University Curriculum Development for Decentralized Wastewater Treatment

Constructed Wetlands: Design Approaches

Text

By

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1. INTRODUCTION

Constructed wetlands are attached-growth biological filters that are distinguished from other treatment processes by the use of plants that are adapted to grow in saturated environments. With the inclusion of plants, treatment wetlands have the appearance of a wetland habitat, and employ many of the same biological processes found in natural wetland ecosystems.

Natural wetlands have historically been used as receiving bodies for effluent discharges, and their use in the United States has been documented as early as 1912 (Kadlec R.H. and Knight R., 1996). With the advent of wetland regulation under the Clean Water Act, natural wetlands are now rarely used for secondary treatment of wastewater, although in certain situations, treated effluent has been applied to natural wetlands to stabilize the wetland hydrology and create wildlife habitat (United States Environmental Protection Agency, 1993a).

Unlike most treatment processes that focus on a single treatment mechanism or type of pollutant, wetlands use numerous interdependent, symbiotic processes for concurrent removal of several different types of pollutants. For instance, sedimentation, filtration, microbial degradation, and volatilization are among the treatment processes that occur simultaneously in constructed wetlands.

1.1 TYPES OF CONSTRUCTED WETLANDS

Constructed wetlands are man-made systems that have been designed to emphasize specific characteristics of the wetland habitat in order to improve their treatment capacity. To date, three major types of constructed wetland reactors have been developed and are in widespread use:

- Free Water Surface (FWS) wetlands have exposed water bodies and are similar to natural marshes.
- Vegetated Submerged Bed (VSB) wetlands employ a gravel bed planted with wetland vegetation. The water is kept below the surface of the gravel, and flows horizontally from the inlet to the outlet.
- Vertical Flow (VF) wetlands distribute water across the surface of a sand or gravel bed. The water percolates down through the plant root zone.

1.1.1 Free Water Surface Wetlands
These wetlands have a combination of open water areas with some floating vegetation as well as emergent plants rooted in the soil bottom. Depending upon local regulations and soil conditions, berms, dikes, and liners are used to control flow and infiltration. As the wastewater flows through the leaves and stems of plants it is treated by the processes of sedimentation, filtration, oxidation, reduction, adsorption, and precipitation (United States Environmental Protection Agency, 2000).

Figure 1.1 FWS Schematic

(Sketch courtesy S. Wallace)

Since FWS constructed wetlands closely resemble natural wetlands it should be no surprise they invariably attract a wide variety of wildlife, namely, protozoa, insects, mollusks, fish, amphibians, reptiles, birds, and mammals (Kadlec R.H. and Knight R., 1996; Knight R.L. et al., 1993). Because of the potential for pathogen exposure, FWS wetlands are rarely used for secondary treatment (United States Environmental Protection Agency, 2002). The most common applications for FWS wetlands it to polish effluent from a lagoon, trickling filter, activated sludge, or other secondary treatment process.

Figure 1.2 Typical Application of a FWS Wetland
Generally FWS are more suitable in warmer climates, because biological decomposition rates are temperature dependent, decreasing with decreasing water temperature. In addition if ice covers the open water surface the transfer of oxygen from the atmosphere is reduced (United States Environmental Protection Agency, 2000)

1.1.2 Vegetated Submerged Bed (VSB) Wetlands

Vegetated submerged Bed (VSB) constructed wetlands (also known as subsurface flow wetlands) consist of gravel and soil beds planted with wetland vegetation. They are typically designed to treat primary effluent up to secondary standards. In contrast to the FWS wetland, the wastewater stays beneath the surface of the media, flows in contact with the roots and rhizomes of the plants. Because the water is not exposed during the treatment process, the risk of pathogen exposure is minimized, and properly operated VSB’s do not provide suitable habitat for mosquitoes and other vectors.

Figure 1.3 VSB Wetland Schematic

A typical conventional VSB, as depicted above, contains inlet piping, clay or synthetic membrane liner, filter media, emergent vegetation, berms, and outlet piping with water level control. They are generally low in cost and maintenance requirements, and are most commonly used for single-family homes.
1.1.3 Vertical Flow (VF) Wetlands

Due to the low oxygen transfer rates in VSB’s they generally cannot oxidize ammonia. Vertical flow wetlands were developed in Europe as a means to produce a nitrified effluent (Cooper P.F. et al., 1997). They may be combined with VSB’s to create nitrification/denitrification treatment trains (Cooper P.F. et al., 1999). Two variations of VF wetlands exist. The type most often used in Europe (see ÖNORM B2505 for typical criteria) employ surface flooding of the bed in a single-pass configuration (1997). These systems are roughly analogous to intermittent sand filters. VF wetlands in the United States are often designed as vegetated recirculating gravel filters (Lemon E. et al., 1996).

Because vertical flow wetlands are similar to existing treatment processes, their design and performance is not discussed further in this module.
2. FREE WATER SURFACE (FWS) WETLAND PROCESSES

As discussed in Section 1, FWS wetlands are man-made analogs of natural marshes. A typical FWS wetland is shown in Figure 2.1:

Figure 2.1 FWS Wetland near Pensacola, Florida

(Photograph courtesy S. Wallace)

2.1 OVERVIEW OF TREATMENT MECHANISMS IN FWS WETLANDS

FWS wetlands can be thought of as having three major components (Breen P.F., 1990):

- A fixed component, which includes the wetland substrate, wetland vegetation, accumulated detritus and litter, and microbial biofilms (which consist of an interaction of bacteria, algae, and microfauna).

- A water component, which includes the influent, the water column in the wetland, the effluent, and their associated pollutants.

- An atmospheric component, which regulates the movement of gases into and out of the water column.

Desirable processes directly involve, or are part of a pathway that transfers or stores pollutants from the water component to the fixed component (i.e. to the wetland substrate, vegetation, and biofilms). Sedimentation, aggregation, adsorption, precipitation, volatilization, and biological uptake, and many other processes occurring in constructed wetlands are desirable. Undesirable processes generally involve the transfer of pollutants from the fixed component back to the water component. These pollutants then have the potential to be retained within the effluent discharge from the wetland. Resuspension of bed particulates, particle shearing due to high flow velocities, and excessive algae growth are examples of undesirable processes. (Sinclair Knight Mertz, 2000).
Table 2.1 Treatment Mechanisms in FWS Wetlands (Breen P.F., 1990; Kadlec R.H. and Knight R., 1996; IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000; Rogers F.E.J. et al., 1985):

**Physical**
- Sedimentation of denser particle fractions
- Filtration of lighter particle fractions by macrophytes and biofilms
- Aggregation of particles leading to removal by either sedimentation or filtration
- Exposure of influent to UV radiation via sunlight

**Chemical**
- Precipitation
- Adsorption onto wetland substratum and detritus
- Volatilization

**Biological**
- Microbial decomposition and mineralization of organic matter
- Microbial nutrient transformation (nitrification/denitrification)
- Direct biological uptake from the water column (algal and bacterial biofilms)
- Microbial competition resulting in the die-off of pathogens
- Direct animal grazing of influent organic material (including pathogens)

2.2 FLOW

Engineers typically use an idealized “plug flow” model to describe uniform movement of water through a treatment vessel. FWS wetlands do not follow this ideal flow pattern; water is mixed as it flows through the leaf litter and by wave action, also preferential flow paths are present in the vegetation. Consequently FWS wetlands display a certain degree of short-circuiting. Making the wetland longer and narrower (increasing the aspect ratio) will partially help correct this, but at a cost of additional earthwork in berms and increased head loss through the system.

Head loss in a FWS wetland can be calculated using the following modified form of Manning’s equation (Crites R. and Tchobanoglous G., 2002):

$$v = \frac{1}{n} \left( d_w \right)^{\frac{2}{3}} \left( s \right)^{\frac{1}{2}}$$

Where:
- \( v \) = liquid flow velocity, ft/s
- \( n \) = Manning’s coefficient s/ft\(^{1/3}\)
- \( d_w \) = depth of water in wetland, ft
- \( s \) = hydraulic gradient or slope of the water surface ft/ft
Manning’s coefficient \( n \) is a function of the density of the vegetation and the flow depth:

\[
n = \frac{a}{d_w^{1/2}}
\]

Where:

\( a = \text{resistance factor}, \ s \cdot ft^{1/6} \)

The resistance factor, \( a \), is dependent on the density of the vegetation and leaf litter (Crites R. and Tchobanoglous G., 2002):

<table>
<thead>
<tr>
<th>Resistance Factor, ( a )</th>
<th>Water depth, ( d_w )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.487 for sparse vegetation</td>
<td>&gt;1.3 ft</td>
</tr>
<tr>
<td>1.949 moderately dense vegetation</td>
<td>1.0 ft</td>
</tr>
<tr>
<td>7.795 very dense vegetation</td>
<td>&lt;1.0 ft</td>
</tr>
</tbody>
</table>

In addition to the wastewater input, wetlands will gain or lose water depending on the local climatic situation. Wetlands gain water through direct capture of precipitation, and lose water through a combination of evaporation and plant transpiration (evapotranspiration). Unless the wetland basin is unlined and water is lost to seepage, or there is a large tributary watershed feeding runoff to the wetland, other gains and losses are small and can generally be ignored.

Evapotranspiration can be calculated by a variety of irrigation formulas, but the most common approach is to use 80% of pan evaporation data, if available (Kadlec R.H. and Knight R., 1996):

**EXAMPLE:**

A 900 square foot FWS wetland is used to store effluent from a single-family home with an aerobic tank located in southern New Mexico. The family of four uses an average of 200 gallons of water per day. Average annual pan evaporation is 115 inches/year, and average annual precipitation is 8 inches/year. How much effluent will exit the FWS wetland?

Evapotranspiration = 80% x 115 in/yr = 92 in/yr
Net water lost to Evapotranspiration = 92 in/yr – 8 in/yr = 84 in/year

\[
\frac{84 \text{ in}}{\text{year}} \times \frac{1 \text{ year}}{365 \text{ days}} \times \frac{1 \text{ ft}}{12 \text{ in}} \times \left(900 \text{ ft}^2\right) \times \left(\frac{7.48 \text{ gallon}}{\text{ft}^3}\right) = 129 \text{ gallons/day}
\]

Water exiting wetland = 200 gal/day – 129 gallons/day = 71 gallons/day
Precipitation data is readily available for most areas in North America. While the average annual rainfall should be checked to see if the system is gaining or losing water, the magnitude of individual storm events will control the hydraulic design of downstream components of the treatment train (subsurface disposal systems, ultraviolet disinfection units, etc.). In areas with large storm events, the wetland should be designed with the capability to temporarily store rainfall and gradually release it over a period of several days.

EXAMPLE:

A 1.2-acre FWS wetland in central Iowa is used to polish effluent from an extended aeration package plant with an average annual flow of 8,000 gpd. The 2-year, 1-hour storm event is 1.4 in/hour. If no flow control is designed into the wetland outlet, what will the discharge rate be just after this storm event?

Base Effluent Flow = 8,000 gpd = 333 gal/hour = 5.6 gpm

Water Input Due to Direct Capture of Rainfall

\[
(1.2 \text{ acres}) \times \left( \frac{43560 \text{ ft}^2}{1 \text{ acre}} \right) \times \left( \frac{1.4 \text{ in}}{1 \text{ hour}} \right) \times \left( \frac{1 \text{ ft}}{12 \text{ in}} \right) \times \left( \frac{7.48 \text{ gallons}}{\text{ft}^3} \right) \times \left( \frac{1 \text{ hour}}{60 \text{ min}} \right) = 760 \text{ gallons/min}
\]

As this example illustrates, precipitation capture can have major consequences on the design of downstream processes such as UV disinfection units.

2.3 ROLE OF EMERGENT PLANTS IN FWS WETLANDS

Although emergent plants are directly mentioned in two of the mechanisms listed in Table 2.1, they influence a range of wetland treatment mechanisms (Sinclair Knight Mertz, 2000; United States Environmental Protection Agency, 2000):

- Increase sedimentation by reducing water column mixing and resuspension
- Provide surface area in the water column to increase biofilm biomass and pollutant uptake.
- Increase the removal of particles from the water column by increasing biofilm and plant surfaces available for particle interception.
- Provide shade from the plant canopy over the water column to reduce algae growth.
- Containing and preserving duckweed fronds which greatly limit reaeration and light penetration into the water column.
• Structurally cause flocculation of smaller colloidal particles into larger, settleable particles.

Since biological transformations within the wetland are largely a function of available biofilm area, the creation of surface area by emergent aquatic plants and associated leaf litter is an important contribution to the treatment process. One method to assess the relative contribution of the plants is to measure the amount of surface area available per square foot of wetland (specific surface area). For instance, a waste stabilization pond would have a specific surface area of 1 ft$^2$/ft$^2$ since the only surface area available is the pond bottom. Specific surface areas for different systems are listed in Table 2.2:

Table 2.2 Biofilm surface areas in various FWS Wetlands(Khatiwada N.R. and Polprasert C., 1999; United States Environmental Protection Agency, 1999)

<table>
<thead>
<tr>
<th>System Type</th>
<th>Vegetation Type</th>
<th>Submerged Area ft$^2$/ft$^2$</th>
</tr>
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<tbody>
<tr>
<td>Waste Stabilization Pond</td>
<td>none</td>
<td>1</td>
</tr>
<tr>
<td>Water Hyacinth Pond</td>
<td><em>Eichornia crassipes</em></td>
<td>2.18</td>
</tr>
<tr>
<td>Arcata CA</td>
<td><em>Scirpus acutus</em></td>
<td>7.6</td>
</tr>
<tr>
<td></td>
<td><em>Typha latifolia</em></td>
<td>2.6</td>
</tr>
<tr>
<td>Benton KY</td>
<td><em>Scirpus cyperinus</em></td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td><em>Typha latifolia</em></td>
<td>1.0</td>
</tr>
<tr>
<td>Houghton Lake MI</td>
<td><em>Carex spp.</em></td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td><em>Typha angustfolia</em></td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td><em>Typha latifolia</em></td>
<td>2.1</td>
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<td>Pembroke KY</td>
<td><em>Scirpus validus</em></td>
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<td></td>
<td><em>Typha angustfolia</em></td>
<td>1.5</td>
</tr>
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Selection of appropriate plants for various wetland systems is discussed in Section 5.

2.4 OXYGEN TRANSFER IN FWS WETLANDS

Organic matter breakdown (BOD reduction) may occur via either aerobic or anaerobic processes. The balance between aerobic and anaerobic conditions is determined by the pollutant mass loading to the wetland and the degree of oxygen transfer within the wetland.

The pollutant mass loading is determined by the wastewater characteristics and the method of loading the wastewater into the wetland. The oxygen demand is greatest at the point of wastewater application. With plug flow designs, wastewater is typically applied only at the inlet end of the wetland. This can lead to low dissolved oxygen levels that are favorable for mosquito growth (see Section 2.11). (Greenway M. and Chapman H., 2002). Consequently, feeding influent from multiple points (step feeding) and/or recirculating the flow within the FWS will maintain more uniform dissolved oxygen levels (Tchobanoglous G., 1987). It is important to note that in a multiple inlet design, all
inlets must be located in a manner to assure the same sequential treatment zones (vegetated zone, open zone, vegetated zone) is available downstream of each inlet point (United States Environmental Protection Agency, 2000).
Oxygen is supplied to the FWS wetland through three passive mechanisms:

- Atmospheric diffusion
- Phytoplankton (algae) photosynthesis
- Plant-mediated oxygen transfer

FWS wetlands can also be mechanically aerated to increase oxygen transfer.

Atmospheric diffusion is the transfer of oxygen into the water column from the atmosphere. This mechanism is greatly influenced by the degree of wind-induced mixing that occurs in the wetland. Wind and wave action is markedly decreased when emergent plants are present in the wetland.

Phytoplankton (algae) photosynthesis is the major oxygen transfer mechanism in FWS wetlands. Open water areas of the wetland that support the growth of suspended phytoplankton, as well as biofilms on plants and detritus, transfer oxygen directly to the water column during photosynthesis. Consequently, there are wide fluctuations in dissolved oxygen (DO) and pH levels as the wetland swings from day to night. These areas function very similarly to facultative lagoons. During daylight hours, DO levels in open water areas are elevated by phytoplankton and may exceed saturation limits (Hammer D.A., 1994; Gearheart R.A. et al., 1992).

Figure 2.2 Effect of Algae Photosynthesis on Dissolved Oxygen Levels (Mara D., 1976)
Plant-mediated oxygen transfer is the transfer of oxygen from the stem and leaf tissues of emergent wetland plants to the root structures (Brix H., 1997). In the case of FWS, this transfer mechanism is very small compared to the applied pollutant oxygen demand, and any diffusion of oxygen from the root hairs is into the wetland substrate, which is not part of the main water flow path. Consequently, plant mediated oxygen transfer is negligible in FWS wetlands.

Generally speaking, increasing the proportion of emergent wetland plants will result in less algae (suspended solids) production, but will also decrease the transfer of oxygen to the water column (Gearheart R.A. et al., 1992). FWS wetlands with 100% emergent vegetation cover will generally be anoxic or anaerobic (Sartoris J. et al., 2000), which would be desirable if denitrification is a process goal. If aerobic transformations are a design objective (i.e., nitrification) then open water areas must be included in the FWS design.(Knight R.L. and Iverson M.E., 1990).

Oxygen transfer at the surface of the water column is approximately 1.36 to 1.76 g/m$^2$ d (Wu M.-Y. et al., 2001). In FWS wetlands, additional oxygen transfer may be achieved by wind/wave action and photosynthesis in open water areas. Flow recirculation is another way to reduce local effects of influent addition and add additional dissolved oxygen. The following loading rates should not be exceeded in FWS wetlands to maintain aerobic conditions (Tchobanoglous G., 1987):

Table 2.3 FWS Wetland Configuration and Organic Loading Rates

<table>
<thead>
<tr>
<th>FWS Type</th>
<th>Typical Loading</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Semiplug-flow</td>
<td>60 kg/ha d</td>
<td>50-200 kg/ha d</td>
</tr>
<tr>
<td>Semiplug flow with 2:1 recycle and step feed</td>
<td>150 kg/ha d</td>
<td>100-200 kg/ha d</td>
</tr>
<tr>
<td>Semiplug flow with step feed, 2:1 recycle</td>
<td>200 kg/ha d</td>
<td>150-300 kg/ha d</td>
</tr>
<tr>
<td>and supplemental aeration</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**2.5 SEDIMENTATION (SUSPENDED SOLIDS)**

FWS wetland systems are generally very efficient (greater than 90%) in removing suspended solids from the influent wastewater, but the overall picture is complicated by the ability of wetlands to produce and resuspended previously settled solids(Gearheart R.A. et al., 1992).

Suspended solids are removed by settling (discrete and flocculent) as well as filtration/interception. Suspended solids are produced primarily by algae growing in open water portions of the FWS, but resuspension of particles can result from animal activity in the wetland.
Sedimentation (Discrete Particle Settling)
The largest and heaviest particles will predominantly settle out in the inlet open water zone. Slightly smaller and lighter particles may only settle out after flowing into the wetland vegetation. Wetland vegetation promotes this enhanced sedimentation by reducing water column mixing and resuspension of particles from the sediment surface.

Aggregation (Flocculation)
Aggregation is a process by which particles naturally tend to flocculate. The degree to which aggregation will occur is determined by a balance between particle attraction (controlled by surface chemistry characteristics) and the strength of the shear forces on the particles. Shear forces within the water column are a function of mixing and turbulence. Emergent and submerged plants within the water column greatly reduce shear forces, resulting in enhanced settling performance.

Interception
The smallest particles (bacteria, colloids, etc.) may not aggregate enough to settle out in the detention time available in the wetland. For these particles, the only removal mechanism available is interception by surfaces within the water column. The main surfaces in the water column are the biofilms growing on emergent wetland plants and associated leaf litter and detritus.

Predation
In quiescent regions of the FWS, predation of suspended solids (phytoplankton and zooplankton) will occur (Gearheart R.A. et al., 1992), since FWS will support populations of rotifers and other higher organisms.

Resuspension
Wetland designs that provide habitat for aquatic vertebrates, such as muskrats, nutria and carp, may experience resuspension of settled particulates due to the activities of these animals (Hey D.L. et al., 1994)

Production
Open water areas within the FWS will promote the production of suspended solids by algae. Consequently, most FWS designs employ a zone of emergent vegetation near the wetland outlet in order to minimize the production of suspended solids in the outlet zone where they could become entrained in the effluent (United States Environmental Protection Agency, 2000)

2.6 ORGANIC MATTER DEGRADATION

Wastewater contains a wide range of organic carbon compounds (and other oxygen demanding substances) that varies from being readily biodegradable to highly refractory. Organic compounds are also present in both soluble and particulate forms.
Particulate matter is removed by the mechanism discussed for suspended solids. Soluble (dissolved) compounds are removed by bacterial biofilms growing on the emergent wetland plants and associated leaf litter, as well as suspended phytoplankton growing in the water column. Readily degradable soluble compounds will be removed first, with more refractory compounds taking longer to degrade, resulting in further penetration down the length of the wetland cell.

In addition to the external organic matter load exerted by the wastewater loading, FWS wetlands have additional organic matter loads. The most significant of these is the growth, dieback, and decomposition of leaf litter associated with the wetland plants.

The nature of these two organic matter loadings is very different. Organic matter in domestic wastewater tends to contain readily degradable compounds (industrial sources will vary widely in the degradability). However, leaf litter and detritus may be quite refractory and will only be broken down slowly, exerting a low-level “background” BOD. Consequently, wetlands that receive no wastewater loading will still discharge low levels of BOD, generally in the range of 3 to 5 mg/L (Crites R. and Tchobanoglous G., 2002; Kadlec R.H. and Knight R., 1996).

Figure 2.3 Carbon Cycling within a FWS Wetland (Kadlec R.H. and Knight R., 1996)
The type of decomposition (aerobic or anaerobic) is determined by the balance between the organic matter loads (internal and external) and the oxygen transfer rate of the wetland. If the oxygen transfer rate is sufficient to satisfy the oxygen demand exerted by the organic matter loads, aerobic conditions will prevail. Aerobic decomposition tends to be rapid, with little accumulation of organic matter within the wetland. If oxygen transfer rates cannot satisfy the oxygen demands, anaerobic conditions will result. Anaerobic decomposition is slower and tends to result in the accumulation of organic matter within the detritus layer of the wetland.

### 2.7 NITROGEN CYCLING

Nitrogen can exist in many different forms (organic matter, ammonia, nitrite, nitrate, or nitrogen gas) depending on the oxidation/reduction (redox) conditions in the wetland, which is a result of the oxygen transfer rate and organic matter loadings (internal and external). In nature, nitrogen is cycled between organic and inorganic forms by the pathways shown in Figure 2.4.

**Figure 2.4. Simplified FWS Nitrogen Cycle (Kadlec R.H. and Knight R., 1996)**

Mineralization
Virtually all nitrogen present in domestic wastewater is in the form of organic nitrogen or ammonia. Mineralization (ammonification) is the process under which organic nitrogen
is converted to ammonia. Mineralization can occur under aerobic or anaerobic conditions. Eventually all organic nitrogen is broken down into ammonia, either in the septic tank or in subsequent pretreatment or soil-based treatment processes.

Under aqueous conditions, ammonia (NH₃) rapidly hydrolyzes to the ammonium ion (NH₄⁺) as follows (Snoeyink V.L. and Jenkins D., 1980):

$$\text{NH}_3 + \text{H}_2\text{O} \rightarrow \text{NH}_4^+ + \text{OH}^-$$

For practical purposes, almost all nitrogen can be considered to be in the ammonium (NH₄⁺) form before further treatment can occur.

Nitrification
Nitrification is usually defined as the biological oxidation of ammonium to nitrate with nitrite as an intermediate reaction product. The nitrifying bacteria consume oxygen, and derive energy from, the oxidation of ammonium to nitrite and the subsequent oxidation of nitrite to nitrate (IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000). The oxidation of ammonium to nitrate is a two-step process and can be written as:

$$\text{NH}_4^+ + 1.5\text{O}_2 \rightarrow \text{NO}_2^- + 2\text{H}^+ + \text{H}_2\text{O}$$

$$\text{NO}_2^- + 0.5\text{O}_2 \rightarrow \text{NO}_3^-$$

$$\text{NH}_4^+ + 2\text{O}_2 \rightarrow \text{NO}_3^- + 2\text{H}^+ + \text{H}_2\text{O}$$

The first step, the oxidation of ammonium to nitrate, is accomplished by strictly aerobic Nitrosomonas bacteria. Because these are strict aerobes, dissolved oxygen levels of at least 1.5 mg/L are recommended for nitrifying processes (Wolverton B.C., 1987). The second step, the oxidation of nitrite to nitrate, is accomplished by the bacteria Nitrobacter winogradskyi (IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000). Because acidity (H⁺ ions) are produced, there must be sufficient alkalinity present in the water to prevent the pH from dropping, as low pH values will affect the bacteria.

Denitrification
When the oxygen within the system has been depleted, bacteria are capable of utilizing the oxygen present in the nitrate (NO₃⁻) ion as an alternate electron acceptor for metabolic purposes. This reaction is irreversible and occurs in the presence of available organic carbon under anaerobic or anoxic conditions (E₀ = +350 to +100 mV). Denitrification results in the production of nitrogen or nitrogen oxide gases, which vent from the water column. The nitrogen gas pathway is illustrated below:

$$6(\text{CH}_2\text{O}) + 4\text{NO}_3^- \rightarrow 6\text{CO}_2 + 2\text{N}_2 + 6\text{H}_2\text{O}$$

(Where CH₂O represents biodegradable organic matter.)
This biodegradable organic matter can be provided by a separate chemical feed (i.e. methanol), or in some systems, the use of the influent CBOD. The presence of an organic carbon source is necessary for denitrification to occur. The carbon source present in most FWS wetland systems is from plant litter and natural detritus (Liehr R.K. and et al., 2000). The denitrification reaction takes place primarily in the wetland sediments and in the periphyton films on the submerged vegetation (United States Environmental Protection Agency, 2000).

Because this reaction takes place under reducing conditions, additional organic matter (CBOD) is usually necessary to remove dissolved oxygen and lower the redox potential ($E_h$) to allow denitrification to occur. There are approximately 17 genera of bacteria capable of denitrification (IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000). As a result of the denitrification process, gas in the form of nitrogen ($N_2$) or nitrogen oxides is formed. This nitrogen gas vents out of the water column into the atmosphere, removing the nitrogen from aqueous solution.

**FWS Configuration**

The design objectives for nitrogen transformation will influence the configuration of the FWS. If oxidation of ammonia to nitrate is a design objective, then the FWS will have to have significant open water areas to sustain the necessary oxygen transfer (Hammer D.A. and Knight R.L., 1992). In a study of a 9.9 ha FWS wetland used to polish ammonia-dominated secondary-treated municipal effluent, (Sartoris J. et al., 2000) discovered that ammonia oxidation (nitrification) declined from 76% to 4% as the wetland became fully vegetated. The excess biomass was then burned off and the wetland was re-configured to have equal areas of emergent marsh and deep open water. Ammonia oxidation (nitrification) improved back to 72%. This and other studies indicate that fully vegetated free water surface wetlands will not support nitrification unless sufficient open water areas are present, or the system is very lightly loaded (United States Environmental Protection Agency, 2000).

Figure 2.5 FWS Wetland at Ft. Deposit, Alabama with Deep Open Water Zones (Knight R.L. and Iverson M.E., 1990)
If denitrification (conversion of nitrate to nitrogen or nitrogen oxide gas) of a nitrified influent is a process objective, then the FWS wetland should have reducing conditions and available organic carbon, both of which are promoted by the presence of emergent wetland plants. For instance, two 1-ha FWS wetlands are used in Connell, Washington to denitrify 1.4 MGD of nitrified potato processing wastewater. Nitrogen removal is in excess of 85%, although additional organic carbon (primary clarifier effluent) is needed because of the very high nitrate loads in this industrial process (Burgoon P.S. et al., 1999).

Most nitrogen either passes through the FWS wetland in the form of soluble nitrate or ammonium, or is lost to the atmosphere as nitrogen gas (if the wetland supports denitrification). The nitrogen uptake capacity of emergent wetland plants, is roughly in the range of 1000 – 2,500 kg N ha\(^{-1}\) year\(^{-1}\) (Vymazal J., 2001). Most of the plant biomass, including the stored nitrogen, decomposes, re-releasing stored nitrogen back to the wetland. Nitrogen stored in the plant biomass usually represents as small amount (~7%) of the applied nitrogen load (Frankenbach R.I. and Meyer J.S., 1999). Consequently, harvesting of plant biomass is not considered a cost-effective nitrogen removal technique.

### 2.8 PHOSPHORUS CYCLING

Phosphorus is one of the most important elements in the natural ecosystem, and occurs in wastewater in soluble or particulate form. Phosphorus is often the limiting nutrient in the eutrophication of fresh water systems and can have large impacts on downstream receiving waters. Due to the phosphorus and nitrogen loadings in constructed wetlands, they are extremely nutrient enriched (eutrophied) compared to natural wetland systems. Initial phosphorus removal is through sorption onto exchange sites within the wetland...
sediments; however this storage compartment is quickly exhausted under normal phosphorus loadings (Kadlec R.H. and Knight R., 1996).

Sustainable removal in a FWS wetland is by accretion on and burial in the bottom sediments (Craft C.B. and Richardson C.J., 1993).

Figure 2.6 “Biomachine” Concept of Nutrient-Driven Sediment accretion in FWS Wetlands (Kadlec R.H., 1994)

Removal rates by sediment accretion are a function of phosphorus loading, wetland size, climate, and vegetation type. In a FWS wetland in northern Michigan, Kadlec determined that about 20% of the phosphorus stored in the biomachine was buried in sediments (Kadlec R.H., 1997), supporting a phosphorus removal rate of approximately 4 g·m⁻²·yr⁻¹. In an evaluation of 13 natural wetlands supporting submerged aquatic vegetation in Florida, Knight et al. determined the mean phosphorus removal rate to be 1.2 g·m⁻²·yr⁻¹ (Knight R.L. et al., 2003). Review of the current literature suggest a mean removal rate of approximately 1 g·m⁻²·yr⁻¹ (Vymazal J., 2001) for sediment accretion.

2.9 PATHOGEN REDUCTION

Intestinal organisms entering the FWS wetland, immediately find themselves in a very hostile environment. Thrust into lower temperatures in an environment with intense predation, most will not survive. Some may be incorporated within TSS and be removed by sedimentation, interception, and sorption (United States Environmental Protection Agency, 2000). In addition, if the organisms are at or near the water surface, UV radiation will reduce their numbers significantly. Collectively, these combined removal mechanisms can achieve reductions in fecal coliform bacteria in the 90-99% range (Crites R. and Tchobanoglous G., 2002).
However, FWS wetlands provide habitat to waterfowl and wildlife that produce fecal coliform bacteria. While effluent from a FWS wetland will have low levels of fecal coliforms (less than 1,000 CFU/100 mL), the final effluent will likely require disinfection if discharged to a surface water with a 200 CFU/100 mL effluent limit (United States Environmental Protection Agency, 2000; Kadlec R.H. and Knight R., 1996).

2.10 MOSQUITO CONTROL

FWS wetlands provide habitat for mosquitoes, and it is incumbent on the wetland designer to design systems that provide for effective mosquito control. Due to their ubiquitous nature, mosquitoes can and will find and use FWS habitats. The goal of mosquito control in FWS design is to create conditions favorable for mosquito larvae predators, such that very few of the eggs that hatch survive to become adult mosquitoes.

Predators of mosquito larvae include crustaceans (copepods, triops, swamp crayfish), coleopterans (whirligig beetles, hydrophilid and dytiscide beetle larvae), dragon fly and damsel fly nymphs, hemipterans (back swimmers), water scorpions, and the surface dwelling mesovilids (water treaders) (Greenway M. and Chapman H., 2002), although the most commonly used predator is the mosquitofish (*Gambusia holbrooki*).

Generally, mosquitoes are favored when access by predators is restricted by one of the following:

- Low dissolved oxygen levels.
- Plant bridging.
- Physical isolation

**Low Dissolved Oxygen**

Low dissolved oxygen restricts access to larval sites by Gambusia and other predator organisms. Loadings to FWS wetlands of less than 100 lb BOD/ac-d (Crites R. and Tchobanoglous G., 2002) is recommended to maintain aerobic conditions, as is step feeding (Tchobanoglous G., 1987; Stowell R. et al., 1985).

**Plant Bridging**

Heavy accumulation of plant detritus results in areas where mosquito larvae are sheltered from predators. Open water areas with water depths greater than 30 to 40 cm rarely support mosquitoes (Sinclair Knight Mertz, 2000), and these areas can be designed as deep water refugia for mosquitofish and other predators. Mosquito production is greatest in thick, heavy monotypic stands of vegetation. Varying water depth and creating open water areas reduces mosquito production. Conversion of the emergent plant zone in a FWS wetland in California to 50% open water resulted in only 25% mosquito production; converting the remaining emergent vegetation zones to a “hummock” design (12’ x 5’) to allow better predator access resulted in only 2% mosquito production (Thullen J. et al., 2002).
Figure 2.7 “Hummock” Design to Allow Predator Access to Mosquito Breeding Areas
(Thullen J. et al., 2002)

Similarly, steep side slopes or a concrete edge wall will reduce mosquito production around the edge of the wetland (Sinclair Knight Mertz, 2000).
Physical Isolation
Designers should take care that predator organisms can access inlet, outlet, and recirculation works; otherwise these can become “mosquito havens” in an otherwise well-designed FWS system.

2.11 WATER TEMPERATURE IN FWS WETLANDS

Constructed wetlands can be effective in cold climates, as evidenced by the number of systems in Canada (Pries J.H., 1994).

Biological treatment processes such as BOD oxidation, nitrification, and denitrification are temperature dependent, however physical and chemical processes (sedimentation, adsorption) are not. Consequently, during winter months, wetlands may store organic matter (suspended solids), which are then degraded as biological processes accelerate in the warmer months.

In northern climates ice formation over the water surface will decrease oxygen transfer rates. Consequently, estimation of the effluent temperature is an important step in the
design process. Energy balance methods have been developed to calculate temperatures in FWS wetlands (Kadlec R.H. and Knight R., 1996).

A simple method is to estimate the water temperature using thermal equations developed for wastewater ponds (United States Environmental Protection Agency, 1983):

\[
T_w = \frac{(0.5A)(T_a) + (Q)(T_i)}{0.5A + Q}
\]

Where:
- \( T_w \) = water temperature, °C
- \( T_a \) = ambient air temperature, °C
- \( A \) = surface area of wetland, m²
- \( Q \) = wastewater flow rate, m³/d

This equation uses the assumption of complete mixing (one CSTR), and consequently will not accurately predict water temperatures near the inlet.

Wetlands designed with effluent water temperatures less than 3 °C will experience ice formation. Additional calculations are needed to predict the water temperature under the ice and the degree of ice formation (Reed S.C. et al., 1995). Additional freeboard and variable water level controls are required to accommodate the extra ice volume in the winter.
3. VEGETATED SUBMERGED BED (VSB) WETLAND PROCESSES

Because no water is exposed during the treatment process, VSB wetland systems are particularly well suited to small-scale applications and situations where freezing is a concern. A typical VSB wetland is shown in Figure 3.1.

Figure 3.1 VSB Wetland for Single-Family Home

3.1 OVERVIEW OF VSB TREATMENT MECHANISMS

Similar to FWS wetlands, VSB wetlands have fixed, water, and atmospheric components. The primary difference is that in a VSB, the flow is forced through the gravel matrix (fixed component). This places the water in intimate contact with the gravel matrix, restricts the influence of the vegetation to the belowground root structures, and restricts gas exchange with the atmospheric component.

Table 3.1 Treatment Mechanisms in VSB Wetlands

<table>
<thead>
<tr>
<th>Category</th>
<th>Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical</td>
<td>Sedimentation of denser particle fractions</td>
</tr>
<tr>
<td></td>
<td>Filtration of lighter particle fractions by biofilms on gravel media</td>
</tr>
<tr>
<td></td>
<td>systems and plant roots</td>
</tr>
<tr>
<td></td>
<td>Aggregation of particles leading to removal by sedimentation or filtration</td>
</tr>
<tr>
<td>Chemical</td>
<td>Precipitation</td>
</tr>
<tr>
<td></td>
<td>Adsorption onto gravel media, plant roots, and biofilms</td>
</tr>
<tr>
<td>Biological</td>
<td>Microbial decomposition and mineralization of organic matter</td>
</tr>
</tbody>
</table>
Microbial nutrient transformations
Direct biological uptake from the water column (bacterial biofilms and plant roots)

3.2 FLOW

Darcy’s Law
Flow in a VSB is supposed to be below the surface. In order to ensure the flow is contained within the gravel matrix, proper application of Darcy’s Law is required (United States Environmental Protection Agency, 1988):

\[ Q = k_s A_s S \]

Where:
\( Q \) = average flow through wetland, m\(^3\)/d
\( k_s \) = hydraulic conductivity, m/d
\( A_s \) = cross-sectional area of bed, m\(^2\)
\( S \) = slope of hydraulic gradeline, m/m

The hydraulic conductivity of the bed media is related to the grain size of the material. Due to organic loading, “dirty” bed media typically has a much lower hydraulic conductivity than “clean” bed media, as indicated below:

Table 3.2 Hydraulic Conductivity of Various VSB Bed Medias (United States Environmental Protection Agency, 2000)

<table>
<thead>
<tr>
<th>Size and Type of Media</th>
<th>“Clean”/“Dirty”</th>
<th>Type of Wastewater</th>
<th>Length of Operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-10 mm gravel</td>
<td>34,000/12,000 (downstream section of bed)</td>
<td>Secondary effluent</td>
<td>2 years</td>
</tr>
<tr>
<td>5-10 mm gravel</td>
<td>34,000/900 (inlet section of bed)</td>
<td>Secondary effluent</td>
<td>2 years</td>
</tr>
<tr>
<td>17 mm creek rock</td>
<td>100,000/44,000</td>
<td>nutrient solution</td>
<td>4 months</td>
</tr>
<tr>
<td>6 mm pea gravel</td>
<td>21,000/9,000</td>
<td>nutrient solution</td>
<td>4 months</td>
</tr>
<tr>
<td>30-40 mm coarse gravel</td>
<td>not recorded/1,000 (inlet section of bed)</td>
<td>secondary effluent</td>
<td>2 years</td>
</tr>
<tr>
<td>5-14 mm fine gravel</td>
<td>not recorded/12,000 (downstream section of bed)</td>
<td>secondary effluent</td>
<td>2 years</td>
</tr>
<tr>
<td>5 mm pea gravel</td>
<td>6,200/600</td>
<td>landfill leachate</td>
<td>26 months</td>
</tr>
<tr>
<td>19 mm rock</td>
<td>120,000/3,000</td>
<td>septic tank</td>
<td>7 months</td>
</tr>
</tbody>
</table>
As Table 3.2 indicates, hydraulic conductivities are much lower at the inlet end where the organic loading is highest. For design purposes, only a small percentage of the “clean” hydraulic conductivity should be used (United States Environmental Protection Agency, 2000):

\[
\begin{align*}
\text{Initial 30\% of VSB:} & \quad \text{design } K \text{ value } = 1\% \text{ of clean bed } K \\
\text{Final 70\% of VSB:} & \quad \text{design } K \text{ value } = 10\% \text{ of clean bed } K
\end{align*}
\]

**Evapotranspiration**

In addition to the wastewater input, VSB’s will gain or lose water depending on the local climatic situation. Evapotranspiration can be calculated using a variety of irrigation formulas (Kadlec R.H. and Knight R., 1996), but the simplest method is to use about 40\% of pan evaporation data, if available:

**EXAMPLE:** A 600 square foot VSB wetland is installed at a single-family home in southern New Mexico with two residents. Average water use by the family is 100 gallons per day (2 people x 50 gallons per person per day). Average annual pan evaporation is 115 inches per year; average annual rainfall is 8 inches per year. How much water will the wetland discharge on average?

\[
\text{Evapotranspiration} = 40\% \times 115 \text{ in/yr} = 46 \text{ in/yr}
\]

\[
\text{Net water lost to Evapotranspiration} = 46 \text{ in/yr} - 8 \text{ in/yr} = 38 \text{ in/year}
\]

\[
\left( \frac{38 \text{ in}}{\text{year}} \right) \times \left( \frac{1 \text{ year}}{365 \text{ days}} \right) \times \left( \frac{1 \text{ ft}}{12 \text{ in}} \right) \times \left( 600 \text{ ft}^2 \right) \times \left( \frac{7.48 \text{ gallon}}{\text{ft}^3} \right) = \frac{39 \text{ gallons}}{\text{day}}
\]

Water exiting wetland = 100 gal/day – 39 gallons/day = 61 gallons/day

**Precipitation**

Similar to FWS wetlands, VSB’s will capture precipitation, and design of downstream processes (such as soil infiltration) must take into account the additional flow generated by precipitation events.

### 3.3 ROLE OF PLANTS IN VSB WETLANDS

Because the flow in a VSB is through the gravel matrix, the only interaction between the water being treated and the plants is within the plant root zone (rhizosphere). Within the rhizosphere, wetland plants affect the environment immediately surrounding the roots by developing symbiotic relationships with bacteria and fungi, excretion of root exudates, and oxygen transfer.
Oxygen transfer by plants was initially thought to be a dominant mechanism in VSB treatment (Kickuth R. and et al., 1987), but later research has demonstrated that the vast majority of the oxygen translocated by the plant is used for root metabolism, and the amount released to the rhizosphere is exceedingly small, about 0.02 g m\(^{-2}\) d\(^{-1}\) (Wu M.-Y. et al., 2001; Brix H. and Schierup H., 1990).

Figure 3.2 Plant-Mediated Oxygen Transfer (g m\(^{-2}\) d\(^{-1}\)) in VSB Wetlands (Brix H., 1990)

Consequently, the concept of the plants as “solar powered aerators” has been abandoned by most modern designers, and current design guidelines recommend assuming that no oxygen is delivered to the wastewater by the plant roots (United States Environmental Protection Agency, 2000).

Within the plant, there is diffusive resistance to oxygen transport (Armstrong J. and Armstrong W., 1990) and consequently there limits as to how far plants can propagate their root systems in a highly reducing environment. (Armstrong J. et al., 1990).

Figure 3.3 Internal Gas Spaces (Aerenchyma) within *Phragmites australis* Root (Armstrong J. and Armstrong W., 1990)
For VSB’s receiving anaerobic influent (i.e. wastewater from a septic tank) root growth will preferentially occur at the top of the water column, which can create preferential flow paths through the lower section of the gravel bed (United States Environmental Protection Agency, 2000). This short circuiting can be exacerbated by density gradients in the wastewater (Rash J.K. and Liehr S.K., 1999). Root penetration throughout the gravel bed is may only potentially occur in systems that receive low oxygen demands (i.e. a nitrified influent), or have some other means of supplemental oxygen transfer (Matthys A., 1999;Behrends L. et al., 1996).

### 3.4 OXYGEN TRANSFER IN VSB WETLANDS

Because the wetland vegetation essentially contributes no oxygen to the water column, this leaves atmospheric diffusion as the remaining oxygen transfer mechanism. Oxygen transfer at the surface of the water column is approximately 3.8g/m² d (38 kg/ha d) (Brix H. and Schierup H., 1990;Hiley P.D., 1994). Air movement at the water surface is impeded by the gravel matrix. Accumulations of leaf litter, insulating mulch, and root biomass (Whitney D. et al., 2003) further restrict oxygen transfer. Consequently, conventional VSB systems have a reducing environment throughout the gravel bed. These systems can remove organic matter (BOD) anaerobically, but cannot supply adequate oxygen for nitrification unless very large bed areas, in excess of 10 m² per person per day, are used (Geller G., 1996).

VSB’s are used as a denitrification process in Europe to remove nitrogen from nitrified trickling filter or vertical flow wetland effluent (Jenssen P.D. et al., 2002;Cooper P.F. et aL., 1999).

The lack of oxygen transfer has lead to the development of enhanced VSB processes that retain the advantages of conventional VSB’s (no pathogen exposure, cold climate operation, small footprint area) but provide sufficient oxygen transfer for nitrification and aerobic BOD removal. These are generally available in the United States as patented processes. Two major types of enhanced VSB processes are available. The first type use frequent water level fluctuation (tidal flow) to induce hydrodynamic movement of air into the gravel bed and oxidize exposed biofilm surfaces (Behrends L., 1999;Zoeller K.E. and Byers M.E., 1999). The second type employ direct aeration of the water column within the gravel bed (Wallace S., 2001;Kickuth R. and et al., 1987;Dufay J.A., 2000;Flowers D.A., 2002).

### 3.5 SEDIMENTATION (SUSPENDED SOLIDS)

Suspended solids are removed by the sedimentation, aggregation, and filtration/interception mechanisms discussed for FWS wetlands. However, in VSB’s wind, wave, and animal-induced mixing of the water column does not occur, so resuspension is minimal.
Because VSB’s are extremely efficient in trapping suspended solids, these solids will accumulate in the interstitial spaces within the gravel matrix. Inorganic solids will continue to accumulate, and trapped non-refractory organic solids will slowly decompose (using anaerobic processes). Accumulation of inorganic and refractory organic (non biodegradable (solids) will eventually lead to plugging of the inlet section of the bed. However, if the organic loading is at a rate at which degradable organic matter accumulates faster than the rate of decomposition, bed plugging is greatly accelerated. This has lead some designers to recommend that suspended solids loading be limited to less than 40 g m\(^{-2}\) d\(^{-1}\) of cross-sectional area (Bavor H.J. and Schulz T.J., 1993), while other designers apply a factor of safety to the hydraulic conductivity (Kadlec R.H. and Knight R., 1996), while others recommend that waste streams high in suspended solids, such as algae-laden lagoon effluent, not be treated in VSB’s (United States Environmental Protection Agency, 2000). Trapping of solids within the bed will affect the hydraulic conductivity of the media as discussed in Section 3.2.

Figure 3.4 Hydraulic Zones Within a VSB Wetland (United States Environmental Protection Agency, 2000)

Where Zone 1 = 1% of clean bed hydraulic conductivity; Zone 2 = 10% of clean bed hydraulic conductivity.

3.6 ORGANIC MATTER DEGRADATION

Particulate matter is removed by the mechanisms discussed for suspended solids above.

Soluble dissolved compounds are removed by microbial biofilms present on the gravel media and plant roots. Readily degradable compounds will be removed first, with more refractory compounds taking longer to degrade, and penetrating further down the VSB.
Due to the minimal oxygen transfer that occurs in conventional VSB’s, this is primarily an anaerobic process.

In addition to the organic matter load exerted by the wastewater, plant biomass will be retained on the surface of the gravel bed. This material will slowly decompose (faster in wet climates, slower in dry climates) and exert a secondary organic load on the system. This results in a low level “background” BOD, which is estimated to be in the range of 2 to 7 mg/L (United States Environmental Protection Agency, 1993b).

In some cold-climate applications, additional mulch material may be deliberately placed on top of the gravel layer as an insulating layer. If this material is not well decomposed, it too will exert a secondary organic loading, elevating the “background” BOD level.

Figure 3.5 Impact of Mulch Material on CBOD5 Removal for VSB Wetlands in Minnesota (Wallace S.D. et al., 2001)

<table>
<thead>
<tr>
<th>Material</th>
<th>Year 1</th>
<th>Year 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Chips</td>
<td>40 mg/L</td>
<td>20 mg/L</td>
</tr>
<tr>
<td>Material</td>
<td>Inlet Concentration (mg/L)</td>
<td>Outlet Concentration (mg/L)</td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>----------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>Poplar Bark (&quot;hog fuel&quot;)</td>
<td>60</td>
<td>20</td>
</tr>
<tr>
<td>Wood Chips buried under Sand</td>
<td>120</td>
<td>80</td>
</tr>
<tr>
<td>Reed-Sedge Peat</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>High Quality Yard Waste Compost</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>
3.7 NITROGEN CYCLING

The nitrogen present in domestic wastewater will be primarily in the protienaceous matter and urea. If a septic tank is used for pretreatment, protein and urea will be broken down to ammonia, present in the water as ammonium (NH$_4^+$). Since VSB systems are predominantly anaerobic, any remaining organic nitrogen would be changed to ammonium (NH$_4^+$) by ammonification.

Due to the limited oxygen transfer in conventional VSB systems, ammonia is typically the end product for nitrogen if septic tank effluent is the feed source (Vymazal J. et al., 1998), unless VSB’s that are very large, in excess of 10 m$^2$ per person per day, are used (Geller G., 1996). If further pretreatment (nitrification) is provided prior to the VSB, denitrification can be achieved provided there is sufficient organic carbon available (Platzer C., 1996; Cooper P.F., 2001).

Plant harvesting in only marginally successful in removing applied nutrients (<10% of applied nitrogen and <5% of applied phosphorus) and is generally not considered a cost-effective nutrient removal technique (Kuusemets V. et al., 2002).

3.8 PHOSPHORUS CYCLING

Generally speaking, VSB’s are not considered to be a phosphorus removal process. The plant component of a VSB reaches equilibrium with the applied phosphorus in the first few growing seasons. Once net plant uptake is exhausted, the remaining mechanisms are sedimentation and adsorption onto the gravel matrix.

Adsorption sites onto the gravel matrix are typically exhausted in the first few months of operation. The exception to this are manmade expanded-clay or –shale aggregates that have much higher phosphorus sorption capacities (Zhu T. et al., 1997). This material is commonly used for VSB systems in Norway (Jenssen P. et al., 1996).

The remaining mechanism, sedimentation, accounts for the majority of phosphorus removal in conventional VSB systems.

3.9 SULFUR CYCLING

Sulfur is not a regulated parameter in effluent discharges; however reduced sulfur can represent a source of maintenance headaches, a large oxygen demand, and a safety issue. Sulfate is the most common anion in surface waters. Sulfate reduction is an indicator of anaerobic conditions and sulfide oxidation is an indicator of aerobic conditions.

The sulfate reducers include desulfovibrio, desulfobulbus, and desulfbacterium. In the absence of oxygen and nitrates, these anaerobic bacteria use sulfate as the terminal
electron acceptor, and convert sulfate to sulfide and hydrogen sulfide in accordance with the following reactions:

$$\text{SO}_4^{2-} + \text{organic compounds} \rightarrow \text{S}^{2-} + \text{CO}_2 + \text{H}_2\text{O}$$

$$\text{S}^{2-} + 2 \text{H}^+ \rightarrow \text{H}_2\text{S}$$

One mg/L of sulfate (SO$_4^{2-}$) will yield approximately 0.33 mg/L of hydrogen sulfide (H$_2$S). H$_2$S is a colorless gas with the characteristic odor of rotten eggs. Accumulation of this gas in manholes, wet wells, and valve vaults can present a serious threat to treatment plant operators.

The presence of sulfate in the influent water represents an alternative oxygen supply (similar to nitrate). The presence of sulfide in the influent water represents an additional oxygen demand (in addition to BOD and ammonia). Because of the reducing conditions prevalent in VSB systems, sulfide oxidation can compete effectively with nitrification.

Some wetland systems are deliberately designed to reduce sulfate to sulfide in order to remove heavy metals (Eger P., 1992). Many metals form highly insoluble sulfide precipitates, as illustrated in Table 2-5 (Palmer et al., 1988):

<table>
<thead>
<tr>
<th>Metal</th>
<th>As hydroxide</th>
<th>As sulfide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cadmium (Cd$^{+2}$)</td>
<td>2.3 x 10$^{-5}$</td>
<td>6.7 x 10$^{-10}$</td>
</tr>
<tr>
<td>Chromium (Cr$^{+3}$)</td>
<td>8.4 x 10$^{-4}$</td>
<td>No precipitate</td>
</tr>
<tr>
<td>Cobalt (Co$^{+2}$)</td>
<td>2.2 x 10$^{-1}$</td>
<td>1.0 x 10$^{-8}$</td>
</tr>
<tr>
<td>Copper (Cu$^{+2}$)</td>
<td>2.2 x 10$^{-2}$</td>
<td>5.8 x 10$^{-18}$</td>
</tr>
<tr>
<td>Iron (Fe$^{+2}$)</td>
<td>8.9 x 10$^{-1}$</td>
<td>3.4 x 10$^{-5}$</td>
</tr>
<tr>
<td>Lead (Pb$^{+2}$)</td>
<td>2.1</td>
<td>3.8 x 10$^{-9}$</td>
</tr>
<tr>
<td>Manganese (Mn$^{+2}$)</td>
<td>1.2</td>
<td>2.1 x 10$^{-3}$</td>
</tr>
<tr>
<td>Mercury (Hg$^{+2}$)</td>
<td>3.9 x 10$^{-4}$</td>
<td>9.0 x 10$^{-20}$</td>
</tr>
<tr>
<td>Nickel (Ni$^{+2}$)</td>
<td>6.9 x 10$^{-3}$</td>
<td>6.9 x 10$^{-8}$</td>
</tr>
<tr>
<td>Silver (Ag$^{+2}$)</td>
<td>13.3</td>
<td>7.4 x 10$^{-12}$</td>
</tr>
<tr>
<td>Tin (Sn$^{+2}$)</td>
<td>1.1 x 10$^{-4}$</td>
<td>3.8 x 10$^{-8}$</td>
</tr>
<tr>
<td>Zinc (Zn$^{+2}$)</td>
<td>1.1</td>
<td>2.3 x 10$^{-7}$</td>
</tr>
</tbody>
</table>

A peat-bed subsurface flow wetland has been used since 1986 to remove copper and nickel from mine drainage at the LTV Dunka Mine near Hoyt Lakes, Minnesota (Eger P. and Lapakko, 1989; Frostman, 1993).
3.10 PATHOGEN REMOVAL

VSB’s are considered to be an effective pathogen reduction process, although very little research has been done on individual removal mechanisms.

For relatively large structures such as helminth ova, sedimentation, filtration and interception are dominant removal processes. By contrast, adsorption and natural die-off are far more important for removal of bacteria and viruses.

Typical removal rates are 98-99% for total and fecal coliforms (Gerba C.P. et al., 1999), 95-99% for viruses (Gersberg R.M. et al., 1989), and 93-99% for helminth ova (Mandi L. et al., 1998; Stott R. et al., 2002)

However, to meet a fecal coliform limit of 200 CFU/100 mL for a surface water discharge, effluent from a VSB will likely require disinfection or another pathogen reduction process (Iowa Department of Natural Resources, 2001).

3.11 WATER TEMPERATURE IN VSB WETLANDS

Energy balance methods are usually used to predict water temperatures in VSB’s (Kadlec R.H. and Knight R., 1996). VSB’s capture solar radiation in the summer, and shed excess energy through evapotranspiration. In the winter, heat loss is attenuated by the leaf litter on top of the system, which functions as a partial insulating layer. However, in cold climates (USDA hardiness zones 3 and colder), the leaf litter may not provide enough insulation to prevent the wetland from freezing if snow cover is not present (Kadlec R.H., 2001). Typically, 3 to 6 inches of insulating mulch is used, although the amount needed can be calculated (Wallace S.D. et al., 2001).
Figure 3.6 Thermal Response of Mulch-Insulated VSB Wetland in Minnesota (Wallace S.D. et al., 2001)

The temperature response of a VSB wetland can be related to air temperature (with a time lag).

Figure 3.7 Annual Course of Air and Water Temperatures for Two VSB Wetlands (Kadlec R.H., 2001)
4. WETLAND DESIGN METHODS

A variety of different wetland design methods are in use in the United States. Each method carries its own set of assumptions, and different equation sets have their own strength and weaknesses. Commonly used design methods include:

- **Natural Systems for Waste Management and Treatment**, 2nd ed. S. Reed, R. Crites, and E. Middlebrooks.


**FWS Wetland Design for BOD**

For BOD, a removal equation based on 4 continuously stirred tank reactors (CSTRs) is proposed to account for short-circuiting and dispersion:

\[
t = \frac{V}{Q} \left[ \frac{1}{(C_n/C_0)^{1/n}} - 1 \right] \times \frac{n}{k_0}
\]

Where:
- \(t\) = detention time for BOD removal, d
- \(V\) = total volume of wetland, ft\(^3\)
- \(Q\) = flow rate, ft\(^3\)/d
- \(C_n\) = effluent BOD concentration from the nth reactor in series, mg/L
- \(C_0\) = influent BOD concentration, mg/L
- \(n\) = number of complete mix reactors in series (4 is recommended)
- \(k_0\) = overall BOD removal rate constant, corrected for temperature, d\(^{-1}\) (1.01 d\(^{-1}\) is recommended at 20°C)

A safety factor of 15 to 25 percent is recommended for the detention time, \(t\).
The effluent BOD calculated by $C_n$ is the residual BOD ($BOD_{RIW}$) resulting from the influent BOD loading. The total effluent BOD ($BOD_{ECW}$) is a combination of $BOD_{RIW}$ plus the “background” BOD resulting from plant decay ($BOD_{PD}$):

$$BOD_{ECW} = BOD_{PD} + BOD_{RIW}$$

Where:

- $BOD_{ECW}$ = effluent BOD from constructed wetland, mg/L
- $BOD_{PD}$ = BOD resulting from plant decay, mg/L (5 mg/L recommended)
- $BOD_{RIW}$ = residual BOD from influent wastewater ($C_n$)

The BOD degradation rate, $k_o$, can be temperature corrected:

$$k_2 = \theta(k_1 - \theta)$$

Where:

- $k_2$ = BOD rate constant at $T_2$, °C
- $k_1$ = BOD rate constant at $T_1$, °C
- $\theta$ = temperature correction factor (1.02-1.06 recommended)

The organic loading rate should be less than 100 lb BOD/ac·d (110 kg BOD/ha·d) to maintain aerobic conditions and can be checked by:

$$L_{org} = \frac{C_o \times d_w \times \eta \times F_1}{t \times F_2}$$

Where:

- $L_{org}$ = organic loading rate, lb BOD/ac×d
- $C_o$ = BOD concentration in influent wastewater, mg/L
- $d_w$ = water depth, typically 1.25 ft
- $\eta$ = plant based void ratio, typically 0.65 to 0.75
- $F_1$ = conversion factor, 8.34 lb/[Mgal×(mg/L)]
- $t$ = detention time, days
- $F_2$ = conversion factor, 3.07 ac×ft/Mgal

The required surface area can be checked after the required detention time is calculated:

$$A = \frac{Q_{ave} \times t \times 3.07}{d_w \times \eta}$$

Where:

- $Q_{ave}$ = average daily flow through FWS wetland, Mgal/d
- $A$ = area, ac
The average flow is used to account for the influence of evapotranspiration (see Section 2.2), where:

\[ Q_{ave} = \frac{Q_{in} + Q_{out}}{2} \]

An aspect ratio (length:width) of between 2:1 and 4:1 is recommended; it is noted that long aspect ratios (greater than 10:1) will require a significant gradient to force the flow through the dense vegetation (See Section 3.2).

**FWS Design for TSS**
The method of Reed, Middlebrooks & Crites (Natural Systems for Waste Management and Treatment, 2nd Ed.) is recommended.

**FWS Design for Nitrogen**
The method of Reed, Middlebrooks & Crites (Natural Systems for Waste Management and Treatment, 2nd Ed.) is recommended for ammonia removal.

**FWS Design for Phosphorus**
The method of Kadlec & Knight (Treatment Wetlands) is recommended.

**FWS Design for Pathogens**
No design guidance.

**VSB Design for BOD**
The detention time for BOD removal is calculated by:

\[ t = \frac{-\ln C/C_o}{k_{apparent}} \]

Where:
- \( t \) = detention time for BOD removal, d
- \( C_o \) = influent BOD concentration, mg/L
- \( C \) = effluent BOD remaining from influent (\( BOD_{RIW} \)), mg/L
- \( k_{apparent} \) = overall BOD removal rate constant, corrected for temperature, d\(^{-1}\) (1.1d\(^{-1}\) recommended)

This equation assumes ideal plug flow, however the rate constant, \( k_{apparent} \) is derived from existing VSB systems and reflects the degree of dispersion and short-circuiting present in those wetlands. The rate constant, \( k_{apparent} \), can be temperature corrected using the procedure outlined for FWS wetlands. The predicted effluent BOD concentration does not include "background" BOD from plant decay, which is indicated to be 2 to 3 mg/L (\( BOD_{PD} \)).

Once the detention time is calculated, the net area of the wetland can be determined from:
\[ A_s = \frac{Q_{\text{ave}} \times t \times 3.07}{\eta \times d_w} \]

Where,

- \( A_s \) = surface area of VSB, ac
- \( Q_{\text{ave}} \) = average flow through wetland, Mgal/d
- \( t \) = detention time, d
- \( \eta \) = porosity of gravel bed media
- \( d_w \) = water depth, ft

The depth of the media may range from 18 to 30 inches, although the plant root depth will typically range between 6 and 12 inches. The water depth is typically 3 to 6 inches below the top of the media.

The aspect ratio of the wetland is a function of the wetland width, which is determined using Darcy’s Law. For flat beds, a water surface gradient, \( S \), of 0.001 is recommended.

**VSB Design for TSS Removal**

No design recommendations are given for TSS removal; however the data of Bavor & Schultz is presented along with a recommendation to keep the inlet TSS loading to less than 0.008 lb/ft\(^2\) of cross-sectional area.

**VSB Design for Nitrogen Removal.**

Area requirements for ammonia removal is presented as (Bavor H.J. et al., 1987):

\[ A = \frac{Q_{\text{ave}} \left( \ln N_o - \ln N_e \right)}{k \times d_w \times \eta \times F} \]

Where:

- \( A \) = surface area of VSB for ammonia removal, ac
- \( Q_{\text{ave}} \) = average flow through wetland, ft\(^3\)/d
- \( N_o \) = influent ammonia concentration, mg/L
- \( N_e \) = effluent ammonia concentration, mg/L
- \( k \) = ammonia removal rate constant, 0.107 d\(^{-1}\) at 20\(^\circ\)C
- \( d_w \) = depth of water in bed, ft
- \( \eta \) = effective porosity of bed media
- \( F \) = conversion factor, 43,560 ft\(^2\)/ac

The ammonia removal rate constant, \( k \), can be temperature corrected using a \( \theta \) value of 1.15.

No design guidance is given on denitrification or total nitrogen removal.
VSB Design for Phosphorus
No design guidance given.

VSB Design for Pathogens
No design guidance given.


FWS Design for BOD and TSS Removal
This EPA manual analyzed data from a few wetlands with well-defined, statistically valid data sets. TSS removal is considered critical to the FWS sizing process, as is evaluation of loadings at average and peak flows. Area loading rates are proposed to size FWS wetlands, and a 3-zone FWS model is proposed:

Figure 4.1 3-Zone FWS Wetland Model (United States Environmental Protection Agency, 2000)

Zone 1 is a densely vegetated region designed for flocculation and sedimentation of influent suspended solids and BOD. Zone 2 is an open water region designed to increase the dissolved oxygen content of the water, allowing for aerobic degradation of soluble BOD and nitrification. Zone 3 is a densely vegetated region similar to Zone 1, which is designed to reduce suspended solids reaching the outlet and provide for denitrification.
Table 4.1 Area Loading Rates for the 3-zone FWS Model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Area Loading</th>
<th>Effluent Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOD</td>
<td>45 kg/ha·d</td>
<td>&lt;20 mg/L</td>
</tr>
<tr>
<td></td>
<td>60 kg/ha·d</td>
<td>30 mg/L</td>
</tr>
<tr>
<td>TSS</td>
<td>30 kg/ha·d</td>
<td>&lt;20 mg/L</td>
</tr>
<tr>
<td></td>
<td>50 kg/ha·d</td>
<td>30 mg/L</td>
</tr>
</tbody>
</table>

It is suggested that the detention time in Zone 1 be approximately 2 to 3 days (longer detention time for colder climates), since flocculation/sedimentation of influent suspended solids is essentially complete at this point. Zone 1 will remove approximately 80% of the influent TSS. Similarly, a 2 to 3-day detention time is recommended for Zone 3 for removal of suspended solids produced in Zone 2.

Zone 2 is recommended to have a detention time less than 2-3 days to avoid algae blooms. Since Zone 2 is essentially the same as a facultative lagoon, BOD removal in this zone should follow existing temperature-dependent equations developed for lagoons (Crites R. and Tchobanoglous G., 1998).

If additional detention time is required in Zone 2, it is recommended to break the open water area into multiple zones (with intervening emergent vegetation zones) to minimize algae production. Evaluation of loadings at average and peak flows is required to determine the limiting design condition. Multiple cells in series are recommended to produce a higher quality effluent.

FWS Design for Nitrogen
It is noted that organic nitrogen associated with sediments will be removed, and nitrate produced in Zone 2 could likely be denitrified in Zone 3. Zone 2 nitrogen reduction can be designed using temperature-dependent equations developed for lagoons (Crites R. and Tchobanoglous G., 1998). Based on an analysis of operating FWS wetlands, only those with open-water sections achieved significant nitrogen reduction since this is the zone where nitrification occurs. Keeping the Total Kjeldahl Nitrogen (TKN) loading to less than 5 kg/ha·d should result in an effluent TKN concentration below 10 mg/L.

FWS Design for Phosphorus
It is noted that phosphorus associated with sediments will be removed. At loadings of less than 0.55 kg/ha·d, effluent phosphorus concentrations are generally less than 1.5 mg/L. Orthophosphate removal rates from the Arcata, CA FWS wetland is also discussed.

FWS Design for Pathogens
It is suggested that the flocculation/sedimentation that occurs in Zone 1 would result in a 1-log fecal coliform reduction, and less than a log reduction in Zone 3. Fecal reduction in Zone 2 could be estimated using equations developed for lagoons (Mara D., 1976)
\[ N_e = \frac{N_i}{1 + K_b \times t} \]

Where:
- \( N_i \) = number of fecal coliform in the influent, cfu/100mL
- \( N_e \) = number of fecal coliform in the effluent, cfu/100mL
- \( K_b \) = \( \begin{cases} \text{first order rate constant for fecal coliform removal, d}^{-1} \\ 2.6(1.19)^{(T-20)}, T = \text{water temperature, } ^\circ\text{C} \end{cases} \)
- \( t \) = retention time, days

Wildlife habitat created in Zone 2 will result in some fecal coliform being re-introduced in the system, and this was estimated to create an average “background” fecal concentration of 200 CFU/100mL (range 50-5000).

**VSB Design for BOD and TSS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Area Loading Rate</th>
<th>Effluent Concentration</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOD</td>
<td>6 g/m²•d</td>
<td>30 mg/L</td>
</tr>
<tr>
<td>TSS</td>
<td>20 g/m²•d</td>
<td>30 mg/L</td>
</tr>
</tbody>
</table>

The necessary width is calculated using Darcy’s Law. Because solids accumulation in the inlet portion of the VSB, a safety factor should be applied to the clean media hydraulic conductivity:

- Initial 30% of VSB \( K_i = 1\% \text{ of clean } K \)
- Final 70% of VSB \( K_f = 10\% \text{ of clean } K \)

Recommended applications for VSB’s are secondary treatment of septic tank effluent and polishing of secondary effluent from activated sludge and trickling filter (or RBC) plants. Using VSB’s to polish pond effluents is not recommended due to the suspended solids loading associated with the algae.

**VSB Design for Nitrogen and Phosphorus**

Due to low oxygen transfer rates, VSB’s cannot oxidize ammonia. Nitrogen removal will be negligible unless the influent is at least partially nitrified. VSB’s should be effective at denitrification. Use of VSB’s for phosphorus reduction is not recommended.

**VSB Design for Pathogens**

VSB’s should be able to achieve at least a 2-log in fecal coliform based on the operation of current systems.

This reference presents a first-order plug flow model to account for the removal of a variety of pollutants, and introduces the concept of a background concentration, $C^*$ to account for rate limiting values. The initial rate constants developed in Treatment Wetlands were from a very large database covering a variety of geographic locations and loading rates. Consequently, some of these wetlands were oxygen transfer limited, and some were not. The k-$C^*$ model developed by Kadlec & Knight was adopted by the International Water Association (IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000). The general form of this model is:

$$\ln \left( \frac{C_e - C^*}{C_i - C^*} \right) = -\frac{k_{A,T}}{q}$$

Where:
- $C_e$ = outlet target concentration, mg/L
- $C_i$ = inlet concentration, mg/L
- $C^*$ = background concentration, mg/L
- $k_{A,T}$ = temperature dependent first-order areal rate constant, m/yr
- $q$ = hydraulic loading rate, m/yr

Rearrangement and a unit conversion give the area required for a particular pollutant:

$$A = \left( \frac{0.0365 \times Q}{k_A} \right) \ln \left( \frac{C_i - C^*}{C_e - C^*} \right)$$

Where:
- $A$ = required wetland area, ha
- $Q$ = water flow rate, m$^3$/d

The rate constant, $k_A$, can be temperature corrected:

$$k_{A,T} = k_{A,20} \theta^{(T-20)}$$

Where:
- $k_{A,T}$ = first-order areal rate constant at temperature $t$, °C
- $k_{A,20}$ = first-order areal rate constant at 20°C
- $\theta$ = temperature correction factor
- $T$ = wetland water temperature, °C

Design of FWS Wetlands
There is not a lot of experience with FWS wetlands in Europe; consequently rate constants from Kadlec & Knight are presented:

Table 4.3 FWS k-C* Model Parameters (Kadlec R.H. and Knight R., 1996)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$k_{A,20}$ (m/yr)</th>
<th>$\theta$</th>
<th>$C^*$, mg/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOD</td>
<td>34</td>
<td>1.00</td>
<td>$3.5 + 0.053C_i$</td>
</tr>
<tr>
<td>TSS</td>
<td>1000</td>
<td>1.00</td>
<td>$5.1+0.16C_i$</td>
</tr>
<tr>
<td>Organic-N (sequential)</td>
<td>17</td>
<td>1.05</td>
<td>1.5</td>
</tr>
<tr>
<td>NH$_4$-N (sequential)</td>
<td>18</td>
<td>1.04</td>
<td>0.00</td>
</tr>
<tr>
<td>NO$_x$-N (sequential)</td>
<td>35</td>
<td>1.09</td>
<td>0.0</td>
</tr>
<tr>
<td>Total-N (overall)</td>
<td>22</td>
<td>1.05</td>
<td>1.50</td>
</tr>
<tr>
<td>Total P</td>
<td>12</td>
<td>1</td>
<td>0.02</td>
</tr>
<tr>
<td>Fecal Coliform</td>
<td>75</td>
<td>1.00</td>
<td>300 cfu/100 mL</td>
</tr>
</tbody>
</table>

Design of VSB Wetlands
The initial rate constants developed in Treatment Wetlands were from a very large database covering a variety of geographic locations and loading rates. Consequently, some of these wetlands were oxygen transfer limited, and some were not. Use of these rate constants tends to under-size VSB wetland systems. There is significant experience with VSB wetland in Europe, and the rate constants presented by IWA reflects the use of VSB’s to treat septic tank effluent.

Table 4.4 VSB k-C* Model Parameters (IWA Specialist Group on Use of Macrophytes in Water Pollution Control, 2000)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$k_{A,20}$ (m/yr)</th>
<th>$\theta$</th>
<th>$C^*$, mg/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOD</td>
<td>117</td>
<td>1.057</td>
<td>3.0</td>
</tr>
<tr>
<td>TSS</td>
<td>43.4</td>
<td>1.00</td>
<td>6.0</td>
</tr>
<tr>
<td>Organic-N (sequential)</td>
<td>35</td>
<td>1.05</td>
<td>1.5</td>
</tr>
<tr>
<td>NH$_4$-N (sequential)</td>
<td>34</td>
<td>1.05</td>
<td>0.00</td>
</tr>
<tr>
<td>NO$_x$-N (sequential)</td>
<td>50</td>
<td>1.05</td>
<td>0</td>
</tr>
<tr>
<td>Total-N (overall)</td>
<td>10</td>
<td>1.05</td>
<td>1.5</td>
</tr>
<tr>
<td>Total P</td>
<td>9.1</td>
<td>1.097</td>
<td>0.0</td>
</tr>
<tr>
<td>Fecal Coliform</td>
<td>100</td>
<td>1.003</td>
<td>200 cfu/100 mL</td>
</tr>
</tbody>
</table>

Note: 1. From Kadlec & Knight
2. From VSB’s treating septic tank effluent in the Czech Republic

4.4 Natural Systems for Waste Management and Treatment, 2nd ed. S. Reed, R. Crites, and E. Middlebrooks.

FWS Design for BOD Removal
FWS wetlands can be loaded up to 100 kg/ha·d.
Where:

\[ \frac{C_e}{C_0} = e^{-K_T \cdot t} \]

- \( C_e \) = effluent BOD, mg/L resulting from influent BOD
- \( C_0 \) = influent BOD, mg/L
- \( K_T \) = temperature dependent, first order rate constant, d\(^{-1}\)
- \( t \) = detention time, days

The calculated effluent BOD, \( C_e \), does not include the “background BOD, which is listed as being between 2 and 7 mg/L. Rearrangement of the above equation gives the area required for a particular pollutant:

\[
A_s = \frac{Q_{ave} \left( \ln C_0 - \ln C_e \right)}{K_T \times d_w \times \eta}
\]

Where:

- \( A_s \) = wetland surface area, m\(^2\)
- \( Q_{ave} \) = average flow rate, m\(^3\)/d
- \( d_w \) = water depth, typically 0.1-0.46 m
- \( \eta \) = wetland porosity, typically 0.65-0.75

The BOD removal rate constant, \( K_T \), can be temperature corrected:

\[
K_T = K_{20} \left( \frac{T}{20} \right)^{\theta}
\]

\[
K_{20} = 0.678 \text{ d}^{-1}
\]

It can be seen that there are many similarities between the Crites, Reed & Middlebrooks (Reed) method and the Kadlec & Knight method for BOD removal. The Kadlec & Knight approach uses an area-based rate constant instead of a volumetric one. Proponents of the area-based approach contend that the factor that primarily controls wetland performance is surface area. Also, volumetric calculations require use of the porosity, \( \eta \), which is not easily field measurable. While proponents of the volumetric approach contend that residence time controls. The two can be related by:

\[
k_a = k \times \eta \times d_w
\]

Where:

- \( k_a \) = area-based rate constant, m/yr or m/d
- \( k \) = volume-based rate constant, d\(^{-1}\) or m\(^{-1}\)
- \( \eta \) = porosity of wetland
- \( d_w \) = wetland water depth

The Kadlec & Knight temperature correction factor, \( \theta \), is 1.06 in the Reed method, and the background concentration, \( C* \) is added in after the effluent BOD is calculated, instead of being included in the equation like the Kadlec & Knight method.
FWS Design for TSS Removal
TSS removal is not considered a critical FWS sizing parameter. A removal equation is proposed based on regression of data from existing FWS systems:

\[ C_e = C_o \left[ 0.1139 + 0.00213(\text{HLR}) \right] \]

Where:
- \( C_e \) = effluent TSS, mg/L
- \( C_o \) = influent TSS, mg/L
- HLR = hydraulic loading rate, cm/d

FWS Design for Nitrogen
Design for nitrification:

\[ \frac{C_e}{C_o} = e^{-K_T \cdot t} \]

\[ A_s = \frac{Q_{\text{ave}} \ln \left( \frac{C_o}{C_e} \right)}{K_T \times d_w \times \eta} \]

Where:
- \( A_s \) = surface area of wetland, m²
- \( C_e \) = effluent ammonia concentration, mg/L
- \( C_o \) = influent TKN concentration, mg/L
- \( K_T \) = temperature dependent rate constant, d⁻¹

\[ K_T = \begin{cases} 0 \text{ d}^{-1} \text{ at } 0^\circ \text{C} \\ 0.2187 (1.048)^{(T-20)} \text{ at } 1^\circ \text{C} \end{cases} \]

\( \eta \) = wetland porosity; typically 0.65-0.75
- \( t \) = hydraulic residence time, d
- \( d_w \) = water depth in wetland, m

\[ Q_{\text{ave}} = \text{average flow through wetland, m}^3/\text{d} = \frac{Q_{\text{in}} - Q_{\text{out}}}{2} \]
Design for denitrification:
\[
\frac{C_e}{C_o} = e^{-K_T \times t}
\]
\[
A_s = \frac{Q_{ave} \ln(C_o/C_e)}{K_T \times d_w \times \eta}
\]
Where:
- \(A_s\) = surface area of wetland, m²
- \(C_e\) = effluent nitrate concentration, mg/L
- \(C_o\) = influent nitrate concentration, mg/L (influ ence + ammonia oxidized in wetland)
- \(K_T\) = temperature dependent rate constant, d⁻¹
- \(K_T = \begin{cases} 
0 \text{ d}^{-1} \text{ at } 0^\circ \text{C} \\
1.000 (1.15)^{(T-20)} \text{ at } 1^\circ \text{C} 
\end{cases}\)
- \(\eta\) = wetland porosity; typically 0.65-0.75
- \(t\) = hydraulic residence time, d
- \(d_w\) = water depth in wetland, m
- \(Q_{ave}\) = average flow through wetland, m³/d = \(\frac{Q_{in} - Q_{out}}{2}\)

FWS Design for Phosphorus
The method of Kadlec & Knight is recommended.

FWS Design for Pathogens
No design recommendations are given.

VSB Design for BOD Removal
VSB wetlands can be loaded up to 100 kg/ha·d.
\[
\frac{C_e}{C_o} = e^{-K_T \times t}
\]
Where:
- \(C_e\) = effluent BOD, mg/L resulting from influent BOD
- \(C_o\) = influent BOD, mg/L
- \(K_T\) = temperature dependent, first order rate constant, d⁻¹
- \(t\) = detention time, days

The calculated effluent BOD, \(C_e\), does not include the “background BOD, which is listed as being approximately 5 mg/L. Rearrangement of the above equation gives the area required for a particular pollutant:
\[ A_s = \frac{Q_{\text{ave}} \left( \ln C_o - \ln C_e \right)}{K_T \times d_w \times \eta} \]

Where:
- \( A_s \) = wetland surface area
- \( Q_{\text{ave}} \) = average flow rate, m³/d
- \( d_w \) = water depth, typically 0.6 m
- \( \eta \) = wetland porosity, dependent on media selected, 0.28-0.45

The BOD removal rate constant, \( K_T \), can be temperature corrected:
\[ K_T = K_{20} \left( \frac{T}{20} \right)^{1.06} \]
\[ K_{20} = 1.104 \text{ d}^{-1} \]

**VSB Design for TSS Removal**

A removal equation is proposed based on regression of data from existing VSB systems:
\[ C_e = C_o \left[ 0.1058 + 0.0011(HLR) \right] \]

Where:
- \( C_e \) = effluent TSS, mg/L
- \( C_o \) = influent TSS, mg/L
- \( HLR \) = hydraulic loading rate, cm/d

**VSB Design for Nitrogen**

Nitrification in VSB systems is dependent on available oxygen transfer, which is presumed to be proportional to the degree of plant root penetration into the gravel bed:
\[ K_{NH} = 0.01854 + 0.3922(rz)^{2.6077} \]

Where:
- \( K_{NH} \) = nitrification rate constant at 20°C, d⁻¹
- \( rz \) = fraction of VSB bed depth occupied by root zone, (0 to 1)
\[
\frac{C_e}{C_o} = e^{-K_T \times t}
\]

\[
A_s = \frac{Q_{ave} \ln \left( \frac{C_o}{C_e} \right)}{K_T \times d_w \times \eta}
\]

Where:

- \(A_s\) = surface area of wetland, m\(^2\)
- \(C_e\) = effluent nitrate concentration, mg/L
- \(C_o\) = influent nitrate concentration, mg/L (influent nitrate + ammonia oxidized in wetland)
- \(K_T\) = temperature dependent rate constant, d\(^{-1}\)
  
  \[
  K_T = \begin{cases} 
  0 \text{ d}^{-1} & \text{at } 0^\circ\text{C} \\
  K_{NH} (0.4103) \text{ at } 1^\circ\text{C} \\
  K_{NH} (1.048)^{(T-20)} \text{ at } 1^\circ\text{C} 
  \end{cases}
  \]

- \(\eta\) = wetland porosity; dependent on bed media
- \(t\) = hydraulic residence time, d
- \(d_w\) = water depth in wetland, m

\(Q_{ave}\) = average flow through wetland, m\(^3\)/d = \(\frac{Q_{in} - Q_{out}}{2}\)

Use of recirculating media filters is discussed as a nitrification process to be used in conjunction with a FWS or VSB wetland for denitrification.

**VSB Design for Denitrification**

The same equation as the one for FWS wetlands is proposed, including the same rate constant. The only difference is that the bed media porosity is used instead of emergent vegetation porosity. It is noted that VSB wetlands have more surface area for biological treatment, but the availability of organic carbon in bed media likely limits the denitrification to a rate comparable to FWS wetlands.

**VSB Design for Phosphorus**

The method of Kadlec & Knight for FWS wetlands is recommended.

**VSB Design for Pathogens**

No design recommendations are given.

This reference discusses design of VSB systems only. No performance expectations are given, but the method is intended to design VSB systems where subsurface infiltration of VSB effluent will be the disposal method.

Sizing Factors

Hydraulic Loading Criteria = 1.3 ft$^2$/gpd (unrestricted area or cold climates).

- 0.87 ft$^2$/gpd (restricted small area).

The hydraulic loading criteria determines the overall surface area of the VSB ($A_s$) and is based on prescriptive flow rates used to size onsite treatment systems (usually 120 to 150 gpd per bedroom), not actual water use.

Organic Loading Criteria = 1.0 ft$^2$/0.05 lb BOD

The organic loading criteria determines the cross-sectional area of the VSB ($A_c$). A per-capita BOD production rate of 0.17 lb/d per person is recommended, along with a 50% reduction in the septic tank, so the loading to the VSB would be 0.085 lb/d per person.

Media Depth: 12” (normal applications)
- 18” (restricted small area or cold climates)

The width of the VSB cell is determined by the organic loading criteria and the media depth. It is recommended that the width be double-checked using Darcy’s Law with a hydraulic conductivity of 850 ft/day (to account for bed clogging) for media (1/8 – ¼ inch in diameter) and a hydraulic gradient, $S$, of 0.005.

One of the recommendations made in the TVA manual is that the VSB area can be divided into two cells, with the second cell unlined for effluent disposal. Experience has shown this method cannot ensure effluent infiltration in fine-grained soils (White K.D., 1994); consequently the use of local sizing criteria for design of soil adsorption systems is recommended instead of the unlined wetland cell.
4.6 FWS DESIGN EXAMPLE

Design a FWS wetland to upgrade the discharge from a lagoon system serving a community of 800 people. Assume that the average flow is 80,000 gpd (303 m³/d), the minimum water temperature is 50 °F (10 °C), and that approximately 20% of the water is lost to evapotranspiration. The current lagoon discharge and permit limits are summarized below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Lagoon Discharge</th>
<th>Permit Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBOD5</td>
<td>50 mg/L</td>
<td>20 mg/L</td>
</tr>
<tr>
<td>TSS</td>
<td>70 mg/L</td>
<td>20 mg/L</td>
</tr>
<tr>
<td>Ammonia</td>
<td>20 mg/L</td>
<td>5 mg/L</td>
</tr>
<tr>
<td>Fecal Coliform</td>
<td>10,000 CFU/100 mL</td>
<td>200 CFU/100 mL</td>
</tr>
</tbody>
</table>

4.6.1 FWS Design Using Crites & Tchobanoglous Method

This reference provides a method for FWS wetland sizing for BOD removal. The method of Reed, Middlebrooks & Crites is recommended for TSS and nitrogen sizing. Sizing criteria for pathogen reduction is not provided.

Design for BOD Removal

The FWS wetland will produce an internal background concentration of 5 mg/L of BOD, so the wetland should be designed to produce an effluent of 20 – 5 = 15 mg/L. Assume that $k_{20} = 1.01$ d⁻¹ at 20 °C, and that a θ factor of 1.06 can be used for temperature correction of the rate constant:

$$k_2 = \theta^{(T_2 - T_1)} k_1$$

$$k_{10} = k_{20} \theta^{(10-20)} = (1.01) 1.06^{(10-20)}$$

$$k_{10} = 0.564 \text{ d}^{-1}$$

Now that the rate constant is temperature corrected, the required retention time to produce an effluent of 15 mg/L can be calculated assuming that internal mixing within the FWS can be replicated as 4 tanks in series:

$$t = \frac{V}{Q} = \left[ \frac{1}{(C_v/C_0)^{1/n} - 1} \right] \times \frac{n}{k_0}$$

Now $t = \left[ \frac{1}{(15/50)^{1/4} - 1} \right] \times \frac{4}{0.564} = 2.5$ days

The organic loading should then be checked. Assume the water depth ($d_w$) is 1.25 feet and the porosity ($\eta$) of the wetland is 0.7:

$$L_{org} = \frac{C_0 \times d_w \times \eta \times F_1}{t \times F_2} = \frac{50 \times 1.25 \times 0.7 \times 8.34}{2.5 \times 3.07} = 45.4 \text{ lb BOD/ac d}$$

45.4 lb BOD/ac d ≤ 100 lb BOD/ac d OK
The area of the wetland can be calculated based on the detention time and the average flow ($Q_{ave}$):

$$A = \frac{Q_{ave} \times t \times 3.07}{d_w \times \eta}$$

$$Q_{ave} = \frac{80,000 + (1-0.2)(80,000)}{2} = 72,000 \text{ gpd}$$

$$A = \frac{\left(\frac{72,000}{1,000,000}\right) \times 2.5 \times 3.07}{1.25 \times 0.7} = 0.63 \text{ ac (0.255 ha)}$$

Since this method recommends a safety factor of 15 to 25 percent, so the net wetland area should be rounded up slightly (0.75 ac).

**4.6.2 FWS Design Using USEPA Method**

The USEPA method required evaluation of loadings at peak and average flows to determine the design-limiting condition. (To provide a comparative basis among design methods, this example only addresses average loadings).

To achieve a 20 mg/L limit for BOD and TSS, loadings must be less than:

- **BOD Loading:** 45 kg/ha·d
- **TSS Loading:** 30 kg/ha·d

Also, the 3-zone FWS model must be used to achieve the required effluent concentrations (see Figure 4.1).

**Design for BOD Removal**

The first step is to calculate the BOD mass loading:

$$\frac{50\text{mg}}{\text{L}} \times \frac{1000\text{L}}{\text{m}^3} \times \frac{303\text{m}^3}{\text{d}} \times \frac{1\text{g}}{1000\text{mg}} \times \frac{1\text{kg}}{1000\text{g}} = 15\text{ kg/d}$$

The next step is to determine the required wetland area:

$$\frac{1\text{ ha d}}{45\text{ kg d}} \times \frac{15\text{ kg}}{d} = 0.33\text{ ha (0.83ac)}$$

Based on the average flow rate, a mean water depth of 0.8m and a mean porosity of 0.8 across the entire wetland (emergent vegetation zones 1 and 3 plus open water zone 3), the hydraulic retention time can be calculated:
Assuming that the hydraulic retention time in each zone is equal results in $7.75/3 = 2.6$ days per zone. Since this is less than 3 days for the open water area (Zone 2), algae blooms should not be a concern.

Calculate the area occupied by the open water (Zone 2) assuming that this zone has a depth of 1.2m and a porosity of 1.0:

$$\frac{1 \text{ha}}{10,000 \text{m}^2} \times \frac{272.5 \text{m}^3}{1 \text{d}} \times \frac{1}{1.0} \times \frac{1}{1.2 \text{m}} \times 2.6 \text{d} = 0.06 \text{ha}$$

The areas of Zones 1 and 3 would then be:

$$0.33 - 0.06 \div 2 = 0.135 \text{ha}$$

The result is shown schematically below:

![Diagram](image)

**Design for TSS Removal**

The first step is to calculate the TSS mass loading:

$$\frac{70 \text{mg}}{\text{L}} \times \frac{1000 \text{L}}{1 \text{m}^3} \times \frac{303 \text{m}^3}{\text{d}} \times \frac{1 \text{g}}{1000 \text{mg}} \times \frac{1 \text{kg}}{1000 \text{g}} = 21.2 \text{kg/d}$$

The next step is to determine the required wetland area:

$$\frac{1 \text{ha d}}{30 \text{kg}} \times \frac{21.2 \text{kg}}{\text{d}} = 0.70 \text{ha (1.75 ac)}$$

Based on the average flow rate, a mean water depth of 0.8m and a mean porosity of 0.8 across the entire wetland (emergent vegetation zones 1 and 3 plus open water zone 3), the hydraulic retention time can be calculated:
Assuming that the hydraulic retention time in each zone is equal results in \( \frac{16.4}{3} = 5.5 \) days per zone. Since exceeds 3 days for the open water area (Zone 2), algae blooms are likely to be a problem. In this case, the open water area should be broken into two compartments with an intervening emergent vegetation zone. This would create a 5-zone wetland with an average retention time of \( \frac{16.4}{5} = 3.3 \) days per zone, close to the recommended 2 to 3 days.

Calculate the area occupied by the open water (Zones 2 and 4) assuming that these zones have a depth of 1.2m and a porosity of 1.0:

\[
\frac{0.70 \text{ha} \times \frac{10,000 \text{m}^2}{\text{ha}} \times 0.8 \text{m} \times 0.8 \times \frac{1 \text{d}}{272.5 \text{m}^3}}{1 \text{d}} = 16.4 \text{d}
\]

The result is shown schematically below:
Based on the TSS design, for Zone 1, a reduction of about 1-log in fecal coliform would occur, reducing the fecals from 10,000 down to 1,000 CFU/100mL. Reduction in the open water zones (the combined residence time in the two zones would be 6.6 days) can be estimated by:

\[
N_e = \frac{N_i}{1 + K_h \times t}
\]

\[
K_h = 2.6(1.19)^{(T-20)} = 2.6(1.19)^{(10-20)} = 0.46d^{-1}
\]

\[
N_e = \frac{1000}{1 + 0.46 \times 6.6} = 249\text{CFU}/100\text{mL}
\]

Some further reduction of fecal coliform would occur in the final vegetated zone. However, the introduction of fecal coliform will create a “background” concentration of approximately 200 CFU/100mL (range 50-5,000), so disinfection would still be necessary to comply with the permit limit.

**4.6.3 FWS Design Using Method of Kadlec & Knight**

The area of the wetland can be calculated by:

\[
A = \left( \frac{0.0365 \times Q}{k_A} \right) \times \ln \left( \frac{C_i - C^*}{C_e - C^*} \right)
\]

Rate constants can be temperature corrected by:

\[
k_{A,T} = k_{A,20} \theta^{(T-20)}
\]

**Design for BOD Removal**

The rate constant for BOD removal is 34 m/yr and \( \theta \) is 1.0, so the rate constant does not change for a water temperature of 10 °C. The background concentration is calculated as 3.5 + 0.053(Ci), or:

\[
C^* = 3.5 + 0.053(50) = 6\text{mg/L}
\]

The required area can be calculated (using the influent flow of 303 m³/d) as:

\[
A = \left( \frac{0.0365 \times 303}{34} \right) \times \ln \left( \frac{50 - 6}{20 - 6} \right) = 0.37\text{ha} (0.92\text{ac})
\]

**Design for TSS Removal**

The rate constant for TSS removal is 1000 m/yr and \( \theta \) is 1.0, so the rate constant does not change for a water temperature of 10 °C. The background concentration is calculated as 5.1 + 0.16(Ci), or:
The required area can be calculated as:

\[
A = \left( \frac{0.0365 \times 303}{1000} \right) \times \ln \left( \frac{70 - 16}{20 - 16} \right) = 0.03 \text{ha (0.07 ac)}
\]

**Design for Ammonia Removal**

The rate constant for ammonia removal is 18 m/yr and θ is 1.04, so the rate constant must be corrected for a water temperature of 10 °C. The background concentration for ammonia is 0 mg/L:

\[
k_{\text{A,T}} = k_{\text{A,20}} \theta^{(T-20)}
\]

\[
k_{\text{A,10}} = 18(1.04)^{(10-20)} = 12 \text{m/yr}
\]

The required area can be calculated as:

\[
A = \left( \frac{0.0365 \times 303}{12} \right) \times \ln \left( \frac{20 - 0}{5 - 0} \right) = 1.3 \text{ha (3.2 ac)}
\]

**Design for Pathogen Removal**

The suggested background concentration C* for fecal coliform is 300 CFU/100mL, so disinfection will be needed to meet the permit limit regardless of the wetland size.

### 4.6.4 FWS Design Using Method of Reed, Middlebrooks & Crites

**Design for BOD Removal**

The rate constant should be temperature corrected for the water temperature of 10 °C:

\[
K_T = K_{\text{20}} \left( \frac{T}{20} \right)^{\frac{1}{4}}
\]

\[
K_{10} = 0.678 \left( \frac{10}{20} \right)^{\frac{1}{4}} = 0.379 \text{d}^{-1}
\]

The wetland will produce a background concentration of BOD that is not included in the sizing calculation. Assuming this background concentration is 5 mg/L, the wetland must be designed for an effluent concentration of 20-5 = 15 mg/L. Assume the average water depth, d_w in the wetland is 1.25 ft (0.38 m) and the porosity, η is 0.70. The average flow rate (272.5 m^3/d) is used:
\[ A_s = \frac{Q_{ave} (\ln C_o - \ln C_e)}{K_r \times d_w \times \eta} \]

\[ A_s = \frac{272.5 (\ln 50 - \ln 15)}{0.379 \times 0.38 \times 0.70} = 3182 \text{m}^2 \Rightarrow 0.32 \text{ha (0.78ac)} \]
Design for TSS Removal

An empirical design equation is used to relate the effluent TSS to the hydraulic loading rate:

\[
C_e = C_o \left[ 0.1139 + 0.00213 (HLR) \right]^{20/70} \left[ 0.1139 + 0.00213 (HLR) \right]
\]

Required HLR = 80 cm/d

\[
\text{HLR} = \frac{\text{Flow}}{\text{Area}}
\]

\[
\text{Area} = \frac{303 \text{m}^3}{d} \times \frac{d}{80 \text{cm}} \times 100 \text{cm} = 375 \text{m}^2 \Rightarrow 0.038 \text{ha} (0.093 \text{ac})
\]

Design for Ammonia Removal

The rate constant should be temperature corrected for the water temperature of 10 °C:

\[
K_r = K_{20} (1.048)^{(T_{-20})}
\]

\[
K_{10} = 0.2187 (1.048)^{(10-20)} = 0.1368 \text{ d}^{-1}
\]

Assume the average water depth, \(d_{w}\) in the wetland is 1.25 ft (0.38 m) and the porosity, \(\eta\) is 0.70. The average flow rate (272.5 m³/d) is used:

\[
A_e = \frac{Q_{ave} (\ln C_o - \ln C_e)}{K_r \times d_{w} \times \eta}
\]

\[
A_e = 272.5 \frac{\left( \ln 20 - \ln 5 \right)}{0.1368 \times 0.38 \times 0.70} = 10,381 \text{m}^2 \Rightarrow 1.04 \text{ ha} (2.56 \text{ ac})
\]

4.6.5 FWS Design Summary

The examples above illustrate that each sizing method has its strength and weaknesses and there is not a consensus on how to size FWS wetlands for specific treatment objectives. It is yet to be proved that the smallest or the largest designs are unrealistic. If a design method produces a sizing factor that is clearly an anomaly, it should be checked against the actual performance of existing wetland systems in that area (if possible), or at a minimum, against other sizing criteria.
Table 4.5 FWS Sizing Results

<table>
<thead>
<tr>
<th>Method</th>
<th>BOD</th>
<th>TSS</th>
<th>Ammonia</th>
<th>Fecal Coliform</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crites &amp; Tchobanoglous, 1998</td>
<td>0.75 ac</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>USEPA, 2000</td>
<td>0.83 ac</td>
<td>1.75 ac</td>
<td>Not recommended</td>
<td>Disinfection Required</td>
</tr>
<tr>
<td>Kadlec &amp; Knight, 1996</td>
<td>0.92 ac</td>
<td>0.07 ac</td>
<td>3.2 ac</td>
<td>Disinfection Required</td>
</tr>
<tr>
<td>Reed, Middlebrooks &amp; Crites, 1995</td>
<td>0.78 ac</td>
<td>0.09 ac</td>
<td>2.56 ac</td>
<td>---</td>
</tr>
</tbody>
</table>

Obviously, the summary presented here is not a complete or exhaustive analysis of each design method. For actual design applications, the designer should refer to the original source material.
4.7 VSB DESIGN EXAMPLE

Design a VSB wetland to treat septic tank effluent from a 50 home residential development with 3 bedroom homes. The development will contain an average of 110 residents with an actual flow of 6,600 gpd (883 ft³/d or 25 m³/d). Assume that the VSB is located in a wet climate and evapotranspiration effects are minimal.

The prescriptive criteria used by the local county for onsite systems is 120 gallons per bedroom. The minimum water temperature is 50 °F (10 °C) and wastewater must be treated to secondary standards in the VSB for discharge to a performance-based soil adsorption system. The state is considering a 10 mg/L Total Nitrogen limit (all of which would be in the form of ammonia). Assume the wetland will have a water depth (d_w) of 1.25 feet (0.38 meters) and a porosity (η) of 0.36. Treatment objectives are outlined below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Septic Tank Effluent</th>
<th>Permit Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBOD5</td>
<td>150 mg/L</td>
<td>30 mg/L</td>
</tr>
<tr>
<td>TSS</td>
<td>85 mg/L</td>
<td>30 mg/L</td>
</tr>
<tr>
<td>Ammonia</td>
<td>40 mg/L</td>
<td>10 mg/L</td>
</tr>
<tr>
<td>Fecal Coliform</td>
<td>100,000 CFU/100 mL</td>
<td>10,000 CFU/100mL</td>
</tr>
</tbody>
</table>

4.7.1 VSB Design Using Method of Crites & Tchobanoglous

Design for BOD Removal
The rate constant, 1.1 d⁻¹ should be temperature corrected to the 10 °C water temperature. Assume the temperature correction factor, θ, is 1.06 This is essentially the same method presented in Natural Systems for Waste Management and Treatment (Reed S.C. et al., 1995):

\[ k_{\text{apparent,10}} = 1.1(1.06)^{(10-20)} = 0.61 \text{ d}^{-1} \]

The VSB will produce a background concentration of BOD of about 3 mg/L. Consequently, the design effluent value will be 30-3 = 27 mg/L. The detention time for BOD removal is calculated by:

\[ t = \frac{\ln C/C_o}{k_{\text{apparent}}} = \frac{-\ln 27/150}{0.61} = 2.8 \text{ days} \]

The net area of the wetland can be determined from:

\[ A_v = \frac{Q_{\text{avg}} \times t \times 3.07}{\eta \times d_w} \]

\[ A_v = \frac{6,600}{1,000,000} \times 2.8 \times 3.07 \times 0.36 \times 1.25 = 0.12 \text{ ac} \Rightarrow 5,492 \text{ ft}^2 \]
The required width of the wetland can be calculated using Darcy’s Law:

\[ Q = k_s A_c S \]

Where:
- \( Q \) = average flow through wetland, \( m^3/d \)
- \( k_s \) = hydraulic conductivity, \( m/d \)
- \( A_c \) = cross-sectional area of bed, \( m^2 \)
- \( S \) = slope of hydraulic gradeline, \( m/m \)

For flat beds, a water surface gradient, \( S \), of 0.001 is recommended. Assume the wetland will have a water depth (\( d_w \)) of 1.25 feet (0.38 meters) and the bed material has a \( D_{10} \) of 8 mm and a porosity (\( \eta \)) of 0.36 resulting in clean bed hydraulic conductivity of 5,000 m/d (16,400 ft/d). No reductions in hydraulic conductivity are required, consequently:

\[ A_c = \frac{Q}{k_s \times S} = \frac{882 ft^3/d}{(16,400 ft/d) \times 0.001} = 53.8 ft^2 \]

\[ d_w = 1.25 \text{ ft} \]

\[ \text{Width} = \frac{53.8 \text{ ft}^2}{1.25 \text{ ft}} = 43 \text{ ft} \]

\[ \text{Length} = \frac{5,492 \text{ ft}^2}{43 \text{ ft}} = 127.6 \text{ ft} \]

\[ \text{Length:Width Ratio} = \frac{127.6}{43} = 3:1 \]

**Design for TSS Removal**

No sizing criteria are given for TSS removal; however the inlet TSS loading should be less than 0.008 lb/ft\(^2\) of cross-sectional area. For the BOD wetland sizing calculated above, this would result in:

\[ \text{TSS Loading} = \left( \frac{6,600}{1,000,000} \right) \times 85 \text{ mg/L} \times 8.34 \text{ lb/gal} = 4.68 \text{ lb/d} \]

\[ A_c = \frac{1\text{ ft}^2\text{d}}{0.008\text{lb}} \times \frac{4.68\text{lb}}{\text{d}} = 585 \text{ ft}^2 \]

\[ \text{Width} = \frac{585\text{ft}^2}{1.25\text{ft}} = 468 \text{ ft} \]

\[ \text{Length} = \frac{5,492\text{ft}^2}{468\text{ft}} = 11.8 \text{ ft} \]

\[ \text{L:W} = 0.025:1 \]

**Design for Ammonia Removal.**
The rate constant, 0.107 d\(^{-1}\) should be temperature corrected to the 10 °C water temperature using a temperature correction factor \(\theta\) of 1.15:
\[
k_{10} = 0.107(1.15)^{10-20} = 0.0264 \text{ d}^{-1}
\]

The area can be calculated as:
\[
A = \frac{Q_{\text{ave}}(\ln N_a - \ln N_c)}{k \times d_w \times \eta \times F}
\]
\[
A = \frac{882(\ln 40 - \ln 10)}{0.0264 \times 1.25 \times 0.36 \times 43560} = 2.36 \text{ ac} \Rightarrow 102,968 \text{ ft}^2
\]

### 4.7.2 VSB Design Using USEPA Method

#### Design for BOD Removal

The loading to the VSB should be less than 6 g/m\(^2\)·d:
\[
\text{BOD Loading} = \frac{150 \text{mg}}{\text{L}} \times \frac{25 \text{m}^3}{\text{d}} \times \frac{1000 \text{L}}{1 \text{m}^3} \times \frac{1 \text{g}}{1000 \text{mg}} = 3,750 \text{ g/d}
\]
\[
\text{Area} = \frac{1 \text{m}^2}{6 \text{g}} \times \frac{3,750 \text{g}}{\text{d}} = 625 \text{ m}^2 \Rightarrow 6,727 \text{ ft}^2
\]

This area should be broken into the primary treatment zone (first 30% of the VSB) and the secondary treatment zone (last 70% of the VSB):

Primary Treatment Area, \(A_{p} = 30\% \times 625 \text{ m}^2 = 187.5 \text{ m}^2\)

Secondary Treatment Area, \(A_{s} = 70\% \times 625 \text{ m}^2 = 435.5 \text{ m}^2\)

The width of the bed should be determined using Darcy’s Law and the “dirty” hydraulic conductivity of the primary treatment area. With a recommended media of 20 – 30 mm gravel, the “clean” hydraulic conductivity is 100,000 m/d. The “dirty” hydraulic conductivity would then be:
\[
K_i = 1\% \times 100,000 \text{ m/d} = 1,000 \text{ m/d}
\]

It is recommended that the head loss in the primary treatment zone be limited to 10% of the bed depth. Since the bed depth in this example is 0.38 m, the allowable head loss would be 10% \(\times 0.38 \text{ m} = 0.038 \text{ m}\). For Darcy’s Law, the gradient is the head loss (change in elevation) divided by the flow length. The flow length in the primary treatment zone is the primary treatment area (\(A_{p}\)) divided by the width of the VSB (W). Substituting these terms into Darcy’s Law and rearranging the terms yields the following equation:
\[ W^2 = \frac{Q \times A_{si}}{K_i \times d_w \times h} \]

Where:

- \( W \) = width of VSB bed, m
- \( Q \) = flowrate, m\(^3\)/d
- \( A_{si} \) = Primary Treatment Area, m\(^2\)
- \( K_i \) = "dirty" hydraulic conductivity in Primary Treatment Zone, m/d
- \( d_w \) = water depth in VSB bed, m
- \( h \) = elevation change of water surface across Primary Treatment Zone, m
  
  (10\% of \( d_w \) recommended)

For this example, the width of the bed can be calculated as:

\[ W^2 = \frac{Q \times A_{si}}{K_i \times d_w \times h} = \frac{\left( 25 \text{ m}^3/\text{d} \right) \left( 187.5 \text{ m}^2 \right)}{(1000 \text{ m/d}) \left( 0.38 \text{ m} \right) \left( 0.038 \text{ m} \right)} \]

\[ W = 18 \text{ m} \]

The calculated width of 18 m (59 feet) is less than the maximum recommended width of 61 m (200 feet). The length of the primary treatment zone, \( A_{si} \), can be calculated to be 10.4 m, and the length of the secondary treatment zone, \( A_{sf} \), can be calculated to be 24.3 m. The total VSB length is then 34.7 m, and the resulting length/width ratio is 1:1.9, which falls within the recommended range of 1:1 to 1:2.

**Design for TSS Removal**

The loading to the VSB should be less than 20 g/m\(^2\)-d:

\[ \text{TSS Loading} = \frac{85 \text{ mg/L} \times 25 \text{ m}^3/\text{d} \times 1000 \text{ L/m}^3 \times 1 \text{ g/1000 mg}}{1000 \text{ mg}} = 2,125 \text{ g/d} \]

\[ \text{Area} = \frac{1 \text{ m}^2/\text{d} \times 2,125 \text{ g/d}}{20 \text{ g/d}} = 106 \text{ m}^2 \Rightarrow 1,144 \text{ ft}^2 \]

**Design for Pathogen Reduction**

No sizing criteria are provided but it is noted that a VSB system should be able to achieve at least a 2-log reduction in fecal coliforms, indicating that compliance with the permit of 10,000 CFU/100mL is likely.
4.7.3 VSB Design Using the Method of Kadlec & Knight and IWA

The area of the wetland can be calculated by:

\[ A = \left( \frac{0.0365 \times Q}{k_A} \right) \times \ln \left( \frac{C_i - C''}{C_e - C''} \right) \]

Rate constants can be temperature corrected by:

\[ k_{A,T} = k_{A,20} \theta^{(T-20)} \]

Design for BOD Removal

The rate constant for BOD removal is 117 m/yr and \( \theta \) is 1.057, so the rate constant needs to be corrected for a water temperature of 10 °C. The background concentration is 3.0 mg/L:

\[ k_{A,T} = k_{A,20} \theta^{(T-20)} \]

\[ k_{A,10} = 117 \times (1.057)^{(10-20)} = 67.2 \text{ m/yr} \]

The required area can be calculated (using the influent flow of 25 m³/d) as:

\[ A = \left( \frac{0.0365 \times 25}{67.2} \right) \times \ln \left( \frac{150 - 3}{30 - 3} \right) = 0.023 \text{ ha (0.057 ac, or 2,477 ft}^2) \]

Assume that the bed is flat and the water surface gradient (S) available is 0.001. In the initial portion of the bed, only 10% of the clean bed hydraulic conductivity should be assumed to be available after clogging. Assume the use of 10 cm gravel, which would have a clean bed hydraulic conductivity of approximately 10,000 m/d (32,800 ft/d) (see Figure 9-18, p. 214)

\[ A_e = \frac{Q}{k_e \times S} = \frac{882 \text{ ft}^3/\text{d}}{32,800 \text{ ft}^3/\text{d} \times 0.1 \times 0.001} = 269 \text{ ft}^2 \]

\[ d_w = 1.25 \text{ ft} \]

Width = \( \frac{269 \text{ ft}^2}{1.25 \text{ ft}} = 215 \text{ ft} \)

Length = \( \frac{2,477 \text{ ft}^2}{215 \text{ ft}} = 11.5 \text{ ft} \)

Length:Width Ratio = \( \frac{11.5}{215} = 0.05 : 1 \)

Design for TSS Removal

The rate constant for TSS removal is 43.4 m/yr and \( \theta \) is 1.00, so the rate constant does not change for a water temperature of 10 °C. The background concentration is 6 mg/L. The required area can be calculated as:
Design for Ammonia Removal
The rate constant for nitrogen removal is 34 m/yr and θ is 1.05, so the rate constant must be corrected for a water temperature of 10 °C. The background concentration is 0:

\[ k_{A,T} = k_{A,20} \theta^{(T-20)} \]

\[ k_{A,10} = 34(1.05)^{(10-20)} = 20.9 \text{ m/yr} \]

The required area can be calculated as:

\[ A = \left( \frac{0.0365 \times 25}{20.9} \right) \times \ln \left( \frac{40 - 0}{10 - 0} \right) = 0.06 \text{ ha (0.15 ac, or 6,515 ft}^2 \) \]

Design for Pathogen Removal
The rate constant for fecal coliform reduction is 100 m/yr and θ is 1.003, so the rate constant must be corrected for a water temperature of 10 °C. The background concentration is 200 CFU/100 mL:

\[ k_{A,T} = k_{A,20} \theta^{(T-20)} \]

\[ k_{A,10} = 100(1.003)^{(10-20)} = 97 \text{ m/yr} \]

The required area can be calculated as:

\[ A = \left( \frac{0.0365 \times 25}{97} \right) \times \ln \left( \frac{100,000 - 200}{10,000 - 200} \right) = 0.02 \text{ ha (0.05 ac, or 2,350 ft}^2 \) \]

4.7.4 VSB Design Using the Method of Reed, Middlebrooks & Crites

Design for BOD Removal
The rate constant, 1.104 d\(^{-1}\) should be temperature corrected to the 10 °C water temperature. Assume the temperature correction factor, θ, is 1.06:

\[ k_{\text{apparent},10} = 1.104(1.06)^{(10-20)} = 0.61 \text{ d}^{-1} \]

The VSB will produce a background concentration of BOD of about 5 mg/L. Consequently, the design effluent value will be 30-5 = 25 mg/L. The detention time for BOD removal is calculated by:

\[ t = \frac{-\ln C/C_o}{k_{\text{apparent}}} = \frac{-\ln 25/150}{0.61} = 2.94 \text{ days} \]

The net area of the wetland can be determined from:
Design for TSS Removal
An empirical design equation is used to relate the effluent TSS to the hydraulic loading rate:

\[
C_e = C_o \left[ 0.1058 + 0.0011(HLR) \right] \\
30 = 85 \left[ 0.1058 + 0.0011(HLR) \right]
\]

Required HLR = 225 cm/d

\[
\text{HLR} = \frac{\text{Flow}}{\text{Area}}
\]

Area = \( \frac{25 \text{m}^3}{\text{d}} \times \frac{\text{d}}{225 \text{cm}} \times \frac{100 \text{cm}}{\text{m}} = 11.1 \text{ m}^2 \Rightarrow 120 \text{ ft}^2 \)

Design for Nitrogen
Nitrification in VSB systems is dependent on available oxygen transfer, which is presumed to be proportional to the degree of plant root penetration into the gravel bed:

\[
K_{NH} = 0.01854 + 0.3922(zr)^2
\]

Where:

- \( K_{NH} \) = nitrification rate constant at 20 °C, d\(^{-1}\)
- \( rz \) = fraction of VSB bed depth occupied by root zone, (0 to 1)

Maximum root penetration for plants with minimal internal diffusive resistance (i.e. \( Phragmites australis \)) is reported to be 0.6m (1990). Assuming a bed depth of 1.25 ft, (0.38m) implies that \( rz \) would be 1.0m, resulting in \( K_{NH} = 0.4107 \text{ d}^{-1} \). This rate constant can be temperature corrected to 10 °C:

\[
K_T = K_{NH} \left(1.048\right)^{\left(10-20\right)}
\]

\[
K_{T0} = 0.4107 \left(1.048\right)^{\left(10-20\right)} = 0.257 \text{ d}^{-1}
\]

The required area can be determined by:

\[
A_s = \frac{Q_{ave} \ln \left( C_o/C_e \right)}{K_T \times d_w \times \eta}
\]

\[
A_s = \frac{25 \ln \left(40/10\right)}{0.257 \times 0.38 \times 0.36} = 986 \text{ m}^2 = 10,610 \text{ ft}^2
\]
4.7.5 VSB Design Using TVA Method

This method is based on prescriptive design flows. Assuming no space limitations, the recommended sizing criteria is 1.3 ft\(^2\)/gpd:

\[
\text{Prescriptive Design Flow} = 50 \text{ homes} \times 3 \text{ bedrooms} \times 120 \text{ gallon/bedroom} = 18,000 \text{ gpd}
\]

\[
\text{Area Required} = \frac{1.3 \text{ ft}^2}{\text{gpd}} \times 18,000 \text{gpd} = 23,400 \text{ ft}^2
\]

(This calculation illustrates the impact of using the prescriptive design flow of 18,000 gpd (2,406 ft\(^3\)/d). Many states allow alternative calculations of prescriptive design flows for cluster systems, which would impact the footprint area of the wetland).

The cross-sectional area (\(A_c\)) is determined based on an organic loading rate of 1.0 ft\(^2\)/0.05 lb BOD.

\[
\text{BOD Loading} = \frac{150 \text{mg}}{L} \times \frac{25 \text{m}^3}{\text{d}} \times \frac{1000 \text{L}}{1 \text{m}^3} \times \frac{1 \text{g}}{1000 \text{mg}} = 3,750 \text{ g/d} (8.3 \text{ lb/d})
\]

\[
A_c = \frac{1 \text{ft}^2 \text{d}}{0.05 \text{lb BOD}} \times \frac{8.3 \text{ lb BOD}}{\text{d}} = 165 \text{ ft}^2
\]

The recommended media for the bed is 1/8 to ¼-inch gravel (3-6mm) and a hydraulic conductivity of 850 ft/d is recommended to account for clogging in the inlet zone with a hydraulic gradient (S) of 0.005. Darcy’s Law indicates that:

\[
A_c = \frac{Q}{k S} = \frac{2,406 \text{ ft}^3/\text{d}}{(850 \text{ ft/d}) \times 0.005} = 566 \text{ ft}^2
\]

\[d_w = 1.25 \text{ ft}\]

\[\text{Width} = \frac{269 \text{ ft}^2}{1.25 \text{ ft}} = 453 \text{ ft}\]

\[\text{Length} = \frac{2,477 \text{ ft}^2}{215 \text{ ft}} = 51.7 \text{ ft}\]

\[\text{Length:Width Ratio} = \frac{51.7}{453} = 0.11:1\]

4.7.6 VSB Design Summary

The examples above illustrate that each sizing method has its strength and weaknesses and there is not a consensus on how to size VSB wetlands for specific treatment objectives. It is yet to be proved that the smallest or the largest designs are unrealistic. If a design method produces a sizing factor that appears to be an anomaly, it should be
checked against the actual performance of existing wetland systems in that area (if possible), or at a minimum, against other sizing criteria. VSB sizing results are summarized below:

Table 4.6 VSB Sizing Results

<table>
<thead>
<tr>
<th>Method</th>
<th>BOD</th>
<th>TSS</th>
<th>Ammonia</th>
<th>Fecal Coliform</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crites &amp; Tchobanoglous, 1998</td>
<td>5,492 ft²</td>
<td>585 ft²</td>
<td>102,968 ft²</td>
<td>---</td>
</tr>
<tr>
<td>USEPA, 2000</td>
<td>6,727 ft²</td>
<td>1,144 ft²</td>
<td>Not recommended</td>
<td>Compliance likely</td>
</tr>
<tr>
<td>Kadlec &amp; Knight, 1996</td>
<td>2,477 ft²</td>
<td>2,690 ft²</td>
<td>6,515 ft²</td>
<td>2,350 ft²</td>
</tr>
<tr>
<td>Reed, Middlebrooks &amp; Crites, 1995</td>
<td>5,766 ft²</td>
<td>120 ft²</td>
<td>10,610 ft²</td>
<td>---</td>
</tr>
<tr>
<td>TVA, 1993 (prescriptive flows)</td>
<td>23,400 ft²</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

Obviously, the summary presented here is not a complete or exhaustive analysis of each design method. For actual design applications, the designer should refer to the original source material.
5. WETLAND VEGETATION

In order for plant establishment in a constructed wetland to be successful, the following criteria must be met:

- The plant species must be matched to the hydrology of the wetland.
- The plant material (seed, tuber, rhizome, pot, etc.) must be viable at the time of planting.
- Water level management during the startup phase must be compatible with the needs of the newly-establishing plants.

Meeting the three criteria above will ensure that vegetation will grow in the wetland. However, in selecting plant species, wetland designers must also take into account the following factors:

- Because it is generally cost-prohibitive to plant new stock at very high densities, selected plant species should establish well and be able to propagate new shoots through the spread of rhizomes to cover the wetland. Unfortunately, many non-native exotic plant species fall into this category.
- Selected plant species should be native to the region and acceptable to local regulators and the public.
- Selected plant species should be available through wetland plant nurseries located in the region of the project. Harvesting plant stock from natural wetlands, if done by hand, is extremely labor intensive. If heavy equipment is used, violation of wetland laws is likely unless permits have been obtained in advance from local regulators.

5.1 WETLAND PLANT HYDROLOGY

A useful approach to matching plants to the hydrology of the wetland is to use the plant indicator status categories originally developed by the U.S. Fish and Wildlife Service. This classification system is summarized below (U.S. Army Corps of Engineers, 1987):
### Table 5.1 Plant Indicator Status Categories

<table>
<thead>
<tr>
<th>Indicator Category</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Obligate Wetland</td>
<td>OBL</td>
<td>Plants that occur almost always (&gt;99%) in wetlands under natural conditions</td>
</tr>
<tr>
<td>Facultative Wetland</td>
<td>FACW</td>
<td>Plants that occur usually (67-99%) in wetlands but can also occur (1-33%) in upland areas</td>
</tr>
<tr>
<td>Facultative</td>
<td>FAC</td>
<td>Plants with a similar likelihood (33-67%) of occurring either in wetlands and nonwetlands (uplands)</td>
</tr>
<tr>
<td>Facultative Upland</td>
<td>FACU</td>
<td>Plants that occur sometimes (1-33%) in wetlands but occur more often (67-99%) in uplands</td>
</tr>
<tr>
<td>Obligate Upland</td>
<td>UPL</td>
<td>Plants that occur almost always (&gt;99%) in uplands</td>
</tr>
</tbody>
</table>

Plant indicator lists have been developed by the U.S. Fish and Wildlife Service for different regions of the United States and are available on the internet. The current web address is [http://www.nwi.fws.gov/bha/](http://www.nwi.fws.gov/bha/).

Generally, one of the following situations will be present in the constructed wetland:

**Figure 5.1 Wetland Planting Zones**
In FWS wetlands, the water is maintained over the rooting substrate; consequently only obligate (OBL) wetland plants can survive in this environment. Most emergent plant zones in FWS wetlands are designed with a water depth between 6 and 18 inches.

At water depths of 3 feet or greater, emergent plant can no longer survive. These will become open water areas within the FWS wetland and can either support submerged aquatic vegetation or suspended and floating algal mats. With increasing phosphorus concentrations, the competitive advantage shifts away from submerged aquatic vegetation towards algae. At phosphorus concentrations above 1 mg/L, algae will dominate over submerged aquatic plants (Moss B. et al., 1996) provided there is adequate retention time to allow algae blooms to occur. Consequently, some design methods (United States Environmental Protection Agency, 2000) recommend limiting the hydraulic retention time in open water zones or breaking the wetland design into multiple open water zones (See Section 4.6.2).

In VSB wetlands, there is an unsaturated zone at the top of the bed. Increasing the depth of this unsaturated zone (either through addition of insulating mulch or accumulation of plant detritus) shifts the competitive advantage away from obligate (OBL) wetland plants towards facultative wetland (FACW) plants. If the unsaturated rooting zone increases to 12 inches or greater, the competitive advantage shifts towards facultative (FAC) plant species.

5.2 PLANTING STOCK VIABILITY

Most planting stocks are rhizomes or tubers delivered from a wetland plant nursery. Bodies and shoots associated with root stock should be rigid to the touch. Bodies and shoots that are soft, mushy and appear rotten or decomposed should be rejected. Established root stock should contain new roots that are clean, white in appearance and rigid to the touch.

The energy stored in the rhizome provides the “fuel” to get the plant started; consequently larger tubers or rhizomes offer a better chance of success than small stock. Root stock in the form of tubers should be at least 5/8-inch in diameter. Root stock in the form of rhizomes should be at least 2 inches long. Plant count should be based on the number of roots or tubers, not the number of buds or shoots.

Potted plants, if used, should be in at least 4-inch pots. The soil/root mass should be the same size as the container (otherwise you are paying for a 2-inch plant in a 4-inch pot). Plants in peat pots should be well rooted through the sides and bottom of the pot and firmly anchored. If plants are growing at the time of installation, they should appear healthy with no leaf spots, insect damage, or wilting.

After delivery to the job site, material should be stored in the shade and kept cool and moist. Plant stock left out in the sun can cook off and be useless within a few hours.
5.3 START-UP WATER LEVEL MANAGEMENT

Wetland plants are adapted to a hydric environment and most have very little drought tolerance. If the plants are allowed to dry out during the establishment phase (where there is very little root network available for the plant to draw on), widespread mortality can occur.

For VSB wetlands, the water level should be raised up to the level of the bed for non-mulched systems and about 1” above the gravel bed (so the mulch is sub-irrigated) for mulched systems. The goal is to provide a continuous source of water to the plant root zone during establishment.

For FWS wetlands, the water level should be drawn down so it just covers the root media (“mud flat”). Emergent plants need access to the air for sunlight and oxygen, and if the new shoots are inundated for extended periods of time, the plants will drown out. As the plant shoots get taller, the water level can be gradually raised to design level but should be kept below the tops of the plants.

![Figure 5.2: FWS Wetland During Plant Establishment Phase](Image)

Photo courtesy of North American Wetland Engineering.

5.4 SELECTION OF WETLAND PLANT SPECIES

There are thousands of plant species adapted to grow in wetlands. Ideally, for use in a constructed wetland treating wastewater, plants should have a high capability of internal gas transport, tolerate high nutrient levels, colonize bare areas through rhizome spread and be readily available from local plant nurseries. Consequently, there are only a few
dozen plant species that have been used widely in constructed wetlands, and most experience is with only a handful of species.

The plant species that probably best meets the ideal goals outlined above is *Phragmites australis* (Common Reed). This plant has a long history of cultural use and is used almost exclusively for wetland treatment systems in Europe. However, *Phragmites* is considered exotic and aggressive by most natural resource agencies in the United States; consequently it is almost never used here.

Selection of plant species is governed by availability through nurseries, hydrologic tolerances, and being adapted to the project area. It is not possible to develop a universal plant list that will work everywhere in the United States. Plants that have been used successfully include:

Table 5.2 Commonly Used Wetland Plant Species

<table>
<thead>
<tr>
<th>Scientific Name</th>
<th>Common Name</th>
<th>Status</th>
<th>Region</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Carex nebrascensis</em></td>
<td>Nebraska sedge</td>
<td>OBL</td>
<td>Southwest</td>
</tr>
<tr>
<td><em>Carex stricta</em></td>
<td>Uptight sedge</td>
<td>OBL</td>
<td>Southwest</td>
</tr>
<tr>
<td><em>Iris missouriensis</em></td>
<td>Rock Mountain Iris</td>
<td>FACW, OBL</td>
<td>West</td>
</tr>
<tr>
<td><em>Iris pseudocorus</em></td>
<td>Yellow Iris</td>
<td>OBL</td>
<td>Midwest, Northeast</td>
</tr>
<tr>
<td><em>Iris versicolor</em></td>
<td>Blueflag Iris</td>
<td>OBL</td>
<td>Midwest, Northeast</td>
</tr>
<tr>
<td><em>Juncus balticus</em></td>
<td>Baltic Rush</td>
<td>FACW, OBL</td>
<td>Southwest</td>
</tr>
<tr>
<td><em>Scirpus acutus</em></td>
<td>Hardstem Bulrush</td>
<td>OBL</td>
<td>across US</td>
</tr>
<tr>
<td><em>Scirpus atrovirens</em></td>
<td>Green Bulrush</td>
<td>OBL</td>
<td>Midwest, East</td>
</tr>
<tr>
<td><em>Scirpus californicus</em></td>
<td>Bulrush (Restorer)</td>
<td>OBL</td>
<td>West</td>
</tr>
<tr>
<td><em>Scirpus fluviatilis</em></td>
<td>River Bulrush</td>
<td>OBL</td>
<td>Midwest, East</td>
</tr>
<tr>
<td><em>Scirpus validus</em></td>
<td>Softstem Bulrush</td>
<td>OBL</td>
<td>across US</td>
</tr>
<tr>
<td><em>Typha latifolia</em></td>
<td>Cattail, Broadleaf</td>
<td>OBL</td>
<td>across US</td>
</tr>
<tr>
<td><em>Typha angustifolia</em></td>
<td>Cattail, Narrowleaf</td>
<td>OBL</td>
<td>northern US</td>
</tr>
</tbody>
</table>

Table 5.2 is provided as a general planning guide. More detailed regional information can be obtained from wetland plant nurseries. There are also many ornamental plants suitable for use in small-scale constructed wetland systems (Pineywoods RC&D, 2001).
Figure 5.3 Residential VSB Wetland with Ornamental Plants

Photo courtesy North American Wetland Engineering.
6. CONSTRUCTED WETLANDS – CURRENT LEVEL OF UNDERSTANDING

Constructed wetlands are still in their evolutionary stage. While constructed wetlands can be designed to achieve overall treatment objectives, internal mechanisms inside the “black box” have not been quantified. Future advances in the design arena will likely develop accurate models suitable for widespread use that can quantify reduction of externally applied loads vs. internally produced loads for substances such as TSS and BOD.

The current state of the art is a “semi-empirical” (part rational, part empirical) approach. Designers typically use one or more of the methods outlined in Section 4 to quantify the overall size that a wetland needs to be to achieve a certain level of treatment. Once the size has been estimated, the designer then looks at the internal configuration of the wetland and designs separate wetland compartments to drive the treatment reactions to their desired outcome. Since designers currently lack data on internal compartments, they are often sized based on best judgment (empirically) or equations developed for other treatment technologies, such as facultative lagoons. A good example of this is the USEPA design approach (United States Environmental Protection Agency, 2000) which breaks up the design of a FWS wetland into separate emergent and open water compartments.
7. Reference List


