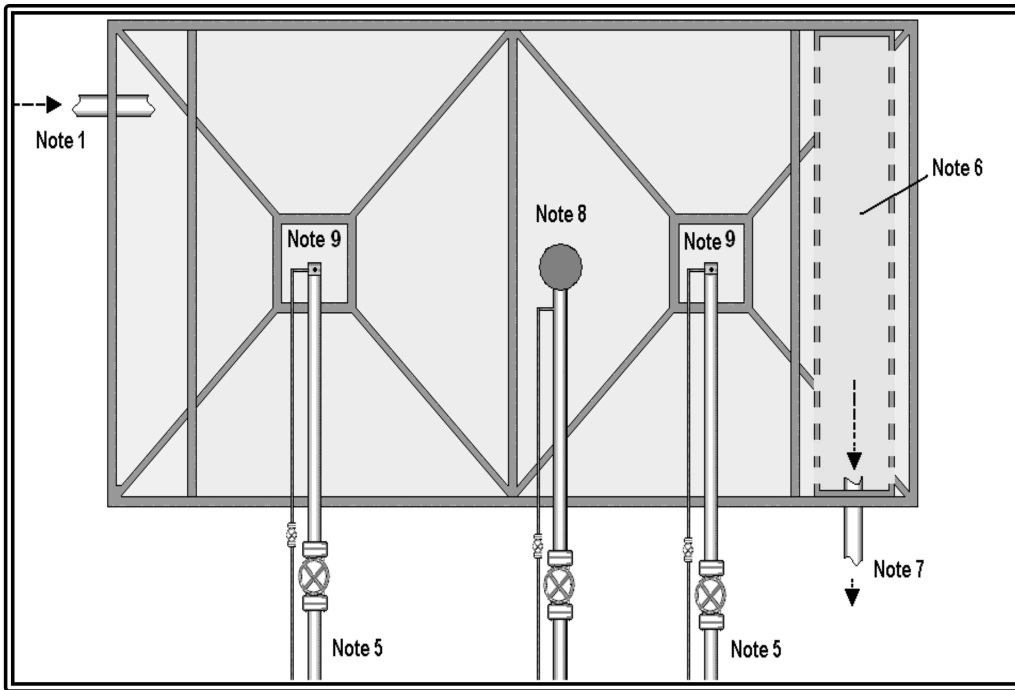
$$Size = M \times GL \times SF \times 2.5 \times LF$$

M = *Number of meals per day*
GL = *4.5 gallons of wastewater per meal*
SF = *Storage capacity factor*
- *1.7 = minimum*
- *2.5 = on site disposal*

LF = *Loading factor:*
- *1.25 = garbage disposal & dishwashing*
- *1.00 = w/o disposal*
- *0.80 = w/o dishwashing*
- *0.50 = w/o disposal & dishwashing*

A.10 Clarifier w/ dual Hopper bottom





Notes:

Note 1:

The transfer line from the aeration tank to the clarifier shall end in an upturned elbow to help dissipate velocity and momentum of the mixed liquor

Note 2:

D_c is the depth of the clarifier D_H is the depth of the hopper

Note 3:

Tank hoppers shall have a minimum side slope of 60 degrees to the horizontal

Note 4:

Hopper bottoms not in excess of one (1) foot square or one (1) foot in diameter

Note 5:

RAS lines - Minimum of 2" diameter pipe

Note 6:

Weir box

Note 7:

Effluent to 3^0 treatment

Note 8:

Skimmer

Note 9:

Cleanout(s)

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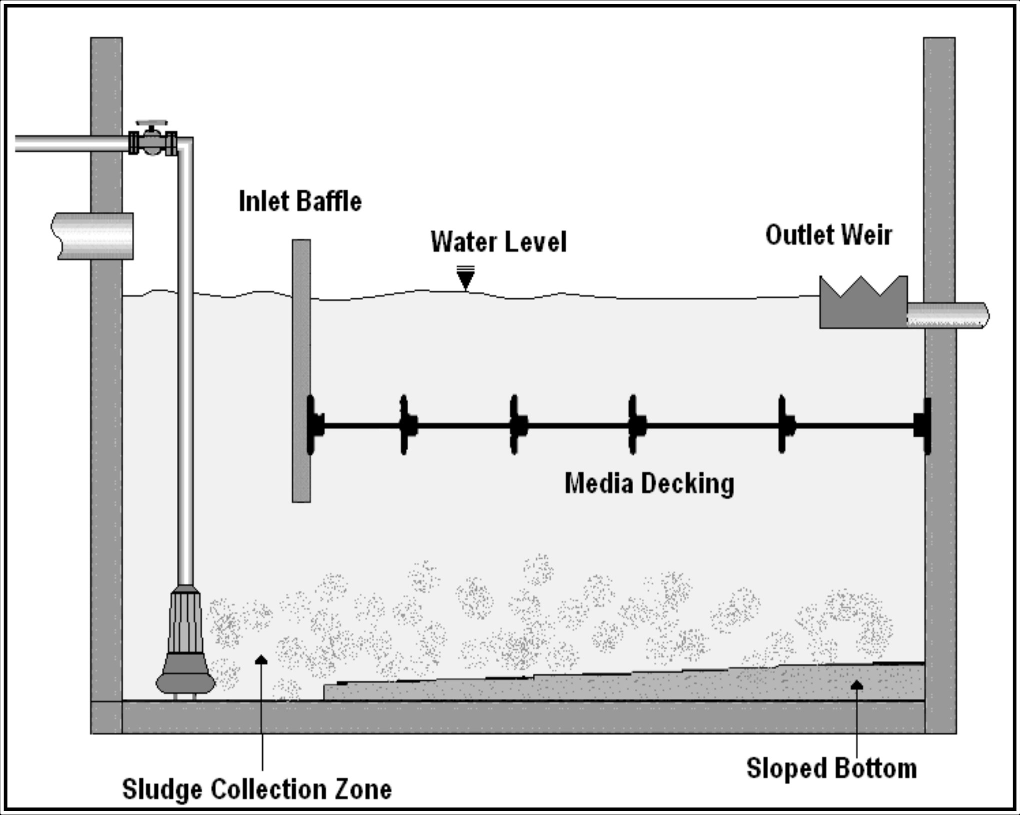
A.11 Up-Flow Fixed Media Clarifier



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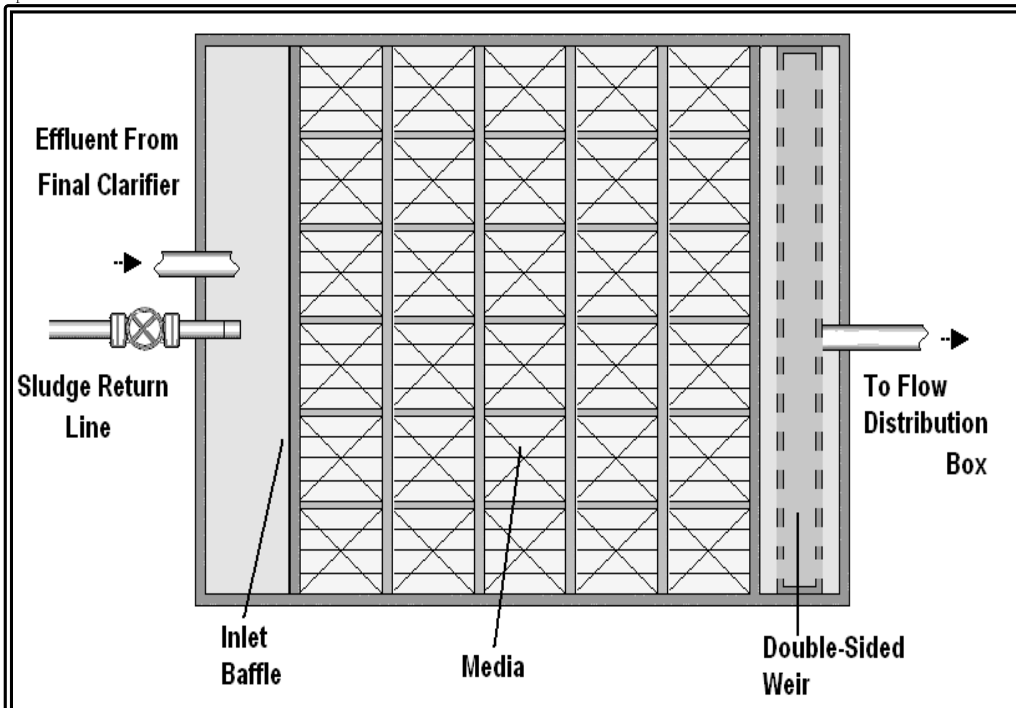

Notes:

Note 1:

UFFM clarifiers shall be no less than three (3) feet deep at the shallowest point and the base shall slope downward toward the sludge collection zone of the tank to facilitate sludge removal.

Note 2:

An inlet baffle shall be installed to direct all flow beneath the filter media to prevent short circuiting of the UFFM clarifier.

Note 3:

The number of in-service UFFM clarifiers shall accommodate the normal peak flow and all UFFM clarifiers in-service shall accommodate the maximum peak flow rate

Note 4:

Outlet weirs shall be double sided to minimize weir overflow rates.

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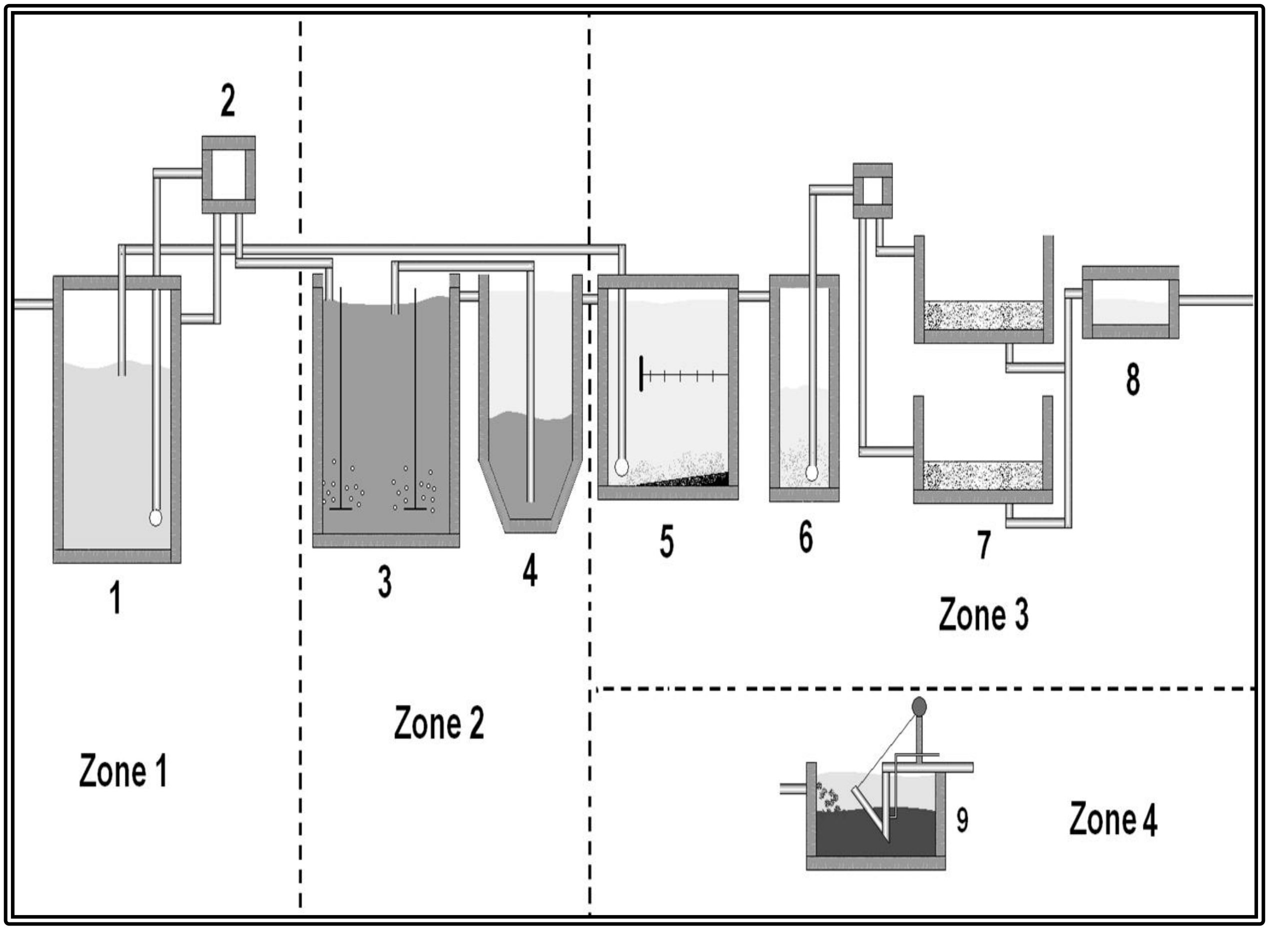
A.12 Design Influence



Zone 1: Influent & Preliminary Process	Zone 3: Tertiary Treatment Process
(See section 4.51 for additional details) *Unit 1: Flow Equalization Basin* *Unit 2: Splitter Box*	*(See section 4.53 for additional details)* *Unit 5: UFFM* *Unit 6: Dosing Chamber* *Unit 7: Sand Filter(s)* *Unit 8: Disinfection*
Zone 2: Secondary Treatment Process	**Zone 4: Solids Handling**
(See section 4.52 for additional details) *Unit 3: Aeration Tank(s)* *Unit 4: Final Clarifier(s)*	*(See section 4.54 for additional details)* *Unit 9: Solids Holding Tank*

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A.13 Ohio EPA District Offices

Ohio EPA District Offices

Central Office
P.O. Box 1049
Columbus, Ohio 43216-1049
(614) 644-3020

Central District Office
P.O. Box 1049
Columbus, Ohio 43216-1049
(614) 728-3778

Northeast District Office
2110 East Aurora Road
Twinsburg, Ohio 44087-1924
(330) 963-1200

Northwest District Office
347 North Dunbridge Road
Bowling Green, Ohio 43402-9398
(419) 352-8461

Southeast District Office
2195 Front Street
Logan, Ohio 43138-8637
(740) 385-8501

Southwest District Office
401 East Fifth Street
Dayton, Ohio 45402-2911
(937) 285-6357

Helpful Tips Index

Number	Location	Description
1	Chapter 3	Influent pump stations w/o Flow Equalization
2	Chapter 3	Influent sewer tributary to wet well
3	Chapter 3	Acceptable peak surface overflow rates
4	Chapter 3	Grinder pump blades
5	Chapter 4	Hydraulic & organic loading issues to consider
6	Chapter 5	High flow scenarios for flow equalization to consider
7	Chapter 5	Process air for treatment
8	Chapter 5	Adjustable overflow weirs
9	Chapter 5	Usage of sufficient data for equalization basin sizing
10	Chapter 7	Operational flexibility of aeration tanks
11	Chapter 7	Invert elevations of transfer ports on aeration tanks
12	Chapter 7	Matching influent loading w/ treatment capacity
13	Chapter 7	Dedicated blowers for each applicable treatment units
14	Chapter 7	Independent operation of blowers
15	Chapter 7	Acceptable peak surface overflow rates
16	Chapter 7	Solids Loading Rate (SLR) on final clarifiers
17	Chapter 7	Airlift pumps – alternative options
18	Chapter 7	Passive skimmer operations
19	Chapter 8	Proper sizing for UFFM.
20	Chapter 8	Maximizing sand filter operations
21	Chapter 8	Recommendations on sand filter designs
22	Chapter 8	Filter sand specification recommendations and considerations
23	Chapter 8	Tablet Chlorination/Dechlorination
24	Chapter 9	Sludge storage aeration considerations

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References

The following documents were used in the revision and development of this guidance document:

Biological Wastewater Treatment – Second Edition – **Grady, Daigger, & Lim**

Design Guidelines for Sewage Works – 2008 Edition - **Ontario Ministry of the Environment**

The Greenbook - Sewage: Collection, Treatment & Disposal Where Public Sewers are Not Available – 1993 Edition - **Ohio Environmental Protection Agency**

Orange Book - Criteria for Sewage Works Design – 2008 Edition – ***Washington State Department of Ecology***

Recommended Standards for Wastewater Treatment Facilities " Ten State Standards" – 2004 Edition - **Great Lakes-Upper Mississippi River Board of State & Provincial Public Health & Environmental Mangers**

Sand/Media Specifications: Rule Development Committee Issue Research Report – 2002, http://www.doh.wa.gov/Portals/1/Documents/Pubs/337-104.pdf - **Washington State Health Department, Wastewater Management Program**

TR-16: Guides for the Design of Wastewater Treatment Works – 1998 & 2011 Editions - **New England Interstate Water Pollution Control Commission**

Wastewater Engineering: Treatment, Disposal, & Reuse – Third Edition – **Metcalf & Eddy, Inc.**

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Acronyms & Abbreviations

Acronym / Abbreviation	Meaning
ADF	Average Daily Flow
ASTM	American Society for Testing and Materials
BOD_5	5-day Biochemical Oxygen Demand
$cBOD_5$	5-day Carbonaceous Biochemical Oxygen Demand
cfm	Cubic Feet per Minute
CFR	Code of Federal Regulations
cfs	Cubic Feet per Second
$C_{mg/L}$	Concentration Measured in Milligrams per Liter
FOG	Fats, Oils, and Grease
gpd	Gallons per Day
I/I	Inflow and Infiltration
mg/L	Milligrams/Liter
MGD	Million Gallons per Day
MLSS	Mixed Liquor Suspended Solids
NEMA	National Electrical Manufacturers Association
O&M	Operation and Maintenance
OLR	Organic Loading Rate
OSHA	Occupational Health and Safety Administration
OSTS	Onsite Sewage Treatment System
PERRP	Public Employment Risk Reduction Program
psi	Pounds per Square Inch
Q_{avg}	Average Design Flow
Q_{max}	Peak Average Design Flow
Q_{mgd}	Flow Measured in Million Gallons per Day
Q_{pump}	Flow through a Pump
RAS	Return Activated Sludge
SA	Surface Area
SCFM	Standard Cubic Feet per Minute
SLR	Solids Loading Rate
SOR	Surface Overflow Rate
tcf	thousand cubic feet
TSS	Total Suspended Solids
UFFM	Up-Flow Fixed Media Clarifier
UV	Ultraviolet radiation
WAS	Waste Activated Sludge
WOR	Weir Overflow Rate

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Glossary of Terms

This glossary defines and explains each term used in this manual. To the extent that the definitions and explanations provided in this glossary differ from those in EPA regulations or other official documents, the definitions used in this glossary are intended for use in understanding this manual only.

Ten States Standards **-** The Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (GLUMRB) in 1950 created a Water Supply Committee consisting of one associate from each state represented on the Board. A representative from the province of Ontario was added in 1978. The term state shall mean a representative state or the Province of Ontario.

208 Areawide Plan **-** A plan authorized under Section 208 of the Clean Water Act to develop a comprehensive program(s) for the treatment of water and for controlling water pollution from all point and non-point sources in the geographic area (*commonly known as areawide plans*). Initial plans were prepared in the 1970's and were a key product necessary for the operation of the construction grant program which provided federal funds for the design and construction of sewage collection and treatment facilities. The 208 plans are updated periodically by the responsible areawide planning agency or the State.

401/404 Requirements **-** Pursuant to the federal Clean Water Act, anyone who wishes to discharge dredged or fill material into the waters of the U.S., regardless of whether on private or public property, must obtain a Section 404 permit from the U.S. Army Corps of Engineers (Corps) and a Section 401 Water Quality Certification (WQC) from the state.

Airlift Return Sludge Pump **-** An airlift pump operates by introducing diffused air into the bottom of a vertical riser pipe in the liquid. This creates a reduction of the density of the liquid inside the pipe relative to the density of the liquid outside the pipe. This produces a pressure imbalance and the liquid rushes into the bottom of the riser pipe to reestablish equilibrium. The momentum of the incoming liquid carries it along the discharge pipe, thus generating the pumping action.

Antidegradation **-** Policies which ensure protection of water quality for a particular water body where the water quality exceeds levels necessary to protect fish and wildlife propagation and recreation on and in the water. This also includes special protection of waters designated as outstanding natural resource waters. Antidegradation plans are adopted by each state to minimize adverse effects on water.

Biosolids **-** See sewage sludge.

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Biochemical Oxygen Demand (5-day) (BOD$_5$) – This is an indirect measure of the concentration of biologically degradable material present in organic/inorganic wastes. It reflects the amount of oxygen consumed in 5 days by biological processes breaking down organic waste.

Clean Water Act - The common name for the Federal Water Pollution Control Act, Public Law 92-500; 33 U.S.C. section 1251 *et seq.*; the statutory authority for the NPDES Permit Program.

Corner Fillets - Easing of an interior corner of a tank to prevent solids accumulation.

Dewatering Bag – Bags, constructed of geotextile material, used to filter waste biosolids from a wastewater treatment plant. Waste sludge is pumped into these bags. Excess water passes through the geotextile material, therefore thickening and drying the biosolids. The filtrate must be returned to the headworks of the wastewater treatment plant and the thickened biosolids are disposed of in accordance with the facility's sludge management plan.

Domestic Wastewater - Also called sanitary wastewater, consists of wastewater discharged from residences and from commercial, institutional, and similar facilities.

Drop Manholes - A manhole that is installed in a sewer where the elevation of the incoming sewer is considerably above that of the outgoing sewer. A drop pipe is installed to transfer the sewage from a higher elevation to the invert of the manhole.

Filtrate - A liquid that has passed through a filter.

Floodplain - Land next to a river that becomes covered by water when the river overflows its banks.

Forward Flow – This is the flow from the splitter box that determines how much flow goes forward for treatment. This flow can be adjustable by altering the forward flow weirs in the splitter box.

Forward Flow Weir – An adjustable weir located in the splitter box. Its elevation, relative to the return flow weir, determines the forward flow rate.

Freeboard - The vertical distance between the normal high water elevation in a tank (or other structure) and the physical top of the tank (or other structure).

Hydraulic Loading - Hydraulic loading is volume of wastewater per day applied to a treatment unit, such as gallons per day per square foot.

Inflow and Infiltration - A term used to describe the penetration of clean water (not sewage) from the soil into sewers or other pipes through joints, connections, or manhole walls (infiltration). The direct entry of rainwater into a sewer system from sources other than infiltration, such as basement drains, down spouts, manhole covers, storm sewer cross connections, and storm drains (inflow).

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***Manning's Formula* -** An empirical equation used to estimate the average hydraulic conditions of gravity flow within a channel cross section, such as a pipe.

***Mixed Liquor* -** A mixture of microorganisms and sewage undergoing treatment in an aeration tank.

***National Pollutant Discharge Elimination System (NPDES)* -** A national program under Section 402 of the Clean Water Act for regulation of discharges of pollutants from point sources to waters of the United States.

***Organic Loading* –** The amount of organic material entering the wastewater treatment plant. Concentrations of $cBOD_5$ constitute the organic material and BOD_5 constitutes the concentrations of NH_3-N and $cBOD_5$.

***Peak Hourly Flow Rate* -** In a wastewater treatment plant, the highest flow sustained over an hours' period of time. This may include diurnal periods and periods of high rain fall or prolonged periods of wet weather.

***Polymer Feed* -** Polymer is used in the solids handling process to help thicken biosolids prior to storage and final disposal.

***Recycle Flow Weir* –** These weirs are located within the splitter box that directs flows that exceed the forward flow back to the flow equalization basin.

***Sewage Sludge* -** Any solid, semisolid, or liquid residue removed during the treatment of municipal wastewater or domestic sewage. Sewage sludge includes solids removed during primary, secondary, or advanced wastewater treatment, scum, septage, portable toilet pumpings, type III marine sanitation device pumpings (33 CFR part 159), and sewage sludge products. Sewage sludge does not include grit or screenings, or ash generated during the incineration of sewage sludge. Sewage sludge is also referred to as ***biosolids*.**

***Suspended Solids* -** Organic and inorganic particles (solids/sediment) in suspension in sewage that is undergoing treatment in a wastewater treatment plant.

***Up-Flow Fixed Media Clarifier (UFFM)* –** An intermediate treatment unit between the final clarifier(s) and tertiary sand filters. These units capture suspended solids that escape the final clarifier, which would otherwise end up on the sand filter.

***Wet Well* -** A wet well is a chamber with a pump that is used to collect wastewater and pump to downstream treatment units and/or processes.

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