

ONSITE NITROGEN REMOVAL

I. NITROGEN IN THE ENVIRONMENT

A. Chemistry of Nitrogen

Nitrogen can exist in nine various forms in the environment due to seven possible oxidation states (WEF, 1998):

<u>Nitrogen Compound</u>	<u>Formula</u>	<u>Oxidation State</u>
Organic nitrogen	Organic-N	-3
Ammonia	NH ₃	-3
Ammonium ion	NH ₄ ⁺	-3
Nitrogen gas	N ₂	0
Nitrous oxide	N ₂ O	+1
Nitric oxide	NO	+2
Nitrite ion	NO ₂ ⁻	+3
Nitrogen dioxide	NO ₂	+4
Nitrate ion	NO ₃ ⁻	+5

The principal forms of nitrogen of concern in onsite wastewater treatment and soil-groundwater interactions are Organic-N, NH₃/NH₄⁺, N₂, NO₂⁻, and NO₃⁻ (Rittman & McCarty, 2001; Sawyer *et al.*, 1994; US EPA, 1993). Because these forms still represent four possible oxidation states that can change in the environment, it is customary to express the various forms of nitrogen in terms of nitrogen rather than the specific chemical compound: Organic-N, NH₃-N, NH₄⁺-N, N₂-N, NO₂⁻-N, and NO₃⁻-N. Thus, for example, 10 mg/L of NO₃⁻-N is equivalent to 45 mg/L of NO₃⁻ ion.

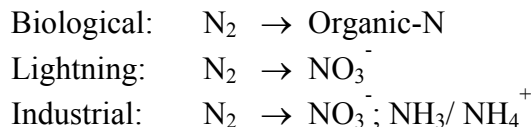
B. The Nitrogen Cycle in Soil-Groundwater Systems

As shown in Figure 1, transformation of the principal nitrogen compounds (Organic-N, NH₃-N, NH₄⁺-N, N₂-N, NO₂⁻-N, and NO₃⁻-N) can occur through several key mechanisms in the environment: fixation, ammonification, synthesis, nitrification, and denitrification (US EPA, 1993).

1. Nitrogen Fixation.

Nitrogen fixation is the conversion of nitrogen gas into nitrogen compounds that can be assimilated by plants. Biological fixation is the most common, but fixation can also occur by

lightning and through industrial processes:



2. Ammonification.

Ammonification is the biochemical degradation of organic-N into NH_3 or NH_4^+ by heterotrophic bacteria under aerobic or anaerobic conditions.



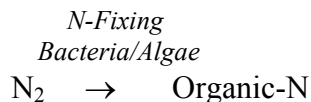
Some organic-N cannot be degraded and becomes part of the humus in soils.

3. Synthesis.

Synthesis is the biochemical mechanism in which NH_4^+ -N or NO_3^- -N is converted into plant protein (Organic-N):



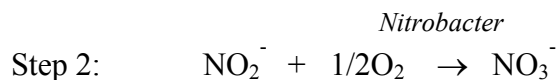
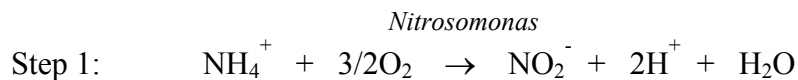
Nitrogen fixation is also a unique form of synthesis that can only be performed by nitrogen-fixing bacteria and algae (WEF, 1998):



4. Nitrification.

Nitrification is the biological oxidation of NH_4^+ to NO_3^- through a two-step autotrophic process by the bacteria *Nitrosomonas* and *Nitrobacter* (Rittman and McCarty, 2001; Sawyer, *et al.*,

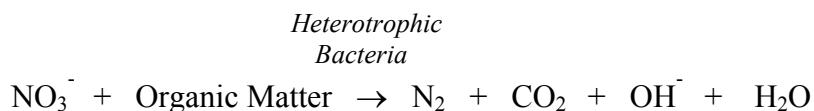
1994):



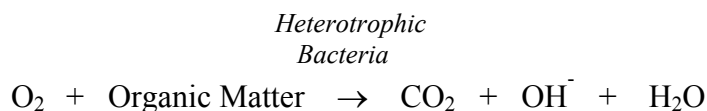
The two-step reactions are usually very rapid and hence it is rare to find nitrite levels higher than 1.0 mg/L in water (Sawyer, *et al.*, 1994). The nitrate formed by nitrification is, in the nitrogen cycle, used by plants as a nitrogen source (synthesis) or reduced to N₂ gas through the process of denitrification. Nitrate can, however, contaminate groundwater if it is not used for synthesis or reduced through denitrification as shown in Figure 1.

5. Denitrification.

NO₃⁻ can be reduced, under anoxic conditions, to N₂ gas through heterotrophic biological denitrification as shown in the following unbalanced equation (US EPA, 1993):

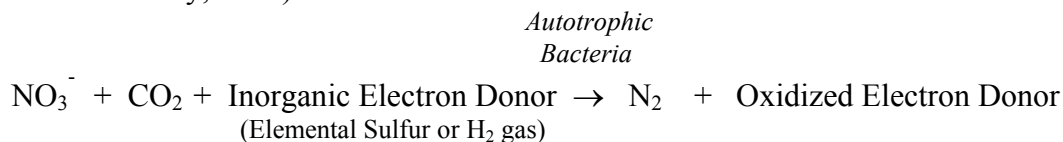


The above equation is identical to the equation for the biological oxidation of organic matter with the exception that NO₃⁻ is used as an electron acceptor instead of O₂:



A large variety of heterotrophic bacteria can use nitrate in lieu of oxygen for the degradation of organic matter under anoxic conditions. If O₂ is present, however, the bacteria will preferentially select it instead of NO₃⁻ (US EPA, 1993). Thus it is very important that anoxic conditions exist in order that NO₃⁻ will be used as the electron acceptor. A carbon source is required as the electron donor in the above equation for denitrification to occur.

Autotrophic denitrification is also possible with either elemental sulfur or hydrogen gas used as the electron donor by autotrophic bacteria as shown in the following unbalanced equation (Rittman and McCarty, 2001):



II. ENVIRONMENTAL EFFECTS OF NITROGEN DISCHARGES

A. Health Effects from Groundwater Contamination with Nitrates

Contamination of groundwater with nitrates is a problem in many parts of the U.S. and has been widely documented (Bouchard, *et al.*, 1992). Potential health concerns where contaminated groundwater is used as a drinking water source include methemoglobinemia, carcinogenesis, and birth defects.

1. Methemoglobinemia.

High nitrate levels in drinking water supplies can cause methemoglobinemia in infants, especially those less than six months old (Bouchard, *et al.*, 1992). After ingestion, nitrate is reduced to nitrite in the gut of the infant. The absorbed nitrite reacts with hemoglobin in the blood, forming methemoglobin. Methemoglobin, unlike hemoglobin, cannot carry oxygen. As more of the blood hemoglobin is converted to methemoglobin, the oxygen-carrying capacity of the blood is significantly reduced. Oxygen starvation of the blood can result in a bluish discoloration of the body, which is called "blue-baby" syndrome or methemoglobinemia. To prevent methemoglobinemia, the maximum contaminant level of nitrate in drinking water has been set at 10 mg/L as NO_3^- -N by the US EPA (Bouchard, *et al.*, 1992).

2. Carcinogenesis.

High nitrate levels in drinking water could potentially have carcinogenic effects through the formation of nitrosamines. Nitrates in the human body can be converted to nitrites and then to

nitrosamines, several forms of which have been classified as potential human carcinogens (Bouchard, *et al.*, 1992). While several scientific studies have shown a positive correlation between some types of cancers and nitrate intake in animals, a cause-effect relationship for risk of cancer has not yet been demonstrated conclusively.

3. Birth Defects.

Epidemiological studies in Canada and South Australia have shown a statistically significant increase in congenital malformations associated with nitrate-rich well water (Bouchard, *et al.*, 1992). These studies, however, are considered to be too limited in scope to deduce a causal association between birth defects and nitrate ingestion. Experimental animal studies have not shown significant effects from elevated nitrate ingestion.

B. Surface Water Pollution with Nitrogen

When excess nitrogen concentrations are discharged to surface waters, several deleterious effects may occur, depending on the environmental conditions.

1. Eutrophication.

Nitrogen is oftentimes the limiting nutrient for the growth of algae and aquatic plants in surface waters. Thus, excess nitrogen can cause the stimulation of growth, resulting in algal blooms or overgrowth of aquatic plants, which can have serious consequences for the receiving water such as odors, accumulation of unsightly biomass, dissolved oxygen depletion due to biomass decay, and loss of fish and shellfish.

2. Oxygen Demand through Nitrification.

The oxidation of Organic-N and $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ to $\text{NO}_3^-\text{-N}$ through the process of nitrification can exert a significant oxygen demand on the receiving water, which is known as the nitrogenous biochemical oxygen demand (NBOD) (Metcalf and Eddy, 1991). The NBOD of a wastewater can even be greater than the carbonaceous biochemical oxygen demand (CBOD), although it may not be exerted as rapidly. The rate of nitrification is dependent on several environmental factors, which include the population of nitrifying bacteria, temperature, alkalinity, and availability of dissolved oxygen.

3. Ammonia Toxicity to Aquatic Organisms.

Nitrogen in the form of $\text{NH}_3\text{-N}$ can cause acute toxicity to several species of fish. Because the concentration of $\text{NH}_3\text{-N}$ as opposed to $\text{NH}_4^+\text{-N}$ is pH dependent, criteria for ambient water quality have been set for unionized ammonia as a function of pH and temperature (Sawyer, *et al.*, 1994). Many municipal wastewater treatment plants in the US are required to nitrify their effluent in order to avoid ammonia toxicity in receiving waters.

B. Anthropogenic Sources of Nitrogen Discharges to Groundwater

1. Agricultural Activities.

Agricultural activities are a significant source of nitrate in groundwater. Nitrate can enter groundwater at elevated levels by excessive or inappropriate use of nitrogen-containing nutrient sources, which include commercial fertilizers, animal manures, and types of crops and cropping systems utilized.

Nitrogen fertilizer use increased five-fold during the period 1955-1988, and it is believed that misuse of nitrogen fertilizers is the most important source of nitrate contamination of groundwater in the US (Power and Schepers, 1989; Hallberg, 1989). Most nitrogen fertilizer is applied as anhydrous ammonia, urea, or as nitrate or ammonium salt. In an aerobic soil environment, much of the applied $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ can be transformed to the conservative anion NO_3^- , which readily migrates to groundwater through most soil types as a result of its negative charge. Under anoxic conditions in the presence of a carbon source, however, $\text{NO}_3^-\text{-N}$ can be reduced to atmospheric nitrogenous gases (N_2 , NO and N_2O) as will be discussed below.

Livestock and dairy practices that concentrate animals, such as feedlots, can also significantly contribute to nitrate contamination of groundwater if the animal wastes generated by the operation are not properly managed. Abandoned feedlots have been shown to pose a greater threat than active ones; this is due to the highly compacted soil in active lots, which prevents the movement of water and oxygen through the soil, inhibiting nitrification and leaching (Bouchard, *et al.*, 1992).

The types of crop and cropping system are also important in determining the potential for nitrate migration to groundwater (Bouchard, *et al.*, 1992). Irrigated agriculture on sandy soils, and

heavily fertilized, shallow-rooted crops, favor nitrate leaching. In animal production areas undercrediting nitrogen contributions from manure and leguminous forages often results in significant nitrogen loading to groundwater (Nowak, *et al.*, 1997).

2. Septic Tank-Soil Absorption Systems.

Contamination of groundwater with nitrates from septic tank-soil absorption systems is also a problem in many parts of the US. The build-up of nitrate in groundwater is one of the most significant long-term consequences of onsite wastewater disposal (Hantzsche and Finnemore, 1992).

As an example, the annual nitrogen contribution for a family of four from a conventional septic system on a quarter acre lot would be approximately 50 lbs. per year (Hantzsche and Finnemore, 1992). The annual nitrogen requirement for a quarter acre of Bermuda grass, much of which may be supplied by fertilizer, is also about 50 lbs. per year (WEF, 2001). The problem, however, is that the nitrogen from septic tank-soil absorption systems is not uniformly distributed throughout a lawn and is discharged at a depth below which plants can utilize it. Nitrogen primarily exists as Organic-N and $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ in septic tank effluent, and is usually transformed into nitrate as the wastewater percolates through the soil column beneath the system's drainfield. Also, the nitrogen loading from high housing densities can greatly exceed any potential plant uptake of nitrogen even if the effluent was properly applied, a common problem in various communities (Gold and Sims, 2000; County of Butte, 1998; Hantzsche and Finnemore, 1992).

C. Control of Nitrogen Discharges from Onsite Systems

As a result of the potential for nitrate groundwater contamination caused by septic-tank soil absorption systems, public health and water pollution control agencies have tried either to limit the number of onsite systems in a given area by quantifying nitrogen loadings (Hantzsche and Finnemore, 1992), or to examine alternative onsite technologies that provide nitrogen removal (Ayres Associates, 1998; California Regional Water Quality Control Board, 1997; Whitmeyer *et al.*, 1991).

D. Quantifying Nitrogen Loading Rates

1. Hantzsche-Finnemore Mass Balance Equation.

The Hantzsche-Finnemore equation estimates nitrate loadings to groundwater based upon the measured factors of rainfall, aquifer recharge, septic system nitrogen loadings, and denitrification (Hantzsche and Finnemore, 1992). The equation takes the following form:

$$n_r = \frac{I \cdot n_w \cdot (1-d) + R \cdot n_b}{(I + R)} \quad (1)$$

where n_r = final NO_3^- -N concentration in groundwater after mixing, mg/L
 I = volume of wastewater entering the soil averaged over the gross developed area, in/yr (m/yr)
 n_w = Total-N concentration of wastewater, mg/L
 d = fraction of NO_3^- -N lost to denitrification
 R = average recharge rate of rainfall, in/yr (m/yr)
 n_b = background NO_3^- -N concentration without wastewater discharge, mg/L

A critical simplifying assumption in equation 1 is that there is uniform and complete mixing of wastewater and percolating rainfall over the entire developed area, and that this is completed at the water table (Hantzsche and Finnemore, 1992). (This assumption obviously has many limitations in practice, since complete mixing of wastewater and rainfall is unlikely to occur. Nevertheless, the Hantzsche-Finnemore equation has been used with success as long as its limitations are recognized and the parameters are carefully monitored for given local conditions, as was done in Butte County, California (County of Butte, 1998).)

If the volume and Total-N concentration of wastewater applied over a development area can be estimated, along with the possible denitrification fraction, then the resultant concentration of nitrate in groundwater can be calculated if rainfall and recharge rates to the aquifer are known.

A common land use planning dilemma is the determination of acceptable housing densities based on nitrogen loadings. The number of gross acres per dwelling unit to ensure that groundwater NO_3^- -N will not exceed 10 mg/L can be calculated from the following equation:

$$A = \frac{0.01344W[n_w - d \cdot n_w - 10]}{R(10 - n_b)} \quad (2)$$

where A = gross acres/dwelling unit (gross m²/dwelling unit)
W = average daily wastewater flow per dwelling unit, gallons (m³/day)

EXAMPLE 1. USE OF THE HANTZCHE-FINNEMORE EQUATION. The Hantzsche-Finnemore Equation has been used by the California Regional Water Quality Control Board to control housing development in the Chico Urban Area, where groundwater has been contaminated with nitrates from septic systems (County of Butte, 1998). You are to determine the maximum concentration of dwelling units per acre to ensure NO₃⁻-N concentrations in groundwater do not exceed 10 mg/L. The following conditions are assumed to apply:

1. The per capita wastewater generation rate is 45 gpd.
2. There is an average of 2.4 residents per household in the Chico Urban Area.
3. The average rate of Total-N discharge per capita is 15 grams/day.
4. Approximately 20% of the Total-N generated is removed in the septic tank.
5. The fractional removal of NO₃⁻-N in the soil column through denitrification found through lysimeter studies is 0.30.
6. The annual recharge rate of the groundwater aquifer is 18 in./yr.
7. The background NO₃⁻-N concentration in groundwater is 0.1 mg/L.

Solution

1. Determine the concentration of Total-N in septic tank effluent.

$$n_w = \frac{(15 \text{ grams N/person-day})(1000 \text{ mg/gram})(0.8)}{(45 \text{ gal/person-day})(3.78 \text{ L/gal})} = 70 \text{ mg/L Total-N}$$

2. Calculate the average daily wastewater flow per dwelling unit (DU).

$$W = (45 \text{ gal/person-day})(2.4 \text{ persons/DU}) = 108 \text{ gal/DU.}$$

3. Determine the number of gross acres per dwelling unit.

$$A = \frac{0.0134(108 \text{ gal/DU})[(70 \text{ mg/L Total-N}) - (0.3)(70 \text{ mg/L}) - 10 \text{ mg/L}]}{(18 \text{ in/yr})(10 \text{ mg/L} - 0.1 \text{ mg/L})} = 0.32 \text{ acres/DU}$$

4. Determine the concentration of NO_3^- -N in groundwater, n_r , assuming 3 dwelling units/acre.

$$I = \frac{(45 \text{ gal/person-day})(2.4 \text{ persons/DU})(3 \text{ DU/acre})(365 \text{ days/yr})}{(43,560 \text{ ft}^3/\text{acre-ft})(7.48 \text{ gal/ft}^3)(1 \text{ ft}/12 \text{ in})} = 4.35 \text{ in/yr}$$

$$\therefore n_r = \frac{(4.35 \text{ in/yr})(70 \text{ mg/L})(1 - 0.3) + (18 \text{ in/yr})(0.1 \text{ mg/L})}{(4.35 \text{ in/yr} + 18 \text{ in/yr})} = 9.6 \text{ mg/L NO}_3^- \text{-N}$$

There are various other methods, some very complicated, that have been used to quantify nitrogen loadings. Most have been developed and used, however, for local conditions and have not been widely disseminated.

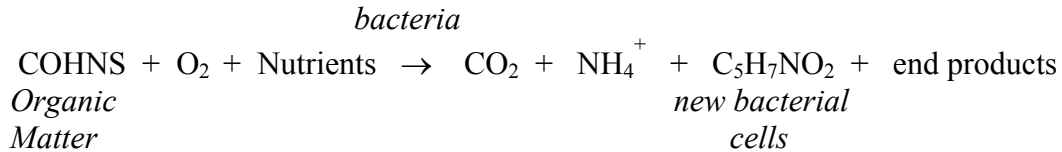
E. Nondegradation Legislation.

In order to protect groundwater quality, several states have promulgated "nondegradation" legislation that, in essence, prohibits any subsurface discharge of contaminants in concentrations higher than those found in groundwater aquifers. Examples of this legislation include the Porter-Cologne Act in California and the Nondegradation of Water Quality Administrative Rule in Montana (Department of Environmental Quality, 1995). Because the maximum contaminant level (MCL) of NO_3^- -N in drinking water supplies is 10 mg/L, there is a tendency among regulatory agencies to take the stance that Total-N in effluents from onsite nitrogen removal systems should not be greater than 10 mg/L. The available performance data from decentralized onsite nitrogen removal systems show, however, that Total-N concentrations of 10 mg/L or less can not be satisfied with any degree of reliability at the present time; this is not the case, however, with large-scale systems that are continuously monitored and receive regular operation and maintenance (US EPA, 1993). Thus, the protection of groundwater quality from onsite systems will likely continue to involve quantification of permissible nitrogen loading rates per unit area coupled with nitrogen removal technologies that will be required to satisfy some type of effluent performance standard. The balance of this module will focus on the theory, design, and performance of onsite nitrogen removal systems.

III. NITROGEN DYNAMICS IN SEPTIC TANK-SOIL ABSORPTION SYSTEMS

A. Wastewater Characteristics

The mass loading of nitrogen in domestic wastewater averages from 4 to 18 lbs. (1.8 to 8.1 kg) of Total-N per capita per year. Untreated domestic wastewater typically contains 20 to 85 mg/L Total-N, with the majority occurring as a mixture of $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ (12-50 mg/L) and Organic-N (8-35 mg/L) (Metcalf & Eddy, 1991). Because the carbon to nitrogen ratio of wastewater is typically on the order of 4:1 to 6:1, there will be excess nitrogen after secondary biological treatment (BOD removal) that cannot be assimilated by microorganisms as shown in the following unbalanced equation (Metcalf & Eddy, 1991):



It is for this reason that special attention must be given to nitrogen removal processes in wastewater engineering.

B. Septic Tanks

The removal of Total-N within septic tanks is on the order of 10 to 30%, with the majority being removed as particulate matter through sedimentation or flotation processes. Because of the septic tank's anaerobic environment, nitrogen exists principally as Organic-N and $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ (TKN). Organic-N is transformed to $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ via ammonification; although some $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ is converted to Organic-N via bacterial cell growth, there will be a net increase of $\text{NH}_3\text{-N}/\text{NH}_4^+\text{-N}$ in the effluent.

C. Subsurface Soil Absorption Trenches

Nitrogen can undergo several transformations within and below subsurface soil absorption trenches (Ayres Associates, 1993). These transformations include: 1) adsorption of NH_4^+ -N in the soil; 2) volatilization of NH_3 -N in alkaline soils at a pH above 8.0; 3) nitrification and subsequent movement of NO_3^- -N towards the groundwater; 4) biological uptake of both NH_3 -N/ NH_4^+ -N and NO_3^- -N; and 5) denitrification if the environmental conditions are appropriate.

Within a well-designed and constructed subsurface absorption trench, diffusion of oxygen into the vadose zone promotes the biological oxidation of NH_4^+ -N to NO_3^- -N through biological nitrification. Depending on soil moisture conditions and organic matter concentrations within the soil column, NO_3^- -N can be reduced, under anoxic conditions, to N_2 gas through heterotrophic denitrification. A carbon source is required for the denitrification to occur. Although denitrification may be significant in some soils (Ayres Associates, 1993), in many instances there may not be sufficient organic substrate at a depth below the 'A' horizon to promote denitrification. Under these conditions NO_3^- -N can migrate downward into the groundwater aquifer, depending on soil moisture conditions (saturated or unsaturated flow). The historical practice of constructing relatively deep subsurface soil absorption trenches (2 to 4 ft.) for septic tank effluents thus may often have the effect of diminishing denitrification potential and enhancing NO_3^- -N movement in the soil column.

Plant uptake of NH_4^+ -N and NO_3^- -N could also be a potential removal mechanism for subsurface absorption systems if trench designs were changed to shallow trenches within the proximity of root systems.

IV. TREATMENT PROCESSES FOR ONSITE NITROGEN REMOVAL

A. Sequential Nitrification/Denitrification processes.

Sequential nitrification/denitrification processes, which attempt to optimize natural biological processes through engineering, form the basis of all biological nitrogen removal technologies that have been used or proposed for onsite wastewater treatment. In these systems aerobic

processes are first used to remove BOD and nitrify organic and NH_4^+ -N; anoxic processes are then used to reduce NO_3^- -N to N_2 gas, either using the wastewater as a carbon source or an external carbon source. Figure 2 shows a conceptual model of biological nitrification/denitrification in onsite wastewater treatment. Although there are other possible processes, some of which will be mentioned below, biological nitrification/denitrification is the only process that has been demonstrated to be feasible, both economically and technically, for onsite nitrogen removal (the same can be said for large-scale wastewater treatment plants) (Whitmeyer, *et al.*, 1991). Table 1 gives a summary of onsite nitrogen removal systems that have been reported in the literature; these systems will be discussed in more detail in a later section.

B. Classification of Biological Nitrogen Removal Systems

Using the terminology of wastewater engineering, the systems outlined in Table 1 are categorized according to whether they are suspended growth or attached-growth processes. Suspended-growth processes are biological treatment processes in which the microorganisms responsible for treatment are maintained in suspension within the liquid, usually through mechanical or diffused-air aeration. Attached-growth processes are those in which the microorganisms responsible for treatment are attached to an inert medium such as sand, gravel, or plastic media, and can include either submerged or nonsubmerged processes (Crites and Tchobanoglous, 1998; Metcalf & Eddy, 1991).

C. Wastewater as the Carbon Source for Denitrification

Theoretically, 2.86 mg of O_2 is equivalent to 1.0 mg of NO_3^- -N in the transfer of one electron (US EPA, 1993). Assuming that the wastewater itself will be the carbon source for denitrification, at least 2.86 mg of oxygen in the form of CBOD is thus required to reduce, through denitrification, 1.0 mg of NO_3^- -N to N_2 gas (EPA, 1993). In practice, as will be discussed below, approximately 4.0 mg of oxygen is actually needed to reduce 1.0 mg of NO_3^- -N because a portion of the CBOD must be used for bacterial growth rather than the reduction of NO_3^- -N. Assuming an average of 175 mg/L BOD_L in a septic tank effluent ($\text{BOD}_5 \approx 120$ mg/L), the maximum possible removal of Total-N would thus be about 44 mg/L. Therefore, at least from a theoretical standpoint and assuming there are no problems with alkalinity or toxicity, there should normally be sufficient indigenous carbon in domestic wastewater from septic tanks to have almost complete nitrogen removal. The problem, of course, is being able to realize this potential through engineering design and appropriate operation and maintenance of biological

Table 1

Examples of Onsite Biological Nitrogen Removal from the Literature

Technology Examples	Total-N Removal Efficiency, %	Effluent Total-N mg/L
<u>Suspended Growth:</u>		
Aerobic units w/pulse aeration	25-61 ¹	37-60 ¹
Sequencing batch reactor	60 ²	15.5 ²
<u>Attached Growth:</u>		
Single-Pass Sand Filters (SPSF)	8-50 ³	30-65 ³
Recirculating Sand/Gravel Filters (RSF)	15-84 ⁴	10-47 ⁴
Multi-Pass Textile Filters	14-31 ⁵	14-17 ⁵
RSF w/Anoxic Filter	40-90 ⁶	7-23 ⁶
RSF w/Anoxic Filter w/external carbon source	74-80 ⁷	10-13 ⁷
RUCK system	29-54 ⁸	18-53 ⁸

¹California Regional Water Quality Control Board (1997); Whitmeyer, *et al.*, (1991).

²Ayres Associates (1998).

³Converse, (1999); Gold, *et al.*, (1992); Loomis, *et al.*, (2001); Nolte & Associates, (1992); Ronayne, *et al.*, (1982).

⁴California Regional Water Quality Control Board (1997); Gold, *et al.* (1992); Loomis, *et al.* (2001); Nolte & Associates (1992); Oakley, *et al.* (1999); Piluk and Peters (1994); Ronayne, *et al.*, (1982).

⁵Leverenz, *et al.* (2001).

⁶Ayres Associates (1998); Sandy, *et al.* (1988).

⁷Gold, *et al.* (1989).

⁸Brooks (1996); Gold, *et al.* (1989).

systems. The removal efficiencies shown in Table 1 underscore the difficulty in approaching the theoretical potential in practice.

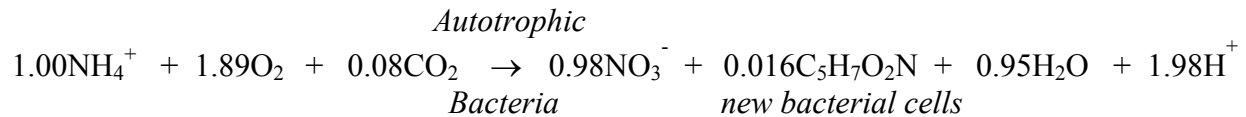
D. Capital Costs, Operation and Maintenance, and Performance Information

As reported in the literature, the capital costs of many onsite nitrogen removal systems have severely limited their widespread application. The costs of those systems designed specifically for nitrogen removal for a single family dwelling ranged from \$5,800 to \$11,300 in Wisconsin in 1991, \$2,000 to \$15,000 in California in 1997, and \$7,872 to \$17,414 in Florida in 1998 (Ayres Associates, 1998; California Regional Water Quality Control Board, 1997; Whitmeyer, *et al.*, 1991). As technology designs improve and housing development pressures build in many areas, however, there is continued interest in implementing nitrogen removal technologies as soon as possible.

V. BIOLOGICAL NITRIFICATION

A. Process Chemistry

As mentioned above, nitrification is a two-step autotrophic process (nitrifiers use CO₂ instead of organic carbon as their carbon source for cell synthesis) for the conversion of NH₄⁺ to NO₃⁻-N. During this energy yielding reaction some of the NH₄⁺ is synthesized into cell tissue giving the following overall oxidation and synthesis reaction (US EPA, 1993):



The above equation poses several key design constraints on nitrification systems. For each mole of NH₄⁺ oxidized, 1.89 moles of oxygen are required and 1.98 moles of hydrogen ions will be produced. Or, in mass terms, 4.32 mg of O₂ are required for each mg of NH₄⁺-N oxidized, with the subsequent loss of 7.1 mg of alkalinity as CaCO₃ in the wastewater, and the synthesis of 0.1 mg of new bacterial cells. Stated yet another way, the oxidation of, for example, 20 mg/L of NH₄⁺-N would require the consumption of 86.4 mg/L of dissolved oxygen, the destruction of

141.4 mg/L of alkalinity as CaCO_3 , and the production of 2.6 mg/L of nitrifying organisms (US EPA, 1993).

Nitrification can thus exert a very high nitrogenous biochemical oxygen demand (NBOD) in addition to the carbonaceous BOD (CBOD) as shown in Figure 3-3. Using the above equation, a septic tank effluent of 40 mg/L NH_4^+ -N would have a NBOD of about 184 mg/L in addition to the CBOD. This factor must be included in the design of nitrification systems to be sure there is sufficient dissolved oxygen (DO) within the system for nitrification to occur. Nitrification can also cause a significant drop in pH if there is not adequate buffering capacity in the wastewater.

B. Process Microbiology

Nitrifying organisms exhibit growth rates that are much lower than those for heterotrophic bacteria. As a result, the rate of nitrification is controlled first by concurrent heterotrophic oxidation of CBOD; as long as there is a high organic (CBOD) loading to the system, the heterotrophic bacteria will dominate. Nitrification systems must thus be designed to allow sufficient detention time within the system for nitrifying bacteria to grow. Heterotrophic organisms can also play a key role in limiting oxygen transfer to nitrifying bacteria, especially in attached-growth systems (Rittman and McCarty, 2001; US EPA, 1993). After competition with heterotrophs, the rate of nitrification will be limited by the concentration of available NH_4^+ -N in the system. Temperature, pH, and chemical inhibitors can also play a key role as discussed below.

Figure 4 shows the relationship between the fraction of nitrifying organisms in suspended-growth wastewater treatment (activated sludge) and the BOD_5/TKN ratio. At low BOD_5/TKN ratios (0.5 to 3) the population of nitrifying bacteria is high and nitrification should not be influenced by heterotrophic oxidation of CBOD (Metcalf & Eddy, 1991); this type of nitrification process is termed separate-stage nitrification. At higher BOD_5/TKN ratios, the fraction of nitrifying organisms in the system is much lower due to heterotrophic competition from oxidation of CBOD; this process is termed single-stage nitrification. Examples of single-stage and separate-stage nitrification are shown in Figure 5.

Separate-stage nitrification is highly desirable from the standpoint of process control and operation. Many onsite systems presently used or proposed for nitrogen removal, however, because of the interest in reducing size and system footprint, employ single-stage nitrification; examples would include aerobic treatment units with short hydraulic detention times and sand filters or media filters that are heavily loaded organically. Single-stage systems may require more rigorous process control to ensure adequate nitrification rates.

C. Dissolved Oxygen Requirements and Organic Loading Rates

1. Suspended Growth Systems.

The concentration of DO has a significant effect on nitrification in wastewater treatment. Although much research has been performed, practical experience has shown that DO levels must be maintained at approximately 2.0 mg/L in suspended-growth (aerobic) systems, especially when NH_4^+ -N loadings are expected to fluctuate widely (US EPA, 1993); this may or may not be the case in domestic onsite wastewater systems.

2. Attached-Growth Systems.

For attached-growth systems, which include both submerged and nonsubmerged processes (Crites and Tchobanoglous, 1998), DO levels must be maintained at levels that are at least 2.7 times greater than the NH_4^+ -N concentrations in order to prevent oxygen transfer through the biofilm from limiting nitrification rates (US EPA, 1993). This is usually overcome in practice by using lower organic surface loadings than what would be normally applied for CBOD removal to allow for growth of nitrifying organisms; otherwise the heterotrophic organisms will dominate the bacterial film within the attached-growth media. For trickling filters, for example, the organic loading rate for nitrification is only about 1/5 to 1/8 of the CBOD loading for CBOD removal (Metcalf & Eddy, 1991; US EPA, 1993). Recirculation of effluent through the attached growth media, and use of special media, such as trickling filter plastic media with high specific surface areas, are also used to lower organic surface loadings and to promote high oxygen transfer rates.

Table 2 shows design organic loading rates for various attached-growth systems to achieve nitrification. Unfortunately, organic loading rates for onsite attached-growth systems are not well defined even for CBOD removal, let alone nitrification (Crites and Tchobanoglous, 1998). The more commonly used hydraulic loading rates as cited in the literature show mixed results for nitrification. This is no doubt due, at least in part, to varying organic loading rates that were not taken into consideration since the CBOD_5 of septic tank effluent can vary greatly, ranging from less than 100 to 480 mg/L (Ayres Associates, 1993).

Table 2

Design Loading Rates for Attached Growth Systems to Achieve >85% Nitrification

Process	Hydraulic Loading Rate, gpd/ft ²	Organic Loading Rate, lbs. BOD/ft ² -day	State of Knowledge for Design
<u>Trickling Filters</u> ¹			
Rock Media	30-900	0.04-0.12 (0.04-0.64)	Well Known
Plastic Media	288-1700	0.10-0.25 (0.50-2.00)	Well Known
<u>Sand Filters</u>			
Single-Pass	0.4-1.2	0.000135-0.002	Lesser Known ²
Recirculating	3-5	0.002-0.008	Lesser Known ²
<u>Textile Filters</u>			
Single-Pass	10	0.01	Lesser Known ²
Multi-Pass ³ (Partial Nitrification)	30	0.03	Lesser Known ²

1. The values for trickling filters given for both hydraulic and organic loadings are the ranges for low rate and high rate filters. Rock filters were assumed to have a depth of 8 ft. and plastic filters a depth of 10 ft. The numbers in parentheses for organic loadings are the values for CBOD removal only without nitrification.

2. These systems have not traditionally been designed using organic loading rates to achieve nitrification. High strength wastes thus could affect nitrification performance.

3. At this organic loading rate only 59-76% nitrification was achieved (Leverenz, *et al.*, 2001).

Adapted from Converse (1999); Crites and Tchobanaglou (1998); Leverenz, *et al.* (2001); Metcalf & Eddy (1991); and US EPA (1993).

EXAMPLE 2. CALCULATION OF ORGANIC SURFACE LOADING RATES FOR NITRIFICATION OF ATTACHED-GROWTH SYSTEMS. Continuing from Example 1, you are to size and determine the organic loading rates for a single pass sand filter and a single pass textile filter for an average single-family dwelling unit. The following conditions apply:

1. The per capita wastewater generation rate is 45 gpd.
2. The peaking factor for flow is 2.5.
3. There is an average of 2.4 residents per household in the Chico Urban Area.
4. The BOD₅ in septic tank effluent is assumed to be 150 mg/L.

Solution

1. Determine the total wastewater flow per day.

$$Q_{\text{wastewater}} = (45 \text{ gpd/person}) \cdot (2.4 \text{ persons}) = 108 \text{ gpd.}$$

$$Q_{\text{peak}} = 2.5 \cdot (108 \text{ gpd}) = 270 \text{ gpd}$$

2. Size the system.

Single Pass Sand Filter

$$\text{Hydraulic loading rate} = 1.2 \text{ gpd/ft}^2 \text{ (Table 2)}$$

$$\text{Surface Area of SPSF} = 270 \text{ gpd} / 1.2 \text{ gpd/ft}^2 = 225 \text{ ft}^2.$$

Single Pass Textile Filter

$$\text{Hydraulic loading rate} = 10 \text{ gpd/ft}^2 \text{ (Table 2)}$$

$$\text{Surface Area of SPTF} = 270 \text{ gpd} / 10 \text{ gpd/ft}^2 = 27 \text{ ft}^2.$$

3. Determine the organic surface loading rate.

Single Pass Sand Filter

Organic loading rate =

$$150 \text{ mg/L} ((1.2 \text{ gpd/ft}^2)(3.78 \text{ l/gal})(1 \text{ g/1000 mg})(1 \text{ lb./454 g}) = 0.0015 \text{ lbs. BOD}_5/\text{ft}^2\text{-day}$$

Single Pass Textile Filter

Organic loading rate =

$$150 \text{ mg/L} ((10 \text{ gpd/ft}^2)(3.78 \text{ l/gal})(1 \text{ g/1000 mg})(1 \text{ lb./454 g}) = 0.012 \text{ lbs. BOD}_5/\text{ft}^2\text{-day}$$

Comment. These types of systems have not been traditionally designed using organic loading rates for nitrification. According to Table 2, both filters may be approaching the organic loading limit for nitrification. Thus it may be appropriate to increase the surface area of each filter in order to ensure nitrification, even though they are within the norms for hydraulic loading rates, which is usually the case since many county codes specify 360-400 ft² of filter surface for a single-family residence. If the BOD₅ were higher in this example, nitrification may be significantly reduced. As an example, a recirculating textile filter only exhibited partial nitrification (59-76%) at an organic loading rate of 0.03 lbs. BOD₅/ft²-day (Leverenz, *et al.*, 2001). To improve design, more data are needed on the range of allowable organic loading rates of attached-growth systems to achieve nitrification.

D. pH and Alkalinity Effects

The optimum pH range for nitrification is 6.5 to 8.0 (US EPA, 1993). Because nitrification consumes about 7.1 mg of alkalinity (as CaCO₃) for every mg of NH₄⁺-N oxidized, in low alkalinity wastewaters there is a risk that nitrification will lower the pH to inhibitory levels. If, for example, it were desired to nitrify 40 mg/L of NH₄⁺-N, approximately 284 mg/L as CaCO₃ would be required to maintain pH levels; this may be beyond the capabilities of some wastewaters derived from water sources that do not contain relatively high alkalinity.

Figures 6 and 7 graphically show the loss of alkalinity with nitrification for septic tank effluent that percolated through the soil column and was measured at a two-foot depth with suction lysimeters (Oakley, *et al.*, 1999). In this particular example, the alkalinity decreased from an average of approximately 400 mg/L to 100 mg/L as CaCO₃ in order to nitrify an average of

about 40 mg/L organic-N and NH_4^+ -N. Figure 8 shows the theoretical relationship of the fraction of TKN that can be nitrified as a function of initial TKN and alkalinity in the wastewater.

E. Temperature Effects

Temperature has a significant effect on nitrification that must be taken into consideration for design (US EPA, 1993). In general, colder temperatures require longer cell residence times in suspended-growth systems and lower hydraulic loading rates in attached-growth systems due to slower growth rates of nitrifying bacteria.

F. Effect of Inhibitors

Nitrifying bacteria are much more sensitive than heterotrophic bacteria and are susceptible to a wide range of organic and inorganic inhibitors as shown in Table 3. As has occurred in centralized wastewater treatment (US EPA, 1993), there is a need to establish a methodology for onsite wastewater systems for assessing the potential for, and occurrence of, nitrification inhibition. Figure 9 illustrates the effect of an inhibitor on nitrification in a septic tank/recirculating trickling filter system; in this particular case a carpet cleaning solvent that was flushed down the toilet contaminated the septic tank and destroyed the nitrifying bacterial population in the attached-growth media (Oakley, *et al.*, 1996). If this system had not been continuously monitored, the effects of the inhibitor on nitrification would have passed unnoticed.

Since heterotrophic bacteria are much more resilient than nitrifying bacteria, and because many of the inhibitory compounds are biodegradable organics, inhibitory effects can oftentimes be controlled by designing separate-stage nitrification systems (US EPA, 1993). In separate-stage systems the CBOD is first removed along with any biodegradable inhibitory compounds; the nitrifying organisms, which are in effect protected in the second stage, are then used to nitrify the low-CBOD, high- NH_4^+ -N effluent (Figure 5).

Table 3

Examples of Nitrification Inhibitors

Inorganic Compounds

Zinc
Free Cyanide
Perchlorate
Copper
Mercury
Chromium
Nickel
Silver
Cobalt
Thiocyanate
Sodium cyanide
Sodium azide
Hydrazine
Sodium cyanate
Potassium chromate
Cadmium
Arsenic
Fluoride
Lead
Free ammonia
Free nitrous acid

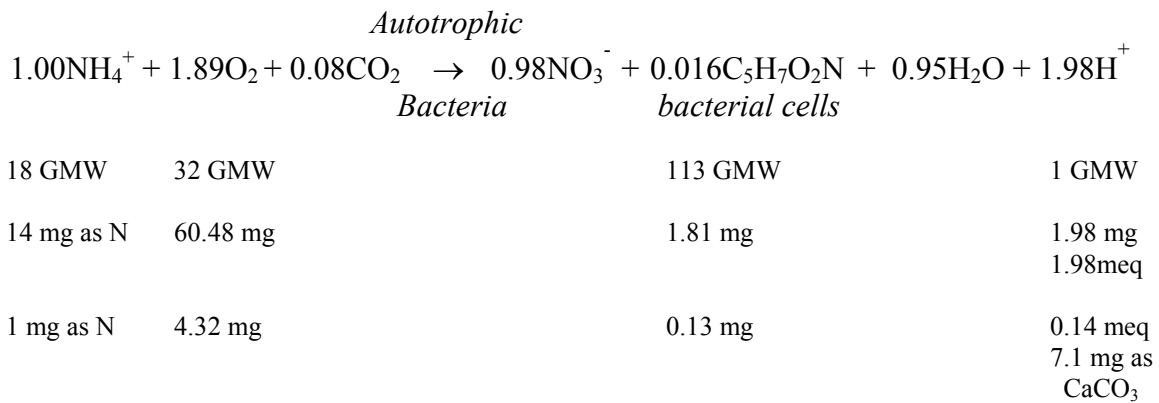
Organic Compounds

Acetone
Carbon Disulfide
Chloroform
Ethanol
Phenol
Ethylenediamine
Hexamethylene diamine
Aniline
Monoethanolamine

EXAMPLE 3. CALCULATION OF ALKALINTIY AND OXYGEN REQUIREMENTS FOR NITRIFICATION. Determine the alkalinity requirements for complete nitrification for a septic tank effluent that has a CBOD of 150 mg/L and an Organic-N and NH_4^+ -N concentration of 40 mg/L. What would be the NBOD of this wastewater?

Solution

1. Write a balanced equation for the nitrification reaction and include mass relationships:



GMW = gram molecular weight

Milliequivalent mass of $\text{CaCO}_3 = 50 \text{ mg/meq}$

$\therefore 0.14 \text{ meq as H}^+$ in terms of equivalent $\text{CaCO}_3 = 0.14 \text{ meq} (50 \text{ mg CaCO}_3/\text{meq}) = 7.1 \text{ mg as CaCO}_3$

2. Determine alkalinity requirements.

$$\text{Alkalinity required} = 40 \text{ mg/L Total-N} \cdot (7.1 \text{ mg/L CaCO}_3/\text{mg N}) = 284 \text{ mg/L as CaCO}_3$$

3. Determine the NBOD.

$$\text{NBOD} = 40 \text{ mg/L total-N} (4.32 \text{ mg O}_2/\text{mg N}) = 173 \text{ mg/L}$$

Comment. The alkalinity requirements here exceed the 200 mg/L as CaCO_3 that has been reported to be a typical alkalinity concentration in strong, untreated domestic wastewater (Metcalf & Eddy, 1991). Alkalinity does increase as a result of water use, and the incremental range for septic tank effluent has been reported from 60-120 mg/L as CaCO_3 (Crites and Tchobanoglous, 1998). In areas with low-alkalinity source waters, however, nitrification could

be limited. Note that the NBOD exceeds the CBOD of the septic tank effluent, which underscores the oxygen requirements for nitrification.

G. Summary of Nitrification Processes

Table 4 summarizes the various onsite technologies and their advantages and disadvantages for effective nitrification based on the factors discussed above. The available information suggests that an effective design strategy for nitrification in onsite systems would be to use attached-growth processes with relatively low organic loadings (compared to CBOD removal only) and deep, well-aerated media (such as a 2 ft. deep SPSF). This type of system would approach a separate-stage nitrification with its advantages while maintaining the cost and simplicity of a single-stage system. In this design the heterotrophic bacteria would grow in the upper levels and remove CBOD and inhibitory compounds; nitrifying bacteria would grow in the lower levels and would be protected both from shock loadings and temperature extremes. A single pass sand filter, which is well known for its nitrification reliability, is an example of this design.

VI. BIOLOGICAL DENITRIFICATION

A. Process Description

Denitrification is a biological process that uses NO_3^- as the electron acceptor (hence nitrification must precede denitrification) instead of O_2 to oxidize organic matter (heterotrophic denitrification) or inorganic matter such as sulfur or hydrogen (autotrophic denitrification) under anoxic conditions (Rittmann and McCarty, 2001). In the process NO_3^- is reduced to N_2 gas. Because the principal biochemical pathway is a modification of aerobic pathways (ie., NO_3^- is used as the electron acceptor instead of O_2), the denitrification process is said to occur under anoxic conditions as opposed to anaerobic conditions (where obligate anaerobic organisms would be present). Denitrifying bacteria, whether heterotrophic or autotrophic, are facultative aerobes and can shift between oxygen respiration and nitrate respiration. For heterotrophic denitrification, the carbon source can come from the original wastewater, bacterial cell material, or an external source such as methanol or acetate. For autotrophic denitrification, which is common in water treatment but not wastewater treatment, the electron donor can come from elemental sulfur or hydrogen gas (Rittmann and McCarty, 2001). The possible process configurations for heterotrophic denitrification are shown in Figure 10.

Table 4

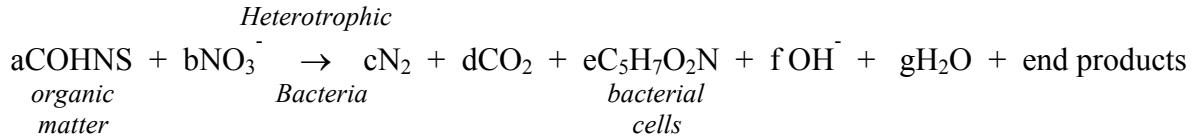
Onsite Technologies for > 85% Nitrification

<u>Process</u>	<u>Effectiveness</u>	<u>Onsite Status</u>
<u>Suspended Growth:</u>		
Aerobic units	Potential	Insufficient design and performance data. Operation and maintenance unknown.
<u>Attached Growth:</u>		
Single-Pass Sand Filters (SPSF)	Proven	Widespread use. Need more design data for organic loadings for nitrification. Fair to good performance in cold climates.
Recirculating Sand Filters (RSF)	Proven	Widespread use. Need more design data for organic loadings for nitrification. Poorer performance in cold climates than SPSFs.
Single-Pass Textile Filters	Potential	Limited data to date. Probably similar to SPSF. Need design data for organic loadings for nitrification.
Multi-Pass Textile Filters	Potential	Limited data to date. Probably similar to RSF. Need design data for organic loadings for nitrification and performance in cold climates.

B. Heterotrophic Denitrification

1. Wastewater as Carbon Source.

The following unbalanced equation illustrates the process when wastewater or bacterial cell material is used as the carbon source (US EPA, 1993):



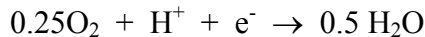
As is shown in the following example, the reduction of 1 mg of NO_3^- is equivalent to 2.86 mg of O_2 . Thus, for example, a wastewater with an ultimate BOD (BOD_L) of 200 mg/L could potentially reduce almost 70 mg/L of NO_3^- -N if the wastewater were used as the carbon source (US EPA, 1993). This does not happen in practice, however, because a portion of the organic carbon in the wastewater must be used for cell synthesis and not nitrate reduction.

EXAMPLE 4. CALCULATION OF STOICHIOMETRIC EQUATIONS FOR NITRATE REDUCTION USING THE WASTEWATER AS THE CARBON SOURCE. Determine the theoretical amount of NO_3^- -N that could be removed if septic tank effluent, which has a BOD_5 of 120 mg/L, is used as the carbon source. What quantity could be removed if the raw wastewater influent to the septic tank, with a BOD_5 of 220 mg/L, were used as the carbon source?

Solution

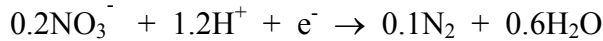
1. Write the half-reactions for oxygen and nitrate as electron acceptors (Rittmann and McCarty, 2001):

Oxygen:



32 GMW
 8 g [0.25(32) = 8]
 8 mg

Nitrate:



62 GMW
 14 GMW as N
 2.8 g [0.2(14) = 2.8]
 2.8 mg

2. Determine the stoichiometric equivalency of oxygen and nitrate.

For the acceptance of one electron, the above equations show that 8 mg of O₂ is equivalent to 2.8 mg of NO₃⁻-N, or that 1.0 mg of NO₃⁻-N is equivalent to 2.86 mg of O₂.

$$\frac{8 \text{ mg O}_2/e^- \text{ equiv.}}{2.8 \text{ mg NO}_3^- \text{-N}/e^- \text{ equiv.}} = 2.86 \text{ mg O}_2/\text{mg NO}_3^- \text{-N}$$

3. Determine the BOD_L of the wastewater.

The stoichiometric equations must be based on the ultimate BOD (BOD_L) rather than the more commonly used BOD₅. The BOD₅ of wastewater can range between 68% to 94% of the BOD_L, depending on the value of the BOD reaction rate constant, *k* (Sawyer, *et al.*, 1994). It will be assumed here that *k* (base e) is 0.23 d⁻¹ at 20 °C, a typical value for domestic wastewater (Metcalf & Eddy, 1991).

Septic Tank Effluent:

$$\text{BOD}_L = \frac{\text{BOD}_5}{(1 - e^{-kt})} = \frac{120}{(1 - e^{-0.23(5)})} = \frac{120}{0.68} = 176 \text{ mg/L}$$

Septic Tank Influent:

$$\text{BOD}_L = \frac{\text{BOD}_5}{(1 - e^{-kt})} = \frac{220}{(1 - e^{-0.23(5)})} = \frac{220}{0.68} = 323 \text{ mg/L}$$

4. Determine the quantity of NO₃⁻-N that could theoretically be reduced.

Septic Tank Effluent:

$$\text{NO}_3^- \text{-N Reduction} = \frac{176 \text{ mg O}_2 \text{ demand/L}}{2.86 \text{ mg O}_2 / \text{mg NO}_3^- \text{-N}} = 61.5 \text{ mg/L}$$

Septic Tank Influent:

$$\text{NO}_3^- \text{-N Reduction} = \frac{323 \text{ mg O}_2 \text{ demand/L}}{2.86 \text{ mg O}_2 / \text{mg NO}_3^- \text{-N}} = 113 \text{ mg/L}$$

In practice the equivalency of 2.86 mg O₂/ mg NO₃⁻-N is not achievable because a portion of the electron donor (ie., the wastewater) must be used to provide carbon for cell synthesis; thus more electron donor will be needed to reduce a given amount of NO₃⁻-N than is predicted by the half-reactions alone. For complex organic matter such as wastewater, the stoichiometric equivalency can range from 3.46-5.07 mg BOD_L/mg NO₃⁻-N, with 4.0 mg BOD_L/ mg NO₃⁻-N used as a rule of thumb (Rittmann and McCarty, 2001). In terms of BOD₅, this amounts to 2.72 mg BOD₅/ mg NO₃⁻-N for k (base e) = 0.23 d⁻¹.

EXAMPLE 5. RECALCULATION OF STOICHIOMETRIC EQUATIONS FOR NITRATE REDUCTION USING THE WASTEWATER AS THE CARBON SOURCE.

Recalculate the amount of NO₃⁻-N that could be removed in Example 4 using the "rule of thumb" stoichiometric equivalency.

Solution

1. Express the stoichiometric equivalency in terms of the commonly used BOD₅.

$$\text{BOD}_5 = 0.68\text{BOD}_L \text{ for } k = 0.23 \text{ d}^{-1} \text{ (base e)}$$

$$\therefore \frac{4.0 \text{ mg BOD}_L}{\text{mg NO}_3^- \text{-N}} = 0.68(4.0) = \frac{2.72 \text{ mg BOD}_5}{\text{mg NO}_3^- \text{-N}}$$

2. Determine the quantity of NO₃⁻-N that could theoretically be reduced.

Septic Tank Effluent:

$$\text{NO}_3^- \text{-N Reduction} = \frac{120 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 44 \text{ mg/L}$$

Septic Tank Influent:

$$\text{NO}_3^- \text{-N Reduction} = \frac{220 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 81 \text{ mg/L}$$

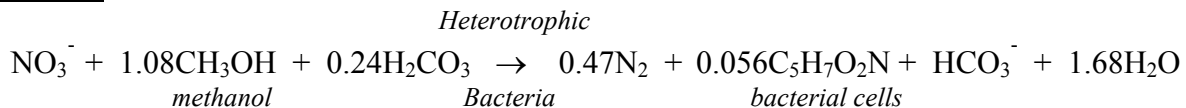
Comment. To achieve the maximum nitrate reduction potential, the wastewater should be used at the point of highest CBOD. This may not occur if septic tank effluent, for example, or a recirculation tank from a packed bed filter system, is used as the point of application of the carbon source. Imperfect mixing of the wastewater carbon source with the nitrified effluent, and the absence of anoxic conditions, can also cause a reduction in denitrification.

Figure 11, which assumes the "rule of thumb" stoichiometric equivalency of 4.0 mg BOD₁/mg NO₃⁻N (2.72 mg BOD₅/mg NO₃⁻N), shows total nitrogen removal as a function of initial TKN and wastewater BOD₅. In this figure it is assumed there is sufficient alkalinity for nitrification, and that $k = 0.23 \text{ d}^{-1}$. It is obvious from Figure 11 that nitrogen removal by denitrification using wastewater as the carbon source is highly feasible for an initial TKN of 40 mg/L or less, but becomes more problematic as the initial TKN increases in relation to BOD₅.

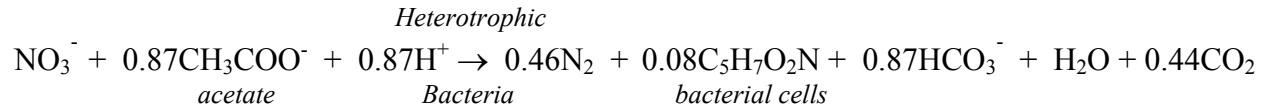
2. External Carbon Source.

In cases where there is insufficient CBOD left in the wastewater to serve as an electron donor for denitrification, an external carbon source must be supplied. Although there are many possibilities, methanol and acetate have been studied the most and their stoichiometry is shown below (Rittmann and McCarty, 2001; US EPA, 1993):

Methanol:



Acetate:



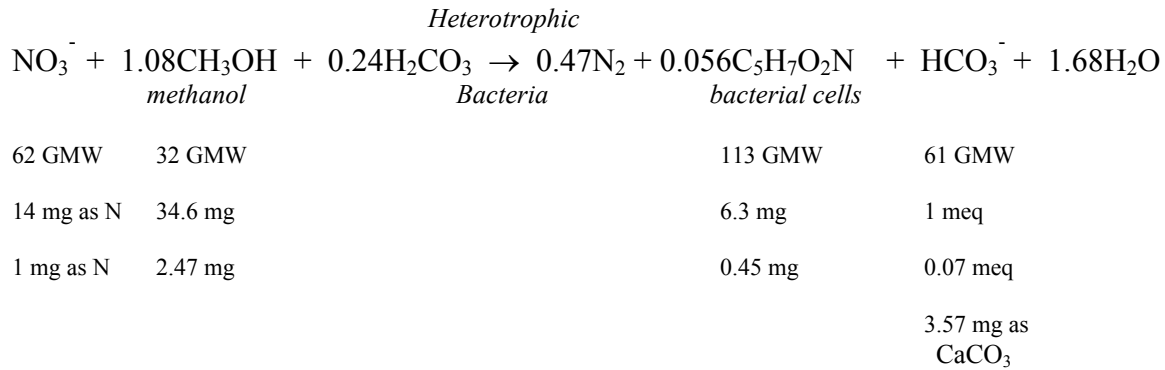
There are few examples in the literature of an external carbon source being used for onsite denitrification. Although methanol has been studied extensively in centralized wastewater treatment plants, it is probably not a good choice for onsite systems because of its toxicity and potential for contaminating groundwater supplies. Gold, *et al.*, (1989) reported on the use of both methanol and ethanol as an external carbon source in a recirculating sand filter system with an anoxic rock filter for denitrification. They noted that although the total nitrogen removal rate was as high as 80%, the use of the chemicals required operation and maintenance of the carbon source supply system, including an on-site storage facility, a metering pump mechanism, and supplying a diluted carbon source solution. They concluded that the external carbon source could probably best be handled by a wastewater management district or a private O & M contractor (Gold, *et al.*, 1989).

EXAMPLE 6. DESIGN OF DENITRIFICATION SYSTEM USING METHANOL AS THE CARBON SOURCE. Determine the methanol requirements for an onsite denitrification system using a recirculating sand filter. The following conditions apply:

1. Household flowrate = 192 gpd.
2. The concentration of NO_3^- -N to be removed is 40 mg/L.
3. Characteristics of Methanol:
 - 99.90% Solution = 0.7913 g $\text{CH}_3\text{OH}/\text{ml}$
 - 10.00% Solution = 0.08 g $\text{CH}_3\text{OH}/\text{ml}$

Solution

1. Write the balanced equation for denitrification and include mass relationships:



2. Determine the required concentration of methanol.

$$\text{Required concentration of CH}_3\text{OH} = \frac{2.47 \text{ mg}}{\text{mg NO}_3^- \cdot \text{N}} (40 \text{ mg/L NO}_3^- \cdot \text{N}) = 98.8 \text{ mg/L} \approx 100 \text{ mg/L}$$

3. Determine the methanol requirement.

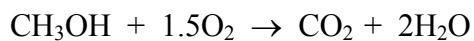
$$\text{CH}_3\text{OH requirement} = (100 \text{ mg/L})(150 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ g}/1000 \text{ mg}) = 56.7 \text{ g/day}$$

10% Solution:

$$\frac{56.7 \text{ g/day}}{0.08 \text{ g/ml}} = 709 \text{ ml/day} = 21.3 \text{ L/month} = 5.6 \text{ gallons/month} \approx \text{one 55 gallon drum}/10 \text{ mos.}$$

$$0.08 \text{ g/ml}$$

4. Determine the BOD_L of methanol.



32 GMW	32 GMW
32 mg	48 mg
1 mg	1.5 mg

$$\therefore 1 \text{ mg of CH}_3\text{OH} \Rightarrow 1.5 \text{ mg BOD}_L$$

$$\therefore 100 \text{ mg/L CH}_3\text{OH} \Rightarrow 150 \text{ mg/L BOD}_L$$

5. Determine the ratio of BOD_L/NO_3^- -N reduced.

$$\frac{150 \text{ mg/L } BOD_L}{40 \text{ mg/L } NO_3^- \text{-N}} = 3.75 \text{ mg } BOD_L/\text{mg } NO_3^- \text{-N reduced.}$$

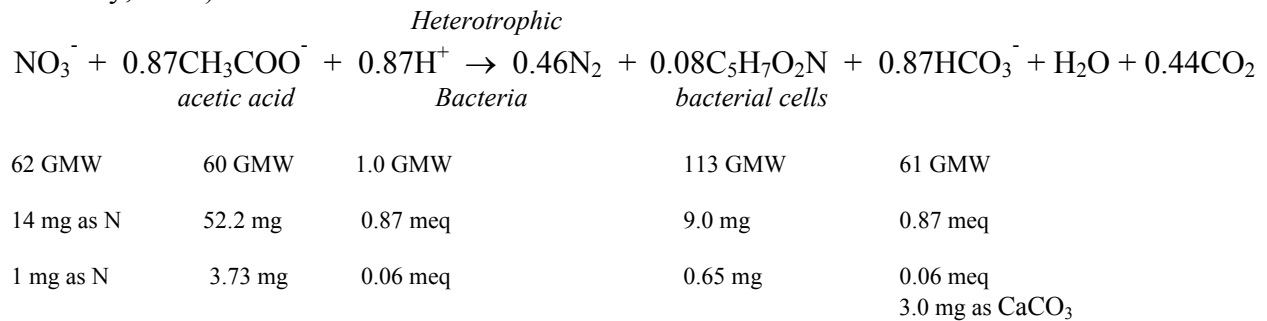
Comment. This example shows that the required BOD_L of methanol is higher than that predicted by half-reactions alone (2.86 mg $BOD_L/\text{mg } NO_3^-$ -N) because a portion of the methanol was used for cell synthesis as can be seen in the balanced equation. Note that 3.57 mg of alkalinity as $CaCO_3$ was produced per mg of NO_3^- -N reduced. Thus approximately half of the alkalinity lost during nitrification can be recovered through denitrification with methanol or wastewater as the carbon source.

EXAMPLE 7. DESIGN OF DENITRIFICATION SYSTEM USING ACETIC ACID AS THE CARBON SOURCE. Determine the acetic acid requirements for an onsite denitrification system using a recirculating sand filter. Assume the acetic acid could be used in the form of vinegar (5% solution). The following conditions from Example 6 apply:

1. Household flowrate = 192 gpd.
2. The concentration of NO_3^- -N to be removed is 40 mg/L.
3. Characteristics of acetic acid:
 - 99.5% Solution = 1.05 g CH_3COOH/ml
 - 5.0% Solution = 0.05 g CH_3COOH/ml

Solution

1. Write the balanced equation for denitrification and include mass relationships (Rittmann and McCarty, 2001):



2. Determine the required concentration of acetic acid.

$$\text{Required concentration of CH}_3\text{COOH} = \frac{3.73 \text{ mg}}{\text{mg NO}_3^- \text{-N}} (40 \text{ mg/L NO}_3^- \text{-N}) = 149 \text{ mg/L}$$

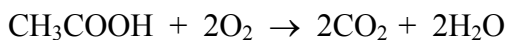
3. Determine the acetic acid requirement.

$$\text{CH}_3\text{COOH requirement} = (149 \text{ mg/L})(150 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ g/1000 mg}) = 84.5 \text{ g/day}$$

5% Solution:

$$\frac{84.5 \text{ g/day}}{0.05 \text{ g/ml}} = 1690 \text{ ml/day} = 50.7 \text{ L/month} = 13.4 \text{ gallons/month} \approx \text{one 55 gallon drum/4 mos.}$$

4. Determine the BOD_L of acetic acid.



60 GMW	32 GMW
60 mg	64 mg
1 mg	1.07 mg

$$\begin{aligned} \therefore 1 \text{ mg of CH}_3\text{COOH} &\Rightarrow 1.07 \text{ mg BOD}_L \\ \therefore 149 \text{ mg/L CH}_3\text{COOH} &\Rightarrow 159 \text{ mg/L BOD}_L \end{aligned}$$

5. Determine the ratio of BOD_L/NO₃⁻-N reduced.

$$\frac{159 \text{ mg/L BOD}_L}{40 \text{ mg/L NO}_3^- \text{-N}} = 3.97 \text{ mg BOD}_L/\text{mg NO}_3^- \text{-N reduced.}$$

Comment. In this example, unlike methanol or wastewater, acetic acid adds acid to the system, which is neutralized by the production of alkalinity through denitrification (0.87 meq of H⁺ from acetic acid would be neutralized by the 0.87 meq of HCO₃⁻ produced by denitrification). Thus there *would not* be a 50% recovery of the alkalinity lost through nitrification if acetic acid were used as the carbon source (Rittmann and McCarty, 2001).

3. Process Microbiology of Heterotrophic Denitrification.

The heterotrophic denitrifying bacteria are facultative aerobes that can use either oxygen or nitrate (under anoxic conditions) as an electron acceptor for the oxidation of organic matter. Denitrifiers are commonly found in nature and are ubiquitous in wastewater. Because of their diversity, the growth and competition problems associated with autotrophic nitrifying bacteria do not generally occur with denitrifying bacteria. Generally, denitrification processes perform similarly to aerobic processes designed for CBOD removal.

When an adequate carbon source is available, the principal problem associated with denitrification is the achievement of anoxic conditions. The dissolved oxygen concentration controls whether or not the denitrifying bacteria use NO_3^- or O_2 as the electron acceptor (US EPA, 1993). Dissolved oxygen must not be present above certain maximum levels or the denitrifying bacteria will preferentially use it for oxidation of organic matter rather than NO_3^- . As a result, the design of anoxic zones is one of the most important factors in denitrification processes.

The differences between the autotrophic nitrifiers and the heterotrophic denitrifiers underscore the complexity of the design and operation of nitrification/denitrification systems. The nitrifiers are slow growers, sensitive to inhibitory compounds, and require low organic carbon concentrations with high dissolved oxygen concentrations; the denitrifiers, on the other hand, are fast growers, resilient to inhibitory compounds, and require high organic carbon concentrations with low or absent dissolved oxygen. The reconciliation of these differences is the greatest challenge to successful onsite nitrogen removal.

4. pH and Alkalinity Effects.

As discussed above, theoretically, 3.57 mg of alkalinity as CaCO_3 is produced for each mg of NO_3^- -N reduced to N_2 gas when the wastewater is used as the carbon source. Thus denitrification can recover approximately half of the alkalinity lost in nitrification and can help overcome pH drops in low alkalinity waters. Because denitrifying organisms are heterotrophic, they normally will be affected by pH changes in the same way heterotrophic bacteria are affected. Generally, it has been found that denitrification rates are depressed below pH 6.0 and above pH 8.0 (US EPA, 1993).

If an external carbon source is used, the alkalinity lost may or may not be recovered, depending

on the chemistry of the carbon source. In the case of methanol the alkalinity is recovered, while with acetic acid the quantity of acid added to the system is neutralized by the alkalinity produced and no alkalinity is recovered through denitrification.

5. Temperature Effects.

The data from the literature suggest that denitrification rates can be significantly affected by temperature drops below 20 °C, with the denitrification rate at 10 °C ranging from 20% to 40% of the rate at 20°C (US EPA, 1993). It can be expected that this decrease is similar to that encountered for heterotrophic organisms removing CBOD and should be taken into consideration for designs in cold climates, where longer hydraulic retention times may be needed. There is also some evidence of a slight depression of denitrification rates at temperatures above 20-25 °C (US EPA, 1993).

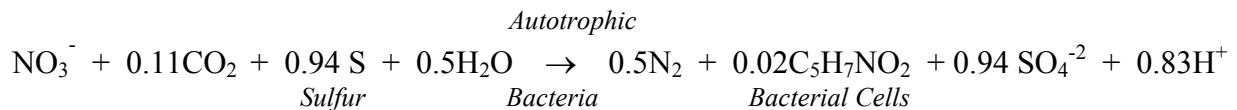
6. Inhibitory Effects.

In general, denitrifiers are much more resilient than nitrifying organisms. Denitrifiers most likely exhibit the same characteristics as heterotrophic bacteria for CBOD removal to inhibitory compounds.

C. Autotrophic Denitrification

It is also theoretically possible to denitrify wastewaters using autotrophic bacteria and elemental sulfur or H₂ gas as the electron donor instead of carbon as shown in the following stoichiometric reactions (Rittmann and McCarty, 2001):

Sulfur as Electron Donor:



remove 1 mg of NO_3^- -N and that 0.2 mg of bacterial cells would be produced; significantly, 3.0 mg of alkalinity as CaCO_3 would also be consumed.

2. Determine the mass of sulfur required per mass of NO_3^- -N reduced.

$$\begin{aligned}\text{Required mass of S} &= \frac{2.2 \text{ mg}}{\text{mg NO}_3^- \text{-N}} (40 \text{ mg/L NO}_3^- \text{-N})(194 \text{ gpd})(3.78 \text{ L/gal}) = 64,532 \text{ mg/day} \\ &= 64,532 \text{ mg/day}(1 \text{ g}/1000 \text{ mg}) = 64.5 \text{ g/day} \\ &= 64.5 \text{ g/day}(1 \text{ lbs.}/454 \text{ g}) = 0.14 \text{ lbs./day} \\ &= 0.14 \text{ lbs./day}(365 \text{ days/yr}) = 52 \text{ lbs./yr}\end{aligned}$$

3. Determine the alkalinity requirements.

From the balanced equation 3.0 mg of alkalinity as CaCO_3 is consumed for each mg of NO_3^- -N reduced.

$$\begin{aligned}\therefore \text{Alkalinity Requirement} &= (40 \text{ mg/L NO}_3^- \text{-N}) \frac{(3.0 \text{ mg of alkalinity as CaCO}_3)}{\text{mg NO}_3^- \text{-N}} \\ &= 120 \text{ mg/L as CaCO}_3\end{aligned}$$

Comment. Autotrophic denitrification using sulfur as the electron donor, unlike heterotrophic denitrification, consumes alkalinity. When coupled with nitrification, approximately 10 mg of alkalinity would be consumed for every mg of Total-N removed. In this particular example, 400 mg/L of alkalinity would be necessary to nitrify and denitrify 40 mg/L Total-N, which may be beyond the buffering capacity of many domestic wastewaters.

D. Summary of Denitrification Processes

Table 5 summarizes the three processes for heterotrophic denitrification (which are shown in Figure 10) with their advantages and disadvantages for onsite nitrogen removal. In summary, organic carbon can be provided in the following ways:

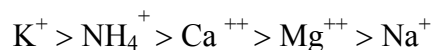
- 1) As an external carbon source to an anoxic reactor after nitrification;
- 2) As an internal source in the form of bacterial cells through a sequential process of aerobic and anoxic zones;
- 3) The influent wastewater can be used as the carbon source by recycling nitrified effluent to an anoxic reactor that precedes the aerobic nitrification reactor, operating alternating aerobic/anoxic zones on one reactor (sequencing batch reactor), or conveying the flow sequentially through alternating aerobic/anoxic zones (US EPA, 1993). Denitrification reactors can be designed as suspended-growth or attached-growth processes.

While there are a number of onsite systems in the US, such as recirculating sand filters, that denitrify on a routine basis, there presently is a lack of reliable design information data at the level that exists for centralized treatment systems. In general, using wastewater as the carbon source has many potential advantages, such as recovery of alkalinity and diminished oxygen requirements ($\approx 50\%$) for CBOD removal since NO_3^- is used as the electron acceptor (US EPA, 1993).

VII. REMOVAL OF NITROGEN BY ION EXCHANGE

A. Ion Exchange with Zeolites

NH_4^+ -N in wastewater can be preferentially removed by naturally occurring ion exchange materials called zeolites (Metcalf & Eddy, 1991). The selectivity of zeolite for the major ions in wastewater has been reported to be the following as reported by the California Regional Water Quality Control Board, (1997):



Only nitrogen in the form of NH_4^+ -N can be removed in wastewater, and this must be done under anaerobic conditions in order to inhibit nitrification. The quantity of NH_4^+ -N that can be removed depends on the zeolite bed volume and equilibrium kinetics.

Table 5

Onsite Processes for Heterotrophic Denitrification

Process	Advantages	Disadvantages
External Carbon Source	High removal rates. Denitrification easily controlled.	Insufficient performance data for onsite systems. Operation and maintenance data lacking. Routine monitoring required. Alkalinity lost through nitrification may or may not be recovered, depending on the carbon source.
Wastewater as Carbon Source	Lower energy and chemical requirements. Fifty percent recovery of alkalinity lost through nitrification. Fifty percent reduction in O ₂ requirements for CBOD removal.	Insufficient performance data. Process difficult to control. Routine monitoring required. Operation and maintenance data lacking.
Bacterial Cells as Carbon Source	Lower energy and chemical requirements.	Insufficient performance data. Process difficult to control. Routine monitoring required. Operation and maintenance data lacking.

B. Zeolite Filters for Onsite Systems.

Zeolite ion exchange filters have been used in several onsite wastewater experiments in California (California Regional Water Quality Control Board, 1997). The results showed that while NH_4^+ -N could be removed (from 16.2% to 93.8% removal was reported), the filter performance was highly variable and the filters required extensive maintenance for replacement or service of the zeolite. Indeed, ion exchange for NH_4^+ -N removal has had limited application in centralized wastewater treatment because of the extensive pretreatment required and concerns about the useful life and regeneration of the zeolite (Metcalf & Eddy, 1991). The use of zeolite for onsite NH_4^+ -N removal must therefore be considered to be in the experimental stage at the present time.

VIII. PROCESS DESIGN FOR BIOLOGICAL NITROGEN REMOVAL

A. Centralized Wastewater Treatment

Nitrogen removal through biological nitrification/denitrification, as practiced in centralized wastewater treatment, is generally classified as an advanced treatment process (Metcalf & Eddy, 1991). Detailed information on wastewater flows and characteristics is required for successful design, operation, and trouble-shooting if nitrogen removal is to be successful. As a result, design and operational parameters, such as alkalinity requirements, organic loading rates necessary to achieve nitrification, and stoichiometric equivalencies for various reactions have been widely published in order to advance knowledge and improve design and operation.

B. Onsite Wastewater Treatment Systems

Unfortunately, the same cannot be said of onsite nitrogen removal. Much of the published literature does not report data in terms of parameters that can be used to rigorously assess systems, compare them to other sites, and improve design and operation. As an example, the

loading rates on single pass sand filter (SPSF) systems are almost exclusively expressed in terms

of hydraulic loading rates; the most useful information in terms of nitrification, however, would be organic loading rates, which have been reported in a few studies such as Converse, (1999). Alkalinity concentrations are also very rarely monitored in onsite wastewater treatment studies, but are fundamental in assessing the limits on nitrification.

It is certain that onsite nitrogen removal technologies will have requirements similar to large-scale systems for their successful design and operation. Indeed, it is unthinkable that one particular design would work for all the ranges of wastewater flows and strengths, and environmental conditions, encountered in single-family dwellings without design changes and operational adjustments for the particular situation at hand. What is needed are general design and operational parameters for the various onsite technologies so their performance can be adequately assessed and designs and operation improved. Prudence dictates that the following information, at minimum, must be taken into account when assessing performance of onsite nitrogen removal technologies.

1. Wastewater Flows.

Wastewater flows from single family dwellings can range from 8 to 85 gallons per capita per day (gpcd), with an average of approximately 45 gpcd (Ayres Associates, 1993). Thus, depending on the number of occupants in a given dwelling, it is possible that the daily flowrate could range from 16 gallons per day (gpd) to 425 gpd if, for example, only two occupants use discharge the lowest range of flows cited in the literature (8 gpcd) or five occupants the highest range (85 gpcd). This range of flowrates could have a significant effect on nitrification/denitrification processes if they are not incorporated into the design or operational adjustments.

In addition, wastewater is generated by discrete events and typical wastewater hydrographs show that wastewater flow varies widely throughout a 24 hour period (US EPA, 1980). This wide fluctuation in flowrates can also have a significant effect of nitrogen removal processes, as an example in the recycling of nitrified effluent to mix with influent for denitrification. Time dosing with programmable timers can help manage peak flowrates better and thus produce a higher potential for nitrogen removal if that is the treatment objective.

2. Wastewater Characteristics.

BOD₅ concentrations in septic tank effluent have been reported to range from 7 to 480 mg/L (Ayres Associates, 1993). If the wastewater is to be used as the carbon source, there can be much variability in nitrogen removal among different systems if the variability of BOD₅ is not taken

into consideration in design and operation. Concentrations of BOD₅ in the high range could inhibit nitrification while concentrations in the low range could inhibit denitrification. Other wastewater characteristics, such as alkalinity, pH, BOD₅/TKN ratios, temperature, and existence of inhibitors, need to be taken into consideration so the performance of one particular system can be adequately assessed, compared with other sites, and the design and operation improved.

3. Technological Assessment and Design Considerations.

Table 6 summarizes the ranges of select wastewater constituents from septic tank effluent relevant to nitrogen removal that have been cited in the literature. An analysis of these data can shed valuable light on the feasibility and limitations of biological nitrogen removal for the range of conditions that can be encountered. Figures 12 and 13, which show nitrogen removal as a function of initial TKN, alkalinity, and BOD₅, have been developed for the range of BOD₅ values (100-140 mg/L) reported for septic tank effluents with an effluent filter shown in Table 6. The development and use of these figures will be discussed in the following example.

Table 6

Ranges of Concentrations of Select Wastewater Constituents in Septic Tank Effluent*

Constituent	Range of Concentrations Cited in Literature, mg/L	
	Without Effluent Filter	With Effluent Filter
BOD ₅	7-480	100-140
TKN	9-125	50-90
Alkalinity (as CaCO ₃)	---	70-594

* Adapted from Adolfsen Associates (1999), Ayres Associates (1993), Converse (1999), Crites and Tchobanoglous (1998), and Oakley, *et al.* (1998).

EXAMPLE 9. FEASIBILITY OF BIOLOGICAL NITROGEN REMOVAL USING SEPTIC TANK EFFLUENT (WITH EFFLUENT FILTER) AS THE CARBON SOURCE.

Using the data from Table 6, assess the feasibility of biological nitrogen removal for the range of possible conditions cited in the literature. The following conditions are assumed to apply:

1. One half of the alkalinity lost through nitrification is recovered with denitrification.
2. The only limitations to nitrification/denitrification are alkalinity and carbon source (electron donor) requirements.

Solution

1. $BOD_5 = 100 \text{ mg/L}$.

i) Condition I: $TKN = 50 \text{ mg/L}$; Alkalinity = $70 \text{ mg/L as CaCO}_3$

a) Alkalinity Requirements for Nitrification

$$\begin{aligned} \text{Alkalinity Requirements} &= 50 \text{ mg/L TKN} \left(\frac{3.57 \text{ mg as CaCO}_3}{\text{mg TKN}} \right) \\ &= 178.5 \text{ mg/L as CaCO}_3 \end{aligned}$$

\therefore There is insufficient alkalinity and nitrogen removal is nitrification limited. This condition can be clearly seen in Figure 12.

b) Available $\text{NO}_3^- \text{-N}$ for nitrification limited condition.

$$\text{Available } \text{NO}_3^- \text{-N} = 70 \text{ mg/L as CaCO}_3 (1 \text{ mg TKN}) / (3.57 \text{ mg as CaCO}_3) = 20 \text{ mg/L}$$

c) Determine the theoretical $\text{NO}_3^- \text{-N}$ Reduction from BOD_5 .

$$\text{NO}_3^- \text{-N Reduction} = \frac{100 \text{ mg } BOD_5/L}{2.72 \text{ mg } BOD_5/\text{mg } \text{NO}_3^- \text{-N}} = 37 \text{ mg/L}$$

This value is greater than the available $\text{NO}_3^- \text{-N}$ (37 vs. 20 mg/L) since nitrogen removal is nitrification limited. Thus only 20 mg/L $\text{NO}_3^- \text{-N}$ can be denitrified.

d) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 50 \text{ mg/L} - 20 \text{ mg/L} = 30 \text{ mg/L}$$

This value can also be estimated from Figure 12. Note that under the assumptions there would be no residual alkalinity, which could cause significant pH changes in the wastewater and inhibit the biological processes. Nitrogen removal, therefore, could be much worse for the given conditions.

ii) Condition II: TKN = 50 mg/L; Alkalinity = 594 mg/L as CaCO_3

a) Alkalinity Requirements for Nitrification

$$\begin{aligned} \text{Alkalinity Requirements} &= 50 \text{ mg/L TKN} \left(\frac{3.57 \text{ mg as CaCO}_3}{\text{mg TKN}} \right) \\ &= 178.5 \text{ mg/L as CaCO}_3 \end{aligned}$$

Alkalinity Requirements < Available Alkalinity
 \therefore Nitrogen removal is not nitrification limited.

b) Determine the theoretical $\text{NO}_3^- \text{-N}$ Reduction from BOD_5 .

$$\text{NO}_3^- \text{-N Reduction} = \frac{100 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 37 \text{ mg/L}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 50 \text{ mg/L} - 37 \text{ mg/L} = 13 \text{ mg/L}$$

This value can also be read from Figure 12.

iii) Condition III: TKN = 90 mg/L; Alkalinity = 70 mg/L as CaCO_3

a) Alkalinity Requirements for Nitrification

$$\begin{aligned} \text{Alkalinity Requirements} &= 90 \text{ mg/L TKN} \left(\frac{3.57 \text{ mg as CaCO}_3}{\text{mg TKN}} \right) \\ &= 321 \text{ mg/L as CaCO}_3 \end{aligned}$$

b) Available NO_3^- -N for nitrification limited condition.

$$\text{Available } \text{NO}_3^- \text{-N} = 70 \text{ mg/l as CaCO}_3 (1 \text{ mg TKN}) / (3.57 \text{ mg as CaCO}_3) = 20 \text{ mg/L}$$

c) Determine the theoretical NO_3^- -N Reduction.

$$\text{NO}_3^- \text{-N Reduction} = \frac{100 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 37 \text{ mg/L}$$

This value is greater than the available NO_3^- -N since nitrogen removal is nitrification limited. Thus only 20 mg/L NO_3^- -N can be denitrified.

d) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 90 \text{ mg/L} - 20 \text{ mg/L} = 70 \text{ mg/L}$$

iv) Condition IV: TKN = 90 mg/L; Alkalinity = 594 mg/L as CaCO_3

a) Alkalinity Requirements for Nitrification

$$\begin{aligned} \text{Alkalinity Requirements} &= 90 \text{ mg/L TKN} \left(\frac{3.57 \text{ mg as CaCO}_3}{\text{mg TKN}} \right) \\ &= 321 \text{ mg/L as CaCO}_3 \end{aligned}$$

Alkalinity Requirements < Available Alkalinity
 \therefore Nitrogen removal is not nitrification limited.

b) Determine the theoretical NO_3^- -N Reduction from BOD_5 .

$$\text{NO}_3^- \text{-N Reduction} = \frac{100 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 37 \text{ mg/L}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 90 \text{ mg/L} - 37 \text{ mg/L} = 53 \text{ mg/L}$$

2. BOD₅ = 140 mg/L.

i) Condition I: TKN = 50 mg/L; Alkalinity = 70 mg/L as CaCO₃

a) Alkalinity Requirements for Nitrification

$$\text{Alkalinity Requirements} = 178.5 \text{ mg/L as CaCO}_3 \text{ from Part 1.}$$

b) Available NO₃⁻-N for nitrification limited condition.

$$\text{Available NO}_3^- \text{-N} = 20 \text{ mg/L from Part 1.}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 50 \text{ mg/L} - 20 \text{ mg/L} = 30 \text{ mg/L}$$

This can be estimated from Figure 13.

ii) Condition II: TKN = 50 mg/L; Alkalinity = 594 mg/L as CaCO₃

a) Alkalinity Requirements for Nitrification

$$\text{Alkalinity Requirements} = 178.5 \text{ mg/L as CaCO}_3 \text{ from Part 1.}$$

Alkalinity Requirements < Available Alkalinity

∴ Nitrogen removal is not nitrification limited.

b) Determine the theoretical NO₃⁻-N Reduction from BOD₅.

$$\text{NO}_3^- \text{-N Reduction} = \frac{140 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 51 \text{ mg/L}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 50 \text{ mg/L} - 51 \text{ mg/L} = 0 \text{ mg/L}$$

This value can also be read from Figure 13.

iii) Condition III: TKN = 90 mg/L; Alkalinity = 70 mg/L as CaCO₃

a) Alkalinity Requirements for Nitrification

$$\text{Alkalinity Requirements} = 321 \text{ mg/L as CaCO}_3 \text{ from Part 1.}$$

b) Available NO₃⁻-N for nitrification limited condition.

$$\text{Available NO}_3^- \text{-N} = 20 \text{ mg/L from Part 1.}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 90 \text{ mg/L} - 20 \text{ mg/L} = 70 \text{ mg/L}$$

iv) Condition IV: TKN = 90 mg/L; Alkalinity = 594 mg/L as CaCO₃

a) Alkalinity Requirements for Nitrification

$$\text{Alkalinity Requirements} = 321 \text{ mg/L as CaCO}_3 \text{ from Part 1.}$$

Alkalinity Requirements < Available Alkalinity
∴ Nitrogen removal is not nitrification limited.

b) Determine the theoretical NO₃⁻-N Reduction from BOD₅.

$$\text{NO}_3^- \text{-N Reduction} = \frac{140 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 51 \text{ mg/L}$$

c) Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 90 \text{ mg/L} - 51 \text{ mg/L} = 39 \text{ mg/L}$$

This can also be seen on Figure 13.

Comment. These answers can be easily read from Figures 12 and 13. Both figures show that alkalinity controls total nitrogen removal up to a certain minimum value of TKN (nitrification limitation), after which BOD₅ becomes the controlling parameter (denitrification limitation). These results clearly show that biological nitrogen removal is highly dependent on wastewater characteristics, and in the absence of chemical additions and operator control can be expected to vary greatly given the range of TKN, alkalinity, and BOD₅ concentrations shown in Table 6.

EXAMPLE 10. FEASIBILITY OF NITROGEN REMOVAL. An equipment manufacturer makes the following claim for nitrogen removal through biological nitrification/denitrification using septic tank effluent as the carbon source:

Parameter	Septic Tank Effluent	Final Effluent
BOD ₅ , mg/L	125	30
Total-N, mg/L	68	15

Is this claim feasible?

Solution

1. Alkalinity Requirements.

No mention is made of alkalinity. Assuming that 3.57 mg/L as CaCO₃ would be recovered through denitrification, then 3.57 mg/L as CaCO₃ would be needed for nitrification.

∴ Required Alkalinity for Nitrification =

$$= 68 \text{ mg/L TKN} \left(\frac{3.57 \text{ mg as CaCO}_3}{\text{mg TKN}} \right)$$

$$= 243 \text{ mg/L as CaCO}_3$$

2. Determine the quantity of NO_3^- -N that could theoretically be reduced with a BOD_5 of 125 mg/L, assuming the process is not nitrification limited (ie., there is sufficient alkalinity).

Since 30 mg/L BOD_5 was left in the effluent, only 95 mg/L was available as a carbon source for denitrification.

$$\text{NO}_3^- \text{-N Reduction} = \frac{95 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 35 \text{ mg/L}$$

3. Determine residual nitrogen that cannot be removed.

$$\text{Residual N} = 68 \text{ mg/L} - 35 \text{ mg/L} = 33 \text{ mg/L}$$

Comment. Given the data, the manufacturer's claim is unreasonable. Even assuming that all of the BOD_5 could be used as the carbon source, the residual nitrogen would still be 22 mg/L, which is 7 mg/L over the manufacturer's claim:

$$\text{NO}_3^- \text{-N Reduction} = \frac{125 \text{ mg BOD}_5/\text{L}}{2.72 \text{ mg BOD}_5/\text{mg NO}_3^- \text{-N}} = 46 \text{ mg/L}$$

$$\text{Residual N} = 68 \text{ mg/L} - 46 \text{ mg/L} = 22 \text{ mg/L}$$

These results could also have been obtained by inspection of Figure 11. Note again that these results assume there is sufficient alkalinity, which may not always be the case.

IX. EXAMPLES OF ONSITE NITROGEN REMOVAL TECHNOLOGIES

Figures 14 and 15 illustrate the process diagrams of onsite nitrogen removal technologies that have been widely discussed in the literature. Table 1 summarizes each system's removal efficiency and effluent Total-N concentration as cited in the literature.

A. Suspended-Growth Systems (Figure 14)

1. Aerobic Units with Pulse Aeration.

These units are, in principal, extended aeration activated sludge systems in which aeration is periodically stopped or pulsed to promote denitrification. Operational data on these systems is lacking although nitrogen removal efficiencies have been reported to be only in the range of 25-35 percent (Whitmeyer, *et al*, 1991).

A recent study in Los Osos, California for one unit installed at a single family residence showed removal efficiencies to range from 38-61 percent, with effluent Total-N concentrations ranging between 37 and 60 mg/L (California Regional Water Quality Control Board, 1997).

2. Sequencing Batch Reactor (SBR).

The SBR differs generally from aerobic units in that fill-and-draw, and alternating aerobic and anoxic cycles, are created within a single reactor; during the anoxic phase sedimentation takes place and the supernatant is pumped from the reactor. Both endogenous phase bacteria and influent wastewater serve as the carbon source. SBR technology has been demonstrated to be an excellent nitrogen control technology for large-scale systems (US EPA, 1993), but there is a paucity of information for onsite systems.

One proprietary onsite system tested in Florida in an experimental facility exhibited an average of 60% Total-N removal, with effluent Total-N concentrations averaging 15.5 mg/L for an average influent TKN concentration of 38.4 mg/L (Ayres Associates, 1998). Although, unfortunately, no mention was made of alkalinity concentrations in this study, there was approximately 92% nitrification of influent TKN, so the system was apparently not nitrification limited. Nevertheless, since the average influent BOD₅ was 170 mg/L (Ayres Associates, 1998), there was ample BOD₅ for a carbon source to remove all of the Total-N as can be seen in Figure

11. Thus it can be assumed the system was not operating efficiently in terms of denitrification.

3. Sludge Bulking in Suspended-Growth Systems.

All suspended growth processes require secondary sedimentation to remove the bacterial flocs. Although it has not been discussed frequently in the literature on onsite systems, because denitrification is being promoted, there is a potential problem with bulking sludge due to rising N_2 gas as is the case with large-scale systems (Metcalf & Eddy, 1991). Whether this problem could be dealt with adequately in onsite systems has yet to be determined.

B. Attached-Growth Systems (Figure 15)

1. Single Pass Sand Filters (SPSF).

SPSF technology is the most studied of all proposed nitrogen removal technologies. The mechanism of nitrogen removal includes a combination of CBOD removal and nitrification within the sand medium at low organic loadings (low BOD_5/TKN ratio), and subsequent denitrification within anoxic microenvironments in the sand. Total-N removal rates with SPSFs have been quoted in the literature as ranging from 8% to 50% (Converse, 1999; Gold, *et al.*, 1992; Loomis, *et al.*, 2001; Nolte & Associates, 1992; Ronayne, *et al.*, 1982).

The greatest advantage of SPSF technology is in the achievement of nitrification. The percentage of TKN nitrification in SPSF systems has been reported to range between 75% to 96% (Converse, 1999; Gold, *et al.*, 1992; Nolte & Associates, 1992; Ronayne, *et al.*, 1982). The results of the work by Converse (1999) and Gold, *et al.* (1992) suggest that nitrification rates in winter months do not change significantly from summer months in buried filters.

Unfortunately, there is a paucity of sound design data for nitrification based on organic loading rates as was discussed previously. Most of the loading rates have been reported in terms of hydraulic loadings rather than organic loadings. Also, accurate data on measured loadings per unit area based on the type of distribution system used, as opposed to calculated loadings, are difficult to come by (Converse, 1999).

Assuming there is sufficient alkalinity for nitrification, it can be expected that SPSF systems will always be denitrification-limited due to the lack of availability of both a carbon source and anoxic conditions.

2. Recirculating Sand/Gravel Filters (RSF).

RSF technology is also very well studied in the literature. Total-N reduction has been reported to range from 15% to 84% (California Regional Water Quality Control Board, 1997; Gold, *et al.*, 1992; Loomis, *et al.*, 2001; Nolte & Associates, 1992; Oakley, *et al.*, 1999; Piluk and Peters, 1994; Ronayne, *et al.*, 1982). RSFs can achieve high nitrification rates and consistently higher denitrification rates than SPSFs because the nitrified effluent can be recycled back to a recirculation tank where it mixes with wastewater from the septic tank, thus using the incoming wastewater as a carbon source.

As with SPSF systems, the organic loading rates for RSF systems are poorly defined in the literature. The available data suggest that organic loading rates that promote nitrification typically are in the range of 0.002-0.008 lbs. BOD₅/ft²-day (0.01-0.04 kg BOD₅/m²-day) (Crites and Tchobanoglous, 1998). The extent of denitrification can be expected to vary widely since RSF systems have not typically been designed and operated specifically for nitrogen removal.

There is no doubt RSF performance could be significantly improved for nitrogen removal with design and operational changes. The recirculation tank is not generally configured to maximize the mixing of septic tank effluent with RSF effluent or to optimize the formation of anoxic conditions for denitrification. This may have the effect of maintaining permanent aerobic conditions in at least a part of the recirculation tank, especially if high recirculation ratios are used (eg., to prevent drying of the filter bed during low flow periods).

A better design to enhance denitrification recycles the filter effluent to the inlet side of an anoxic recirculation tank, or, an anoxic rock filter, where it mixes with septic tank effluent as shown in Figure 15; the final effluent for discharge is then taken from the filter. This type of system has been termed "classical predenitrification" (Rittmann and McCarty, 2001). The rock filter fosters anoxic conditions by preventing hydraulic short-circuiting and allows denitrifying organisms to grow on the rock surfaces (Whitmeyer, *et al.*, 1991). Systems using this type of design have been reported in the literature (Ayres Associates, 1998; Sandy, *et al.*, 1988). While one system in Florida exhibited a mean Total-N removal of 40% with a mean effluent concentration of 23 mg/L Total-N (Ayres Associates, 1998), another study reported Total-N removals of 80 to 90% and effluent Total-N concentrations ranging from 7-10 mg/L (Sandy, *et al.*, 1988). Sludge accumulation in the rock tank, however, can potentially cause serious operation and maintenance problems.

Operational changes that could improve nitrogen removal include optimizing the recirculation ratio in order to i) minimize dissolved oxygen in the recirculation tank and ii) maximize

denitrification. The recirculation ratio for denitrification must be at least 4:1 or greater in order to remove at minimum 80% of the $\text{NO}_3^- \text{-N}$ (Rittmann and McCarty, 2001). Many RSFs in operation may be below this minimum since the range of recommended recirculation ratios is 3:1 to 5:1 (Crites and Tchobanoglous, 1998). Also, as mentioned above, very high recirculation ratios used to prevent filter drying during low-flow periods can inhibit denitrification because they cause high dissolved oxygen concentrations in the recirculation tank. The optimization of the recirculation ratio for Total-N removal has to be done on a site-specific basis, which likely precludes its being done on many individual onsite systems.

The results of several studies have shown that nitrification rates in winter months in RSF systems in cold climates do change significantly from summer months (57% versus 84%) due to the fact that the RSF was exposed to surface temperatures (Gold, *et al.*, 1992; Ronayne, *et al.*, 1982). In cold climates it is thus recommended that the filter be covered, and the septic tank and recirculating tank be insulated (Loudon, *et al.*, 1984).

3. Single Pass (SPTF) and Recirculating Textile Filters (RTF).

Textile filters are a relatively new technology. The design configurations and operational characteristics of single pass and recirculating (multiple pass) textile filters are essentially the same as for sand/gravel filters with one important exception: hydraulic and organic surface loading rates are much higher due to the specific surface area of the textile medium as shown below (Leverenz, *et al.*, 2001; Crites and Tchobanoglous, 1998):

Media	Specific Surface Area ft^2/ft^3	Hydraulic Loading Rate, gpd/ft^2	Organic Loading Rate, $\text{lbs. BOD}_5/\text{ft}^2\text{-d}$
Coarse Sand for RSF	387 (1270 m^2/m^3)	3-5 (0.12-0.2 m/d)	0.002-0.008 (0.01-0.04 kg $\text{BOD}_5/\text{m}^2\text{-d}$)
Medium Sand for SPSF	2,100 (6890 m^2/m^3)	1.25 (0.05 m/d)	0.000135-0.002 (0.0007-0.01 kg $\text{BOD}_5/\text{m}^2\text{-d}$)
Textile Fabric for SPTF	5,000 (16,404 m^2/m^3)	10 (0.4 m/d)	0.01 (0.05 kg $\text{BOD}_5/\text{m}^2\text{-d}$)
Textile Fabric for RTF	5,000 (16,404 m^2/m^3)	30 (1.2 m/d)	0.03 (0.15 kg $\text{BOD}_5/\text{m}^2\text{-d}$)

Thus it is argued, for example, that RTFs require a much smaller footprint for equivalent attached-growth treatment, with as much as a 30-fold difference in area requirements and 850-

fold difference in media weight from conventional SPSFs (Leverenz, *et al.*, 2001).

Unfortunately, there are a paucity of data for textile filters in relation to both nitrification and Total-N removal. One detailed study of RTFs showed that 83-95% nitrification occurred with as much as 14-29% Total-N removal at an organic loading rate of 0.01 lbs. BOD₅/ft²-day (0.05 kg BOD₅/m²-d), and that only 59-76% nitrification occurred with from 17-31% Total-N removal at an organic loading of 0.03 lbs. BOD₅/ft²-day (0.15 kg BOD₅/m²-d) (Leverenz, *et al.*, 2001). In this study alkalinity did not limit nitrification.

As with sand and gravel filters, more data are needed to adequately characterize textile media in terms of design and operational parameters for both nitrification and Total-N removal.

4. Peat Filters.

Peat filters have been used in a manner similar to single pass sand filters, with similar hydraulic and organic loading rates (Crites and Tchobanoglous, 1998). The results of a few studies show that Total-N removal can be very high, with 80% removal and effluent concentrations less than 10 mg/L Total-N (Crites and Tchobanoglous, 1998). It can be assumed that the peat would serve as a carbon source for reduction of NO₃⁻-N after nitrification has occurred in the filter. A few detailed design and operational data are available in the literature to adequately characterize the various peat media in terms of design and operational parameters for both nitrification and Total-N removal (Geerts, *et al.*, 2001; Crites and Tchobanoglous, 1998).

5. Recirculating Sand/Gravel Filters with Anoxic Filter and External Carbon Source.

This system is similar to the RSF above with an anoxic rock filter except the anoxic rock filter now follows the RSF and an external carbon source is added as shown in Figure 15. Part of the RSF effluent is recycled to the recirculation tank, and another part is discharged to the anoxic rock filter where the external carbon source is added. One detailed study in Rhode Island on pilot scale systems showed the following results for two different external carbon sources (Gold, *et al.*, 1989):

External Carbon Source for RSF Anoxic Filter System	Mean Total-N % Removal	Mean Effluent Total-N, mg/L
Methanol	74	13
Ethanol	80	10

As mentioned previously, the authors concluded that, because of chemical handling and operational requirements of the external carbon source, the system could probably best be handled by a wastewater management district or a private O & M contractor (Gold, *et al.*, 1989).

6. RUCK System.

The RUCK system, a version of which is shown in Figure 15, is a proprietary system that uses source separation for nitrification and denitrification. Separate collection systems are designed for greywater and blackwater, with each having its own septic tank. The original system was configured as follows (Laak, 1986): The blackwater, originally defined as wastewater from toilets, showers and baths, was discharged to an SPSF for nitrification and then passed to an anoxic rock filter or tank; the greywater, defined as kitchen and laundry wastewater, passed from its septic tank directly to the anoxic rock filter or tank, where it served as the carbon source. There are newer configurations that have been used (Loomis, 2002), but there is a paucity of information on them in the published literature.

While the RUCK system has often been cited as a potential technology for nitrogen removal, there is also a paucity of performance data that have been published. While the process is intended to provide at least 80% Total-N removal (Laak, 1986), results from a few studies have shown much poorer removal rates of from 29-54% Total-N removal (Brooks, 1996; Gold, *et al.*, 1989). The variability in nitrogen removal efficiency is no doubt due to the complexity of the system, the variability of the quality of greywater, and the need to adjust the operation to site-specific conditions, as is the case with a RSF system. Indeed, the RUCK system is even more complicated than a RSF and likely requires significant adjustment to blackwater and greywater characteristics and site conditions.

EXAMPLE 12. ASSESSMENT OF RUCK SYSTEM FOR NITROGEN REMOVAL.

Determine the feasibility of using a RUCK system for nitrogen removal for a single family residence for conditions of greywater with and without kitchen food waste. The following conditions are assumed to apply:

1. The contributions of BOD₅ and Total-N from the different wastewater sources within a residence are assumed to be the averages published by the US EPA (1980):

Table A

Wastewater Source	gpd/capita	BOD₅ grams/capita-day	Total-N grams/capita-day
Kitchen Food-Waste Grinder	2	18.0	0.6
Toilet	16	16.7	8.7
Basins, sinks, appliances	29	28.5	1.9
Total	47	63.2	11.2

2. The distribution of wastewater flows among the different sources is assumed to be the following (Crites and Tchobanoglous, 1998; Laak, 1986):

Table B

Wastewater Source	Percent of Total Flow w/ Food-Waste Grinder	Percent of Total Flow w/o Food-Waste Grinder
Bathing (bath/shower)	20	21
Dishwashing	7	7
Laundry	20	21
Kitchen Food-Waste Grinder	4	0
Faucets	10	11
Toilet	34	35
Miscellaneous	5	5

3. Calculations are to be based on per capita.

Solution

1. Greywater with kitchen food-waste.

a) Determine flowrates of blackwater and greywater.

i) Blackwater.

Toilet wastewater = (16 gpd/capita)(1 person) = 16 gpd (Table A)

Basins, sinks, appliances = 29 gpd/capita (Table A)

$$\text{Bathing fraction from basins, sinks, appliances} = \frac{20}{(20 + 7 + 20 + 10)} = 0.35 \text{ (Table B)}$$

$$\text{Bathing wastewater} = 0.35(29 \text{ gpd/capita})(1 \text{ person}) = 10.1 \text{ gpd}$$

$$\therefore \text{Total Blackwater Flowrate} = 16 + 10.1 = 26.1 \text{ gpd}$$

ii) Greywater.

$$\begin{aligned} \text{Total Greywater Flowrate} &= \text{Kitchen Food-Waste} + 0.65(\text{Basins, sinks, appliances}) \\ &= [2 \text{ gpd/capita} + 0.65(29 \text{ gpd/capita})](1 \text{ person}) = 20.8 \text{ gpd} \end{aligned}$$

b) Determine the Total-N that must be nitrified.

The Total-N in the blackwater includes toilet wastewater and wastewater from bathing.

i) Toilet wastewater

$$\text{Total-N (TKN)} = (8.7 \text{ grams/capita-day})(1 \text{ person}) = 8.7 \text{ grams/day}$$

ii) Bathing wastewater

It is assumed the bathing wastewater contains nitrogen in the form of TKN.

$$\text{Total-N (TKN)} = 0.35(1.9 \text{ grams/capita-day})(1 \text{ person}) = 0.7 \text{ grams/day}$$

iii) TKN in Blackwater

Assume that 20% of TKN is removed in the blackwater septic tank

$$\therefore \text{TKN} = 0.8[8.7 + 0.7] = 7.5 \text{ grams/day}$$

c) Determine the TKN concentration in the blackwater.

$$\text{TKN} = \frac{7.5 \text{ grams/day}}{(16 + 10.1 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 76.0 \text{ mg/L}$$

d) Determine the alkalinity requirements for nitrification of blackwater within the SPSF.

$$\text{Alkalinity Requirement} = \frac{(76.0 \text{ mg/L TKN})(7.1 \text{ mg CaCO}_3)}{\text{mg TKN}} = 540 \text{ mg/L as CaCO}_3$$

This alkalinity requirement is significant and nitrification limitation is a likely possibility.

e) Determine the BOD₅/TKN ratio for nitrification.

$$\text{BOD}_5 \text{ for bathing wastewater} = 0.35(28.5 \text{ grams/day}) = 10 \text{ grams/day}$$

$$\text{BOD}_5 \text{ from toilet} = 16.7 \text{ grams/day}$$

Assume 50% of BOD₅ is removed in the septic tank.

$$\therefore \text{BOD}_5/\text{TKN} = \frac{(0.5)[10 + 16.7 \text{ grams/day}]}{7.5 \text{ grams/day}} = 1.78$$

This value is within the range where nitrification should not be overly affected by heterotrophic oxidation of CBOD as discussed previously (See Figure 4).

The concentration of BOD₅ in the septic tank effluent would be:

$$\text{BOD}_5 = \frac{(0.5)[10 + 16.7 \text{ grams/day}]}{(16 + 10.1 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 135 \text{ mg/L}$$

f) Determine the potential NO₃⁻-N reduction from the BOD₅ in the greywater.

Because there are unequal volumes of blackwater and greywater, NO₃⁻-N reduction from BOD₅ must be determined on a mass rather than a concentration basis.

$$\begin{aligned} \text{BOD}_5 \text{ from basins, sinks, appliances} &= 0.65(28.5 \text{ grams/capita-day})(1 \text{ person}) \\ &= 18.5 \text{ grams/day} \end{aligned}$$

$$\text{BOD}_5 \text{ from food-waste grinder} = 18.0 \text{ grams/day}$$

$$\text{BOD}_5 \text{ in raw greywater} = 36.5 \text{ grams/day}$$

Assume that 50% of this BOD₅ is removed in the greywater septic tank.

$$\therefore \text{Potential NO}_3^- \text{-N Reduction} = \frac{(0.5)(36.5 \text{ grams BOD}_5/\text{day})}{2.72 \text{ grams BOD}_5/\text{gram NO}_3^- \text{-N}} = 6.7 \text{ grams/day}$$

∴ Not all of the NO₃⁻-N (7.5 grams/day) originally derived from the TKN in the blackwater can be removed using the greywater with food waste as a carbon source. (This result would of course be different if less than 50% of the BOD₅ were removed in the greywater septic tank.

g) Determine the residual nitrogen in the greywater that cannot be removed.

$$\begin{aligned} \text{Remaining Total-N from basins, sinks, appliances} &= 0.65(1.9 \text{ grams/capita-day})(1 \text{ person}) \\ &= 1.2 \text{ grams/day} \end{aligned}$$

$$\text{Remaining Total-N from food-waste grinder} = 0.6 \text{ grams/day}$$

$$\text{Residual N}_{\text{greywater}} = \frac{(0.6 + 1.2 \text{ grams/day})}{(2 + 18.8 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 22.8 \text{ mg/L}$$

h) Determine the final effluent combined (blackwater & greywater) Total-N concentration.

$$\text{Remaining Total-N from blackwater} = 0.8 \text{ mg/L}$$

$$\text{Total-N}_{\text{effluent}} = \frac{(0.8 + 0.6 + 1.2 \text{ grams/capita-day})}{(2 + 29 + 16 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 14.6 \text{ mg/L}$$

2. Greywater without kitchen food-waste grinder.

a) Determine flowrates of blackwater and greywater.

i) Blackwater.

$$\text{Toilet wastewater} = 16 \text{ gpd}$$

$$\text{Basins, sinks, appliances} = 29 \text{ gpd/capita (Table A)}$$

$$\text{Bathing fraction from basins, sinks, appliances} = \frac{21}{(21 + 7 + 21 + 11)} = 0.35 \text{ (Table B)}$$

$$\text{Bathing wastewater} = 0.35(29 \text{ gpd/capita})(1 \text{ person}) = 10.1 \text{ gpd}$$

$$\therefore \text{Total Blackwater Flowrate} = 16 + 10.1 = 26.1 \text{ gpd}$$

ii) Greywater.

$$\text{Total Greywater Flowrate} = 0.65(\text{Basins, sinks, appliances}) = 0.65(29 \text{ gpd}) = 18.8 \text{ gpd}$$

b) Total-N that must be nitrified.

$$\text{Total-N (TKN)} = 7.1 \text{ grams/day (Part 1)}$$

c) TKN concentration in the blackwater.

$$\text{TKN} = 76.0 \text{ mg/L}$$

d) Alkalinity requirements for nitrification of blackwater within the SPSF.

$$\text{Alkalinity Requirement} = 540 \text{ mg/L as CaCO}_3$$

e) BOD₅/TKN ratio for nitrification.

$$\text{BOD}_5/\text{TKN} = 1.78$$

f) Determine the potential NO₃⁻-N reduction from the BOD₅ in the greywater.

Because there are still unequal volumes of blackwater and greywater, NO₃⁻-N reduction from BOD₅ must be determined on a mass basis.

$$\text{BOD}_5 \text{ from basins, sinks, appliances} = 0.65(28.5 \text{ grams/day}) = 18.5 \text{ grams/day}$$

$$\text{Available BOD}_5 \text{ in greywater} = 18.5 \text{ grams/day}$$

Assume 50% BOD₅ removal in greywater septic tank

$$\therefore \text{Potential NO}_3^- \text{-N Reduction} = \frac{0.5(18.5 \text{ grams BOD}_5/\text{day})}{2.72 \text{ grams BOD}_5/\text{gram NO}_3^- \text{-N}} = 3.4 \text{ grams/day}$$

$$\text{Percent Removal of NO}_3^- \text{-N} = \frac{3.4 \text{ grams/day}}{7.1 \text{ grams/day}} = 0.48 = 48\%$$

$$\text{Residual Total-N in Blackwater} = 7.1 - 3.4 = 3.7 \text{ grams/day}$$

g) Determine the residual nitrogen in the greywater that cannot be removed.

$$\text{Residual N}_{\text{greywater}} = \frac{1.2 \text{ grams/day}}{(18.8 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 16.9 \text{ mg/L}$$

h) Determine the residual nitrogen in the final wastewater effluent that cannot be removed.

$$\text{Total-N}_{\text{effluent}} = \frac{(1.2 + 3.4 \text{ grams/day})}{(45 \text{ gpd})(3.78 \text{ L/gal})(1 \text{ gram}/1000 \text{ mg})} = 27.0 \text{ mg/L}$$

Comment. This example clearly shows the complexity of the RUCK system. While the system may have sufficient carbon in the greywater with food waste for significant Total-N removal for the average conditions used, the alkalinity demand for nitrification of the blackwater is excessive and probably could not be met in most wastewaters. Since the filter used in the RUCK system for nitrification is single pass, the alkalinity lost cannot be recovered to minimize nitrification alkalinity demands; the system would thus likely be nitrification-limited in most cases unless alkalinity was added from an external source. The mixing of blackwater and greywater for anoxic denitrification is critical since different volumes of each are produced on a daily basis. The percent removal of BOD₅ in the greywater septic tank can have a significant effect on available carbon for denitrification. It is interesting to note that the absence of food waste in the greywater also has a significant effect on the availability of carbon for denitrification. It is obvious that adequate performance of a RUCK system requires site-specific monitoring and operational adjustments. Also note that there may be different configurations of the RUCK system that have been used that may have different effects on operation (Loomis, 2002).

C. Shallow Trench and Subsurface Drip Distribution Systems.

The use of either shallow trench or subsurface drip distribution (SDD) systems has been proposed as an alternative means to remove Total-N in the soil column (Ayres Associates, 1998; Crites and Tchobanoglous, 1998; Oakley, *et al.*, 1998, 1999). Both systems have the potential to promote nitrogen uptake by plant roots if effluent was discharged directly within the root zone. There is also a potential that both systems within the 'A' horizon of the soil could promote denitrification of nitrified effluents if there was sufficient organic matter present, either naturally or added, and if the conditions were conducive for denitrification (ie., anoxic). This type of denitrification has been demonstrated with the use of a reactive porous media barrier using sawdust as a carbon source, which was used to denitrify nitrified septic tank effluents percolating through the soil column (Robertson and Cherry, 1995).

To date, the results on the use of shallow trenches or SDD systems for onsite nitrogen removal is mixed at best, with removal efficiencies of Total-N ranging from 0 to 40% (Ayres Associates, 1998; Bohrer and Converse, 2001; Oakley, *et al.*, 1998, 1999). Coupling nitrogen loadings with plant uptake requires significant operational monitoring and adjustment. Denitrification, if it is desired, cannot be easily controlled within a trench system or the soil column as it can within a treatment reactor above ground. Monitoring of nitrogen removal in the soil column is also a significant problem since lysimeter systems have to be used, and they require some degree of sophistication in installation and sample collection (Oakley, *et al.*, 1999).

D. Proprietary Technologies.

There are any number of proprietary technologies and devices that are promoted for nitrogen removal, and it is beyond the scope of this paper to discuss them. None to date, however, have been found to offer a simple solution to the complex problem of biological nitrogen removal. The case of the septic tank with recirculating trickling filter (STRTF) is an example of what oftentimes occurs with proprietary nitrogen removal technologies.

The STRTF was originally promoted as a promising solution to onsite nitrogen removal (Graydon, *et al.*, 1995; Brooks, 1996; Crites and Tchobanoglous, 1998). A STRTF system, which is shown in Figure 15, consists of a septic tank fitted with a small trickling filter with plastic trickling filter media. A recirculation pump confined within a screened vault recycles the septic tank effluent to the trickling filter. Nitrification and CBOD reduction occur within the trickling filter as the effluent is internally recirculated. As the trickling filter effluent returns to

the influent side of the septic tank, it can be denitrified under anoxic conditions using the influent wastewater as a carbon source.

Preliminary data showed that an STRTF system was capable of removing Total-N with a removal efficiency of from 66 to 78 percent (Graydon *et al.*, 1995; Crites and Tchobanoglous, 1998). Later studies concluded, however, that while the technology held promise, more research and development work needed to be done in order to evaluate process performance and reliability under varying flowrates, wastewater strengths, and environmental conditions, and that the level of operational adjustment and monitoring required for site-specific conditions would probably preclude its use for individual family residences (County of Butte, 1998; Oakley, *et al.*, 1996). The technology was subsequently abandoned by the manufacturer.

Given the nature of biological nitrification/denitrification and the inherent variability of onsite wastewater flowrates and characteristics, all technologies for onsite nitrogen removal, whether proprietary or not, will require some degree of monitoring and operational adjustment for site-specific conditions. Even assuming adequate monitoring and operation, Total-N removal might be poor not as a result of any particular technology, but as a result of wastewater characteristics, as has been shown in previous examples. Nitrogen removal has been widely successful in large-scale wastewater treatment plants as a result of continuous operator attention and chemical addition if necessary. Whether this type of success can be accomplished in onsite wastewater treatment remains to be seen. Meanwhile, the search continues for an alternative or innovative technology that will solve the problem of onsite nitrogen removal.

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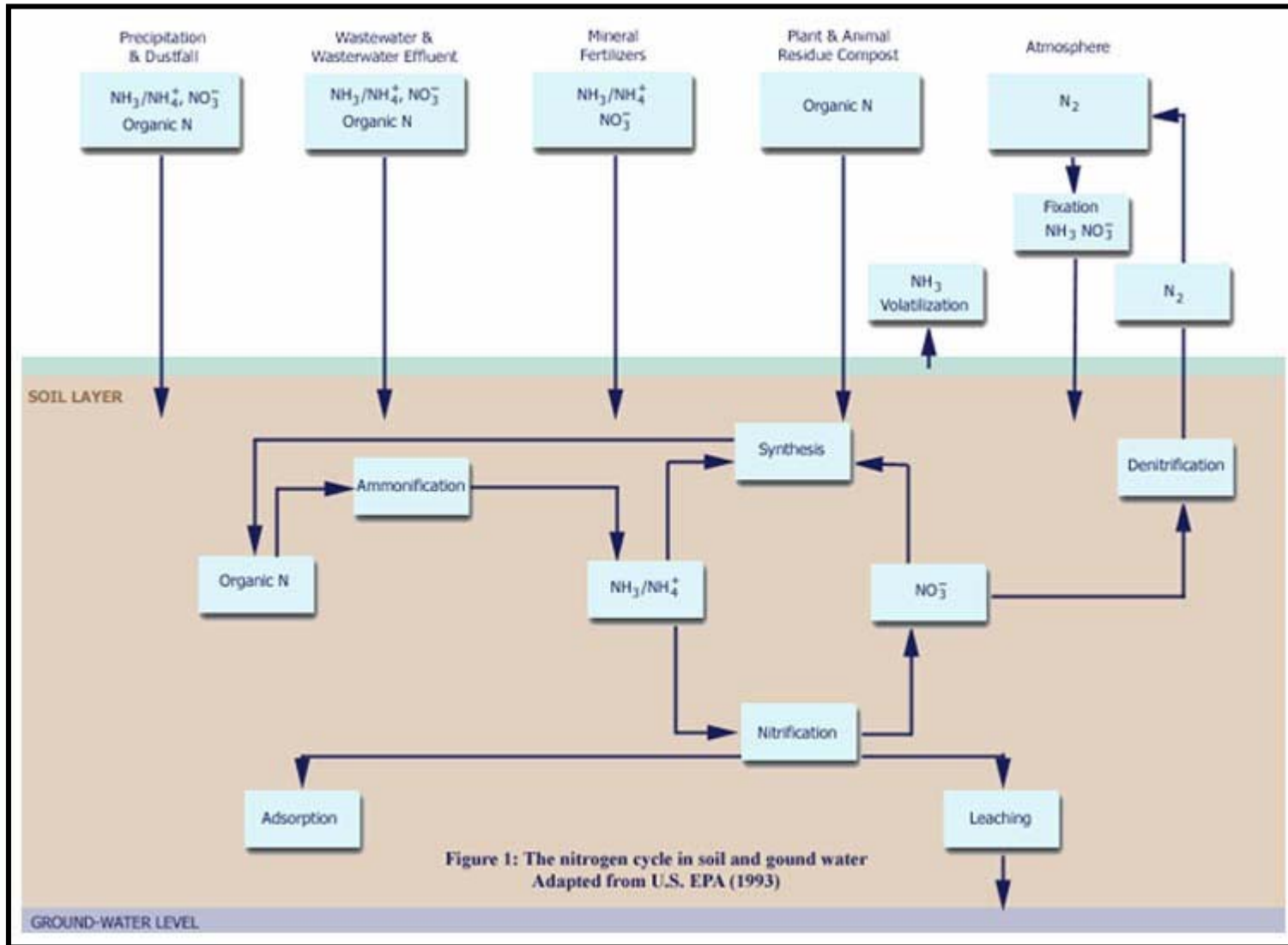
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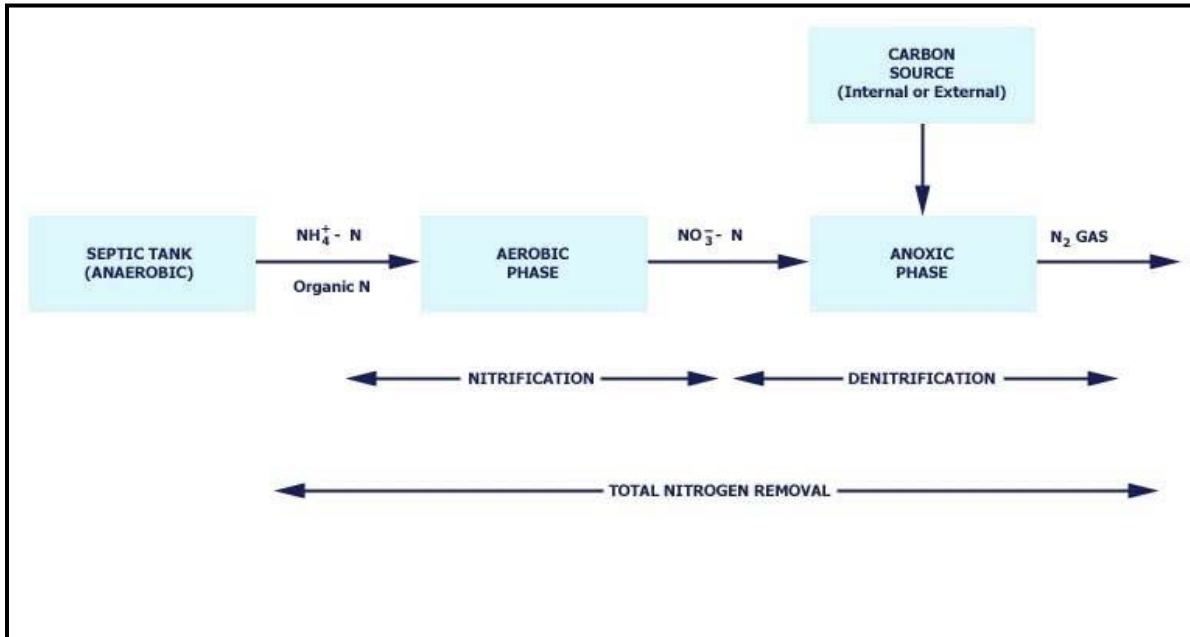


Figure 2: Biological Nitrification / Denitrification in Onsite Wastewater Treatment
Adapted from Gold, et al. (1989)

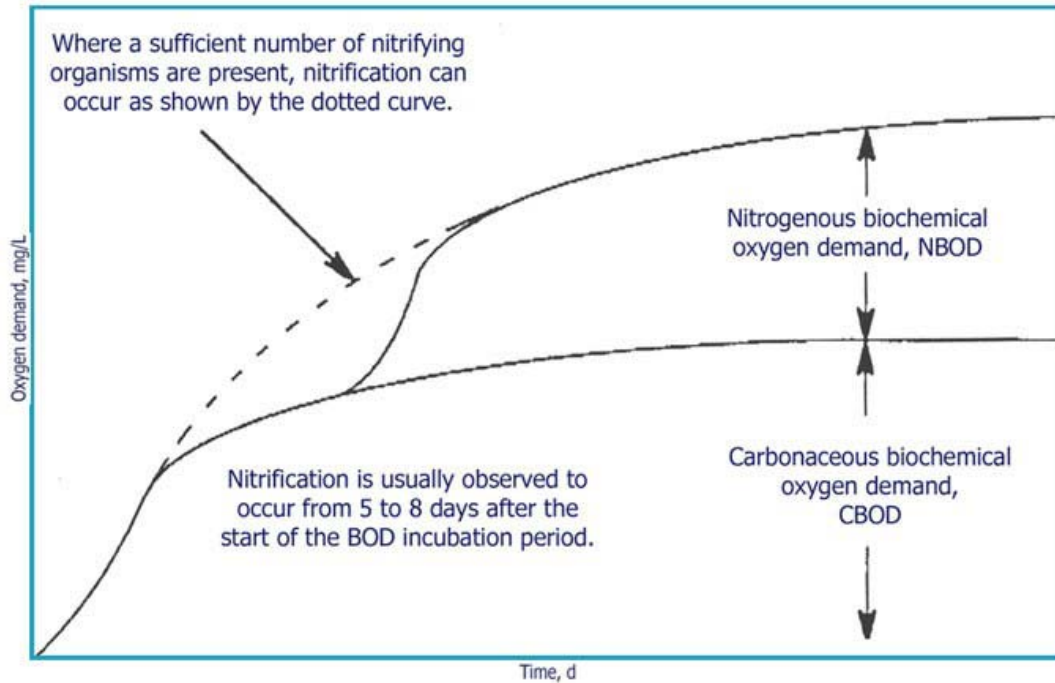
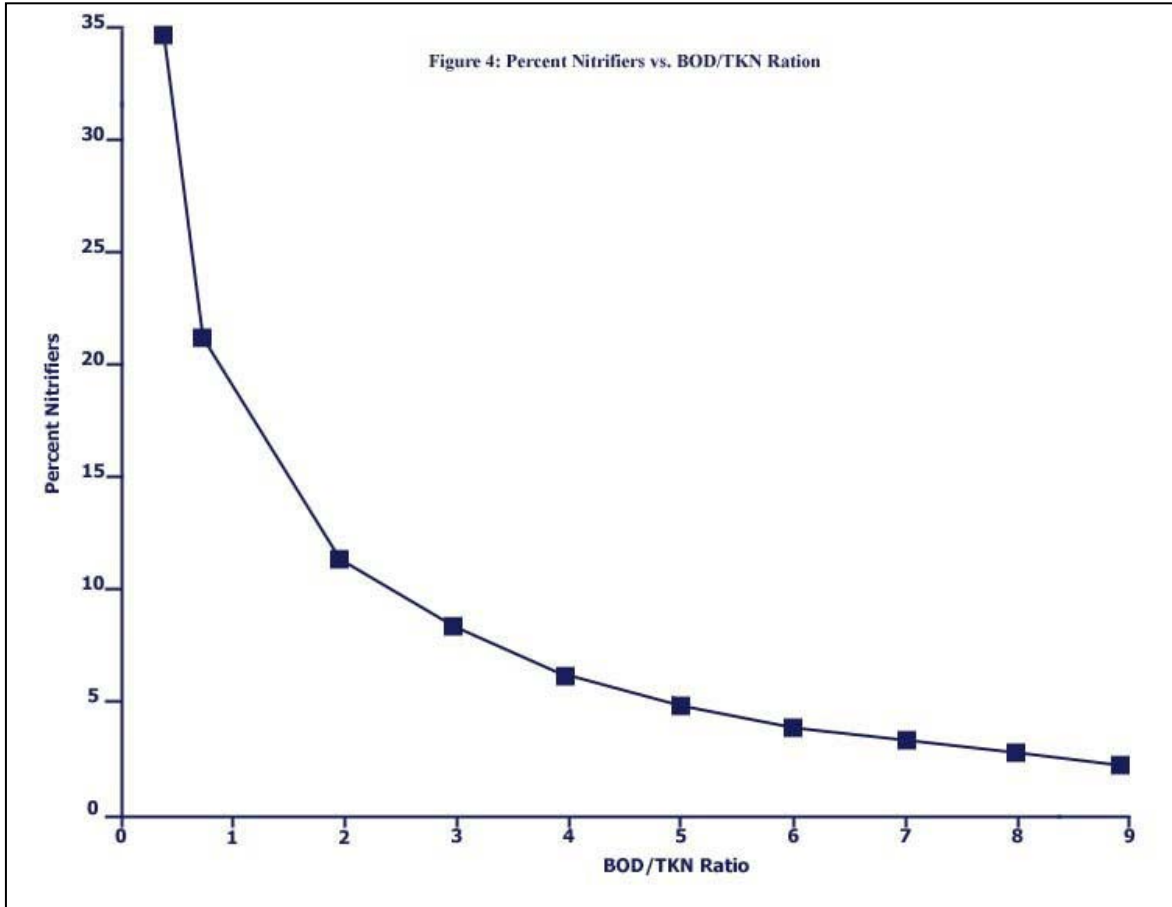


Figure 3: Carbonaceous and Nitrogenous Biochemical Oxygen Demand
Adapted from Metcalf and Eddy (1991)



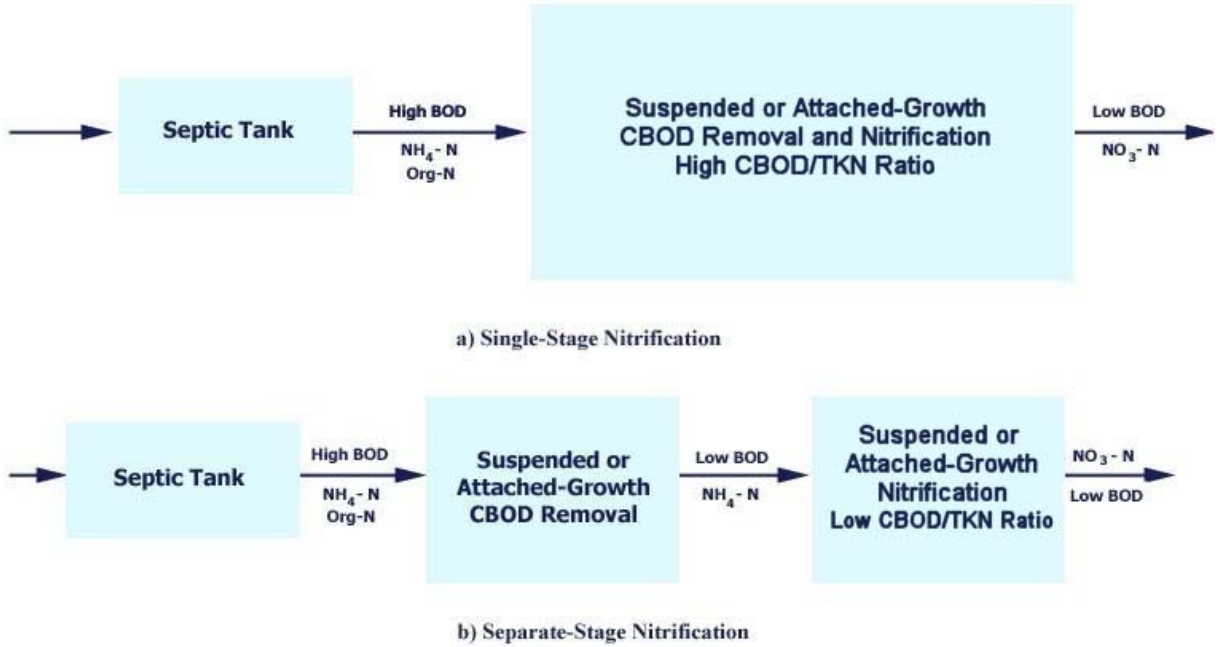


Figure 5: Single-Stage and Separate-Stage Nitrification

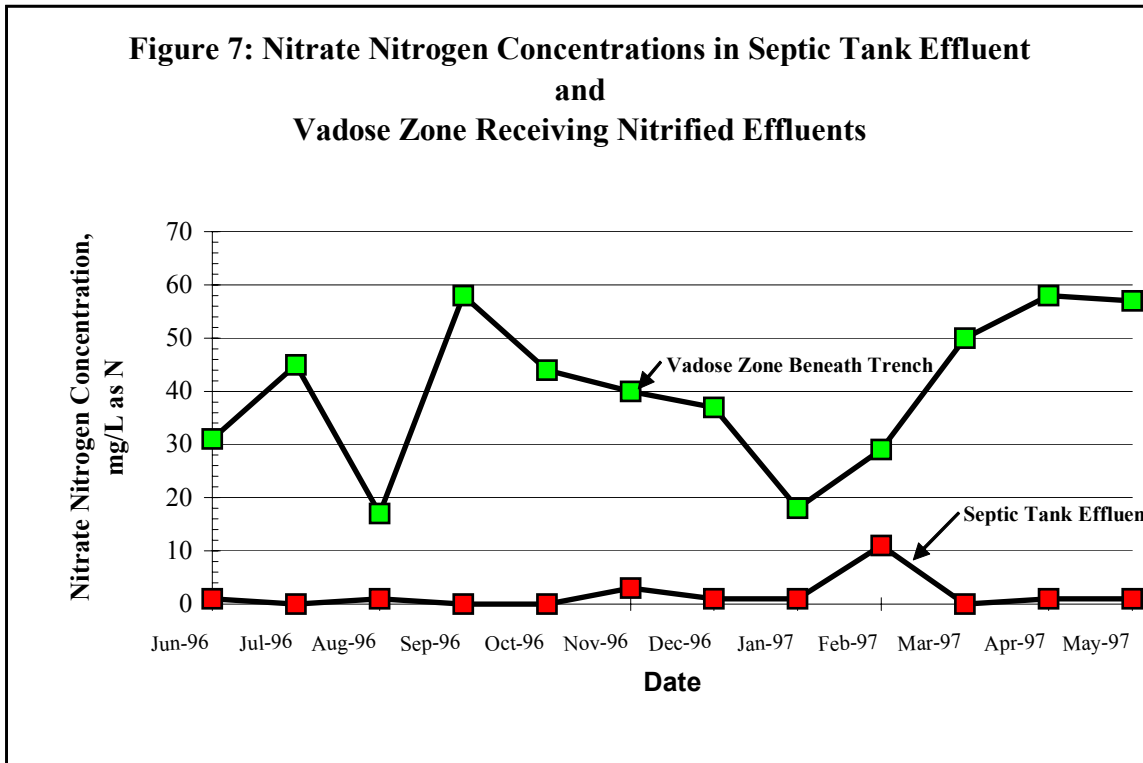
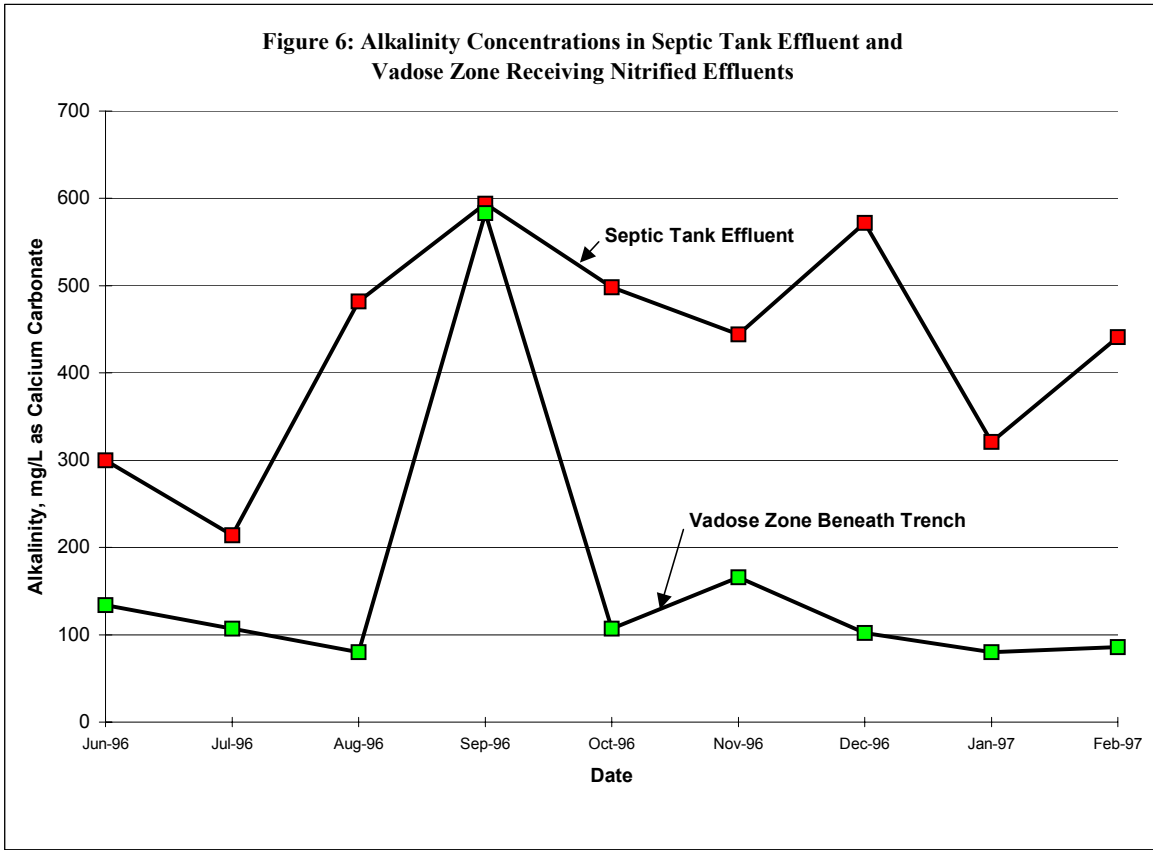
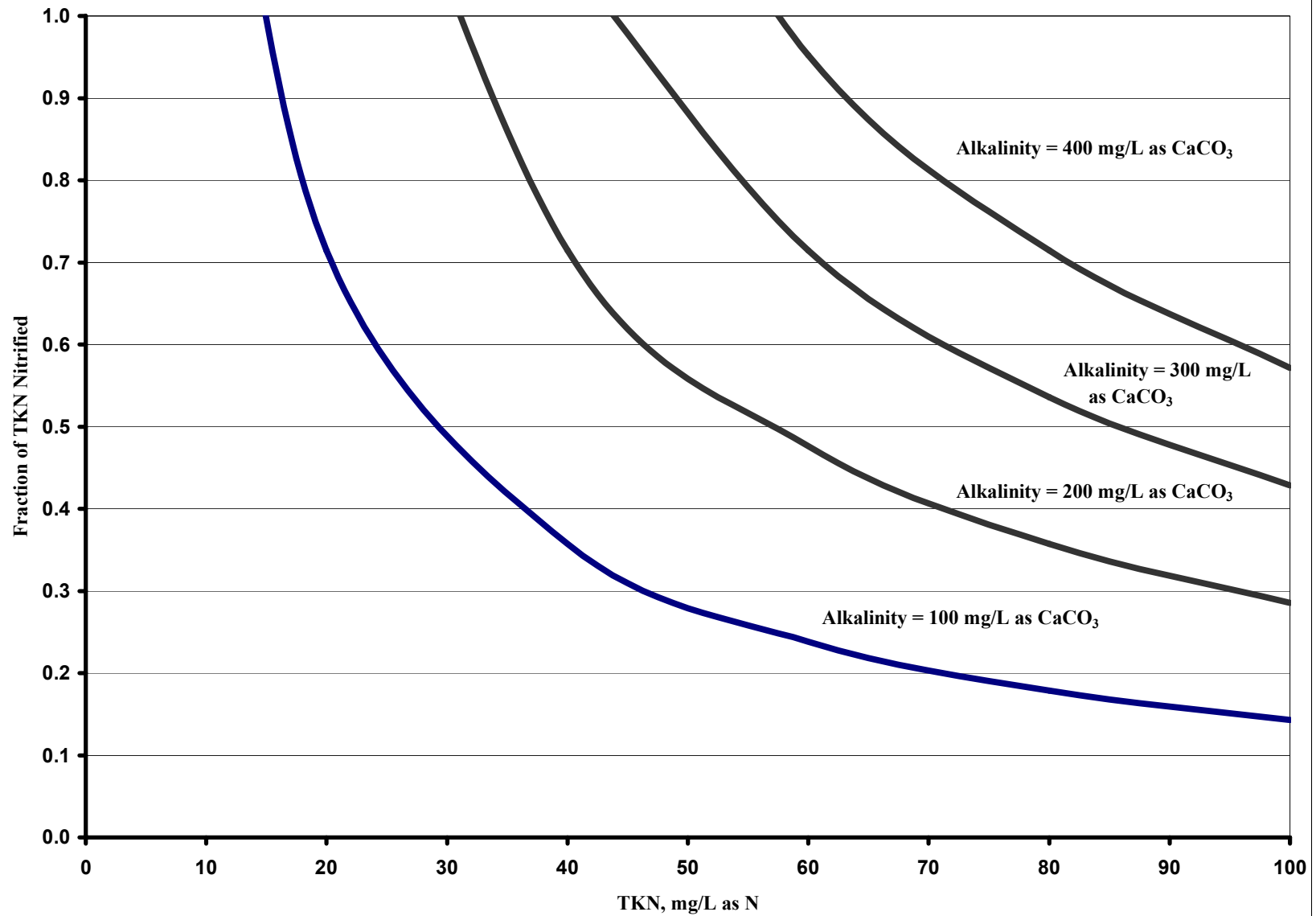
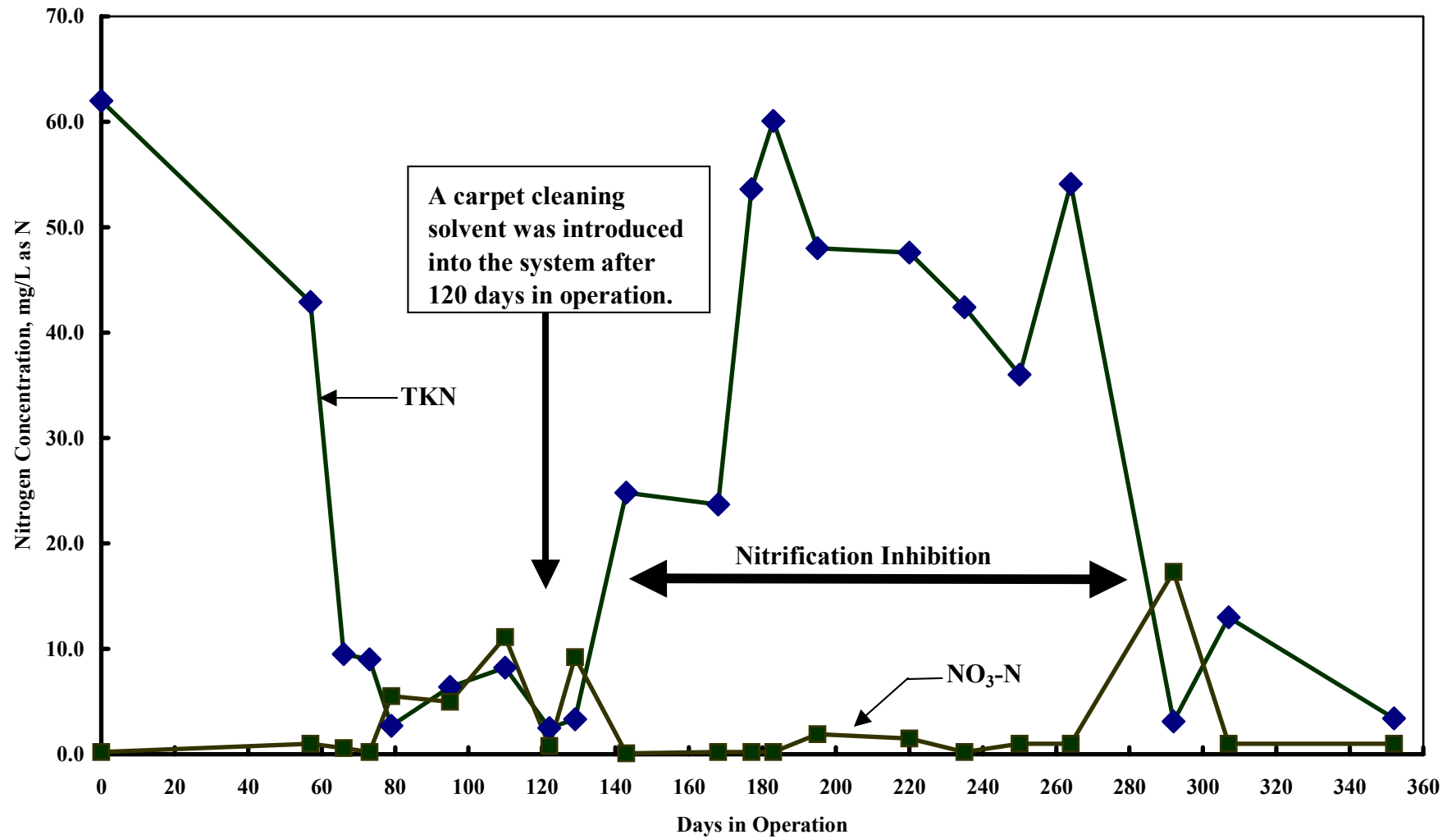
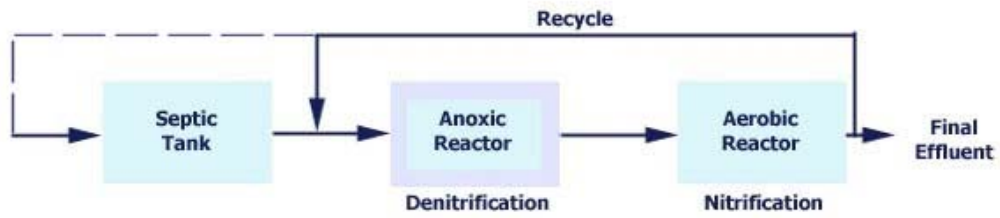


Figure 8: Nitrification as a Function of Initial TKN and Alkalinity

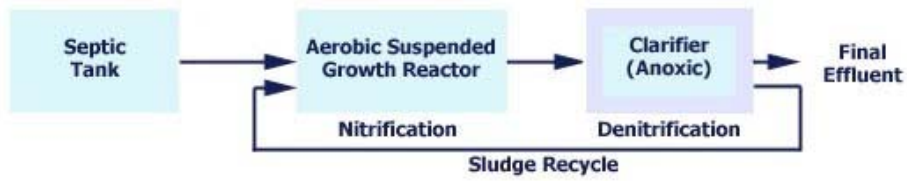


**Figure 9 Nitrogen Removal in a Septic Tank Recirculating Trickling Filter System
After Oakley, *et al.* (1996).**

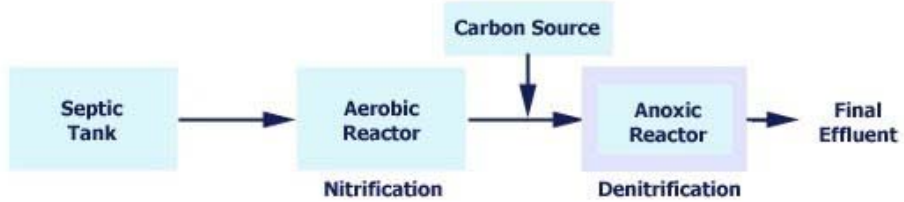




a) Wastewater as a Carbon Source



b) Bacterial Cells as Carbon Source



c) External Carbon Source

Figure 10: Heterotrophic Denitrification Configurations

Figure 11: Nitrogen Removal as a Function of Initial TKN and Wastewater BOD₅ When Wastewater is Used as Carbon Source.

(It is assumed there is sufficient alkalinity, the initial TKN is nitrified, and that the stoichiometric equivalency is 4.0 mg BOD_L/mg NO₃⁻-N = 2.72 mg BOD₅/mg NO₃⁻-N for $k = 0.23 \text{ d}^{-1}$.)

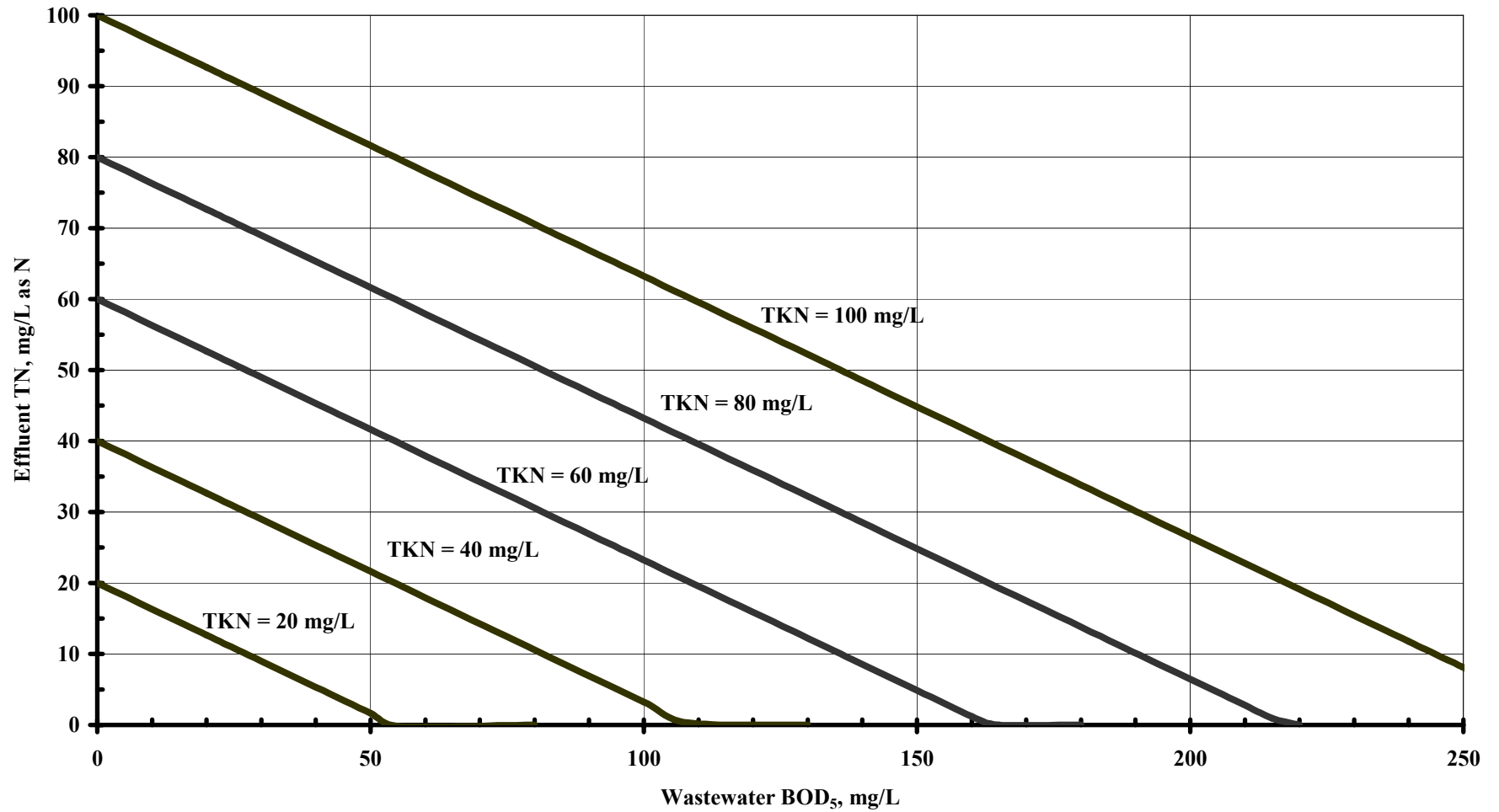


Figure 12 Nitrogen Removal Using Wastewater as Carbon Source as a Function of TKN and Alkalinity

(It is assumed the $BOD_5 = 120$ mg/L, that 50% of the alkalinity lost by nitrification is recovered in denitrification, and that the stoichiometric equivalency is 4.0 mg BOD_1 /mg NO_3^- -N = 2.72 mg BOD_5 /mg NO_3^- -N for $k = 0.23$ d⁻¹.)

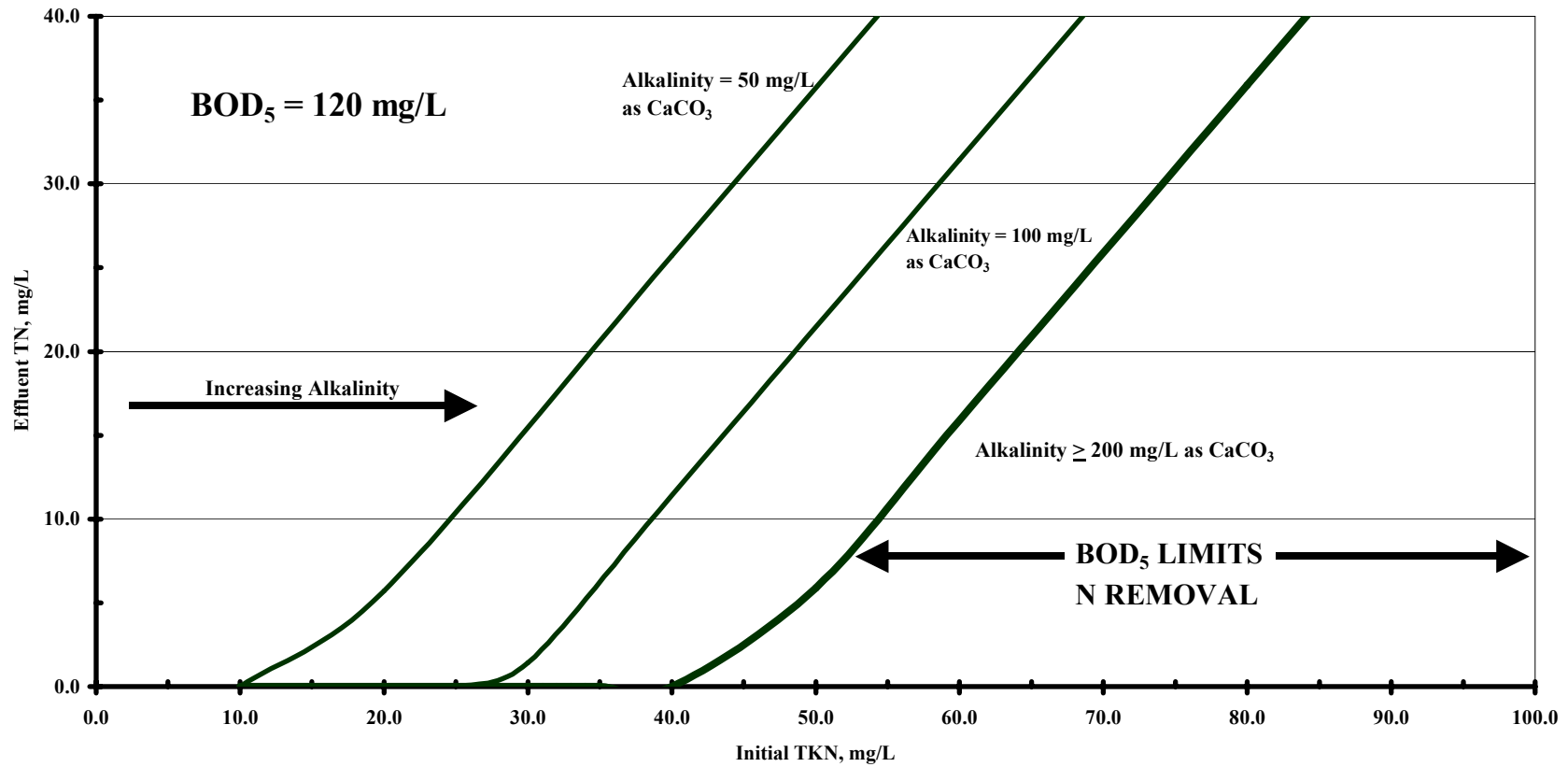
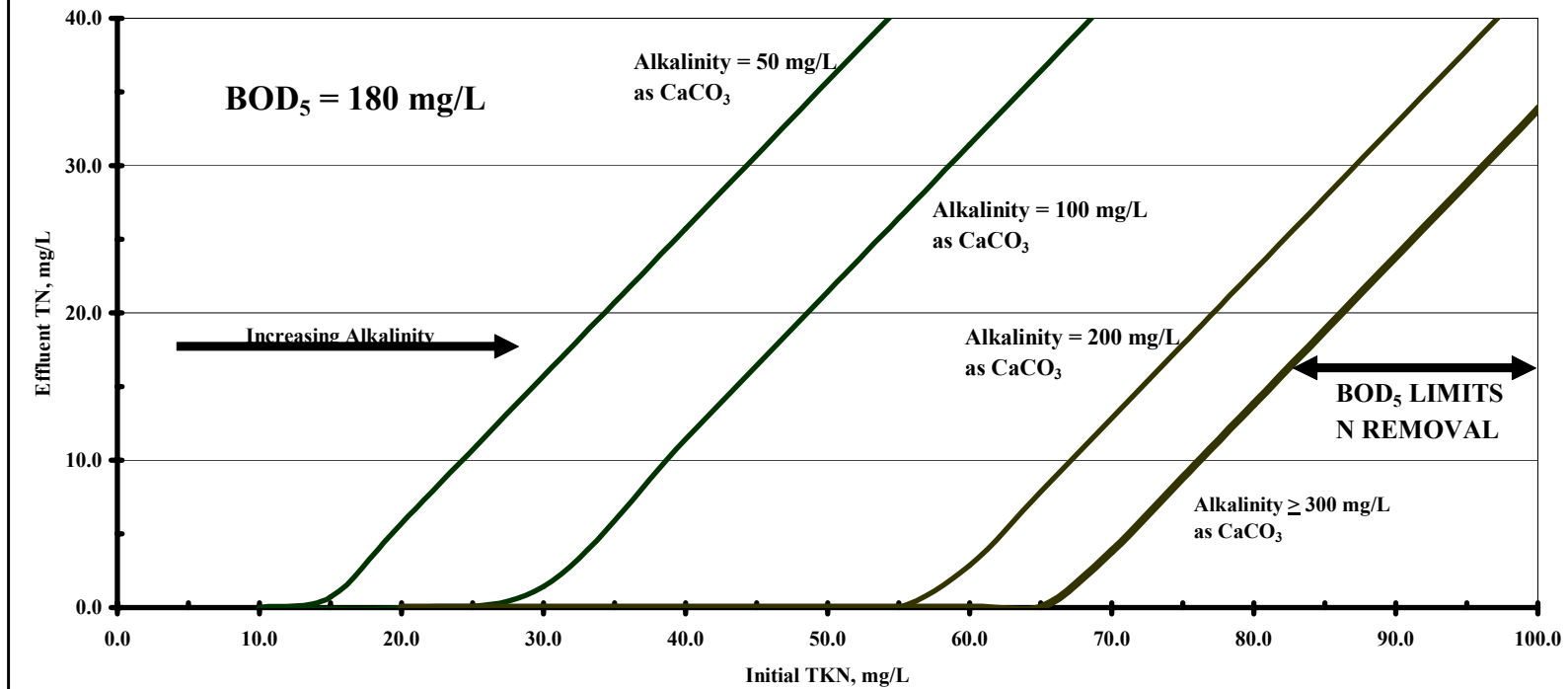


Figure 13 Nitrogen Removal Using Wastewater as Carbon Source as a Function of TKN and Alkalinity

(It is assumed the $BOD_5 = 180$ mg/L, that 50% of the alkalinity lost by nitrification is recovered in denitrification, the stoichiometric equivalency is 4.0 mg BOD_L /mg NO_3^- -N = 2.72 mg BOD_5 /mg NO_3^- -N for $k = 0.23$ d⁻¹.)



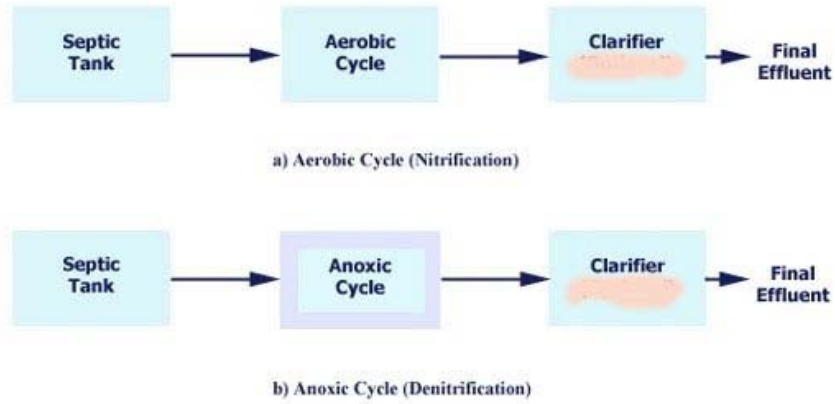
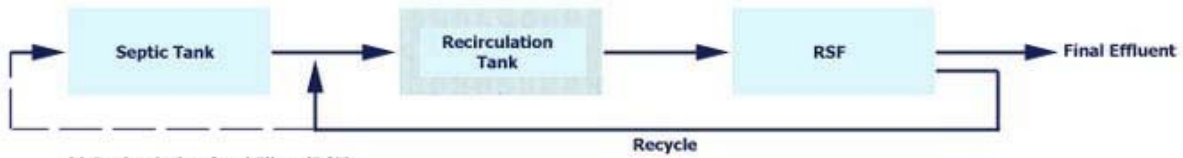


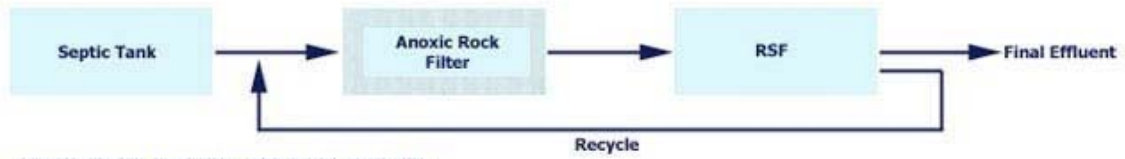
Figure 14: Process Diagram Suspended-Growth Systems: Pulse Aeration and Sequencing Batch Reactors with Aerobic/Anoxic Cycles



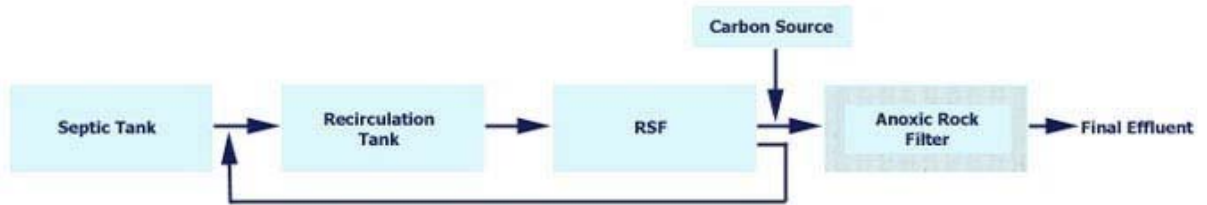
a) Single Pass Sand Filter (SPSF)



b) Recirculating Sand Filter (RSF)

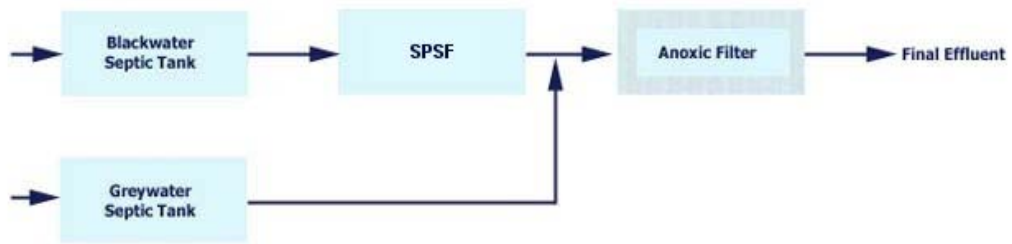


c) Recirculating Sand Filter with Anoxic Rock Filter

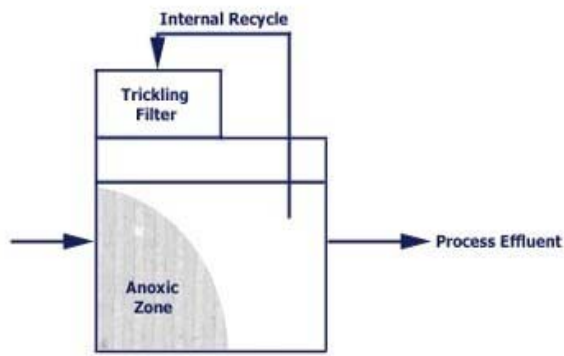


d) RSF with Anoxic Rock Filter and External Carbon Source

Figure 15: Process Diagrams for Attached-Growth Systems



e) RUCK System



f) Septic Tank/Recirculating Trickling Filter

Figure 15 (continued): Process Diagram for Attached-Growth Systems