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Energy requirements for nitrification and biological nitrogen removal in engineered wetlands

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ABSTRACT

Nitrogen in wastewater degrades aquifer and surface water quality. To protect water quality in the United States, nitrogen discharge standards are strict: typically 1.0 mg/L NH₄-N for discharge to surface water and 10 mg/L total nitrogen (TN) for discharge to soil. Passive constructed wetland treatment systems cannot meet the nitrification standards discussed in this paper, using loading rates commonly considered to be cost-effective based on economic conditions in North America. Although partial nitrification can be achieved with some vertically or intermittently loaded, subsurface flow (SSF) wetlands, complete nitrification cannot be achieved in these passive wetland treatment systems. Engineered wetlands (EWs) use mechanical power inputs via pumping of air or water to nitrify wastewater, and have evolved in large part to nitrify wastewater. The design energy requirements for these power inputs have yet to be described in the wetland treatment literature. Our paper investigates the energy and area requirements of three wetland technologies: aerated subsurface flow, tidal flow, and pulse-fed wetland treatment, compared to a mechanical activated-sludge treatment system.

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1. Introduction

Nitrogen in wastewater degrades aquifer and surface water quality. To protect water quality in the United States, nitrogen discharge standards are strict: typically 1.0 mg/L NH₄-N for discharge to surface water and 10 mg/L total nitrogen (TN) for discharge to soil. Near complete nitrification ($\geq 95\%$) is needed to meet these treatment standards for domestic wastewater. In activated-sludge wastewater treatment systems, nitrification sets a high aeration energy demand compared to secondary treatment alone (Metcalf and Eddy Inc., 2003).

Passive constructed wetland treatment systems cannot meet the nitrification standards discussed in this paper, using loading rates commonly considered to be cost-effective based

on economic conditions in North America. Although partial nitrification can be achieved with some vertically or intermittently loaded, subsurface flow (SSF) wetlands (Laber et al., 1997; Farahbakhshazad et al., 2000; Aguirre et al., 2005) or hybrid (combination) systems (Vymazal, 2005), complete nitrification cannot be achieved reliably in passive wetland treatment systems unless loaded at very low rates that are impracticable for most applications.

Engineered wetlands (EWs) have evolved in large part to nitrify wastewater (Behrends et al., 2001; Austin et al., 2003; Brix and Arias, 2005; Molle et al., 2005; Wallace et al., 2006). EWs use mechanical power inputs via pumping of air or water to nitrify wastewater. The design energy requirements for these power inputs have yet to be described in the wetland treat-

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Table 1 – Wastewater characteristics used for all process design calculations

Flow (m ³ /d)	1000
COD _{u,in} ^a (mg/L)	300
COD _{u,out} ^a (mg/L)	10
TN _{in} ^b (mg/L)	60
TN _{out} ^c (mg/L)	10

^a COD_u is the labile fraction of chemical oxygen demand (COD).
^b Influent total nitrogen (TN) is 100% organic-N, of which 97% hydrolyzes to ammonium nitrogen (NH₄-N).
^c Effluent TN is comprised of 2.0 mg/L organic-N, 1.0 mg/L NH₄-N, and 7.0 mg/L NO₃-N.

ment literature. Our paper investigates the energy and area requirements of three technologies: aerated subsurface flow, tidal flow, and pulse-fed wetland treatment.

As shall be seen, energy requirements do not depend strictly on a technology. Rather, site conditions, design decisions, and total nitrogen removal requirements substantially affect design energy requirements. Obviously, wastewater characteristics can induce large differences in specific energy requirements (kWh/m³ treated) across various applications of the same technology. Therefore, we structure this paper as a design exercise using the same hypothetical domestic wastewater, discharge standards, and flow for all technologies. Further, these three engineered wetland technologies will be compared to a conventional, modified Ludzack–Ettinger (MLE) activated-sludge process design. The authors are experienced in the design of the technologies described in this report.

2. Methods

Methods used to perform the design exercise are presented here. Actual calculations and process-specific components are presented in Section 3. The design exercise is conducted using a hypothetical wastewater (Table 1) to examine the energy requirements for the conventional (activated sludge) treatment as compared to the EW technologies. A design flow of 1000 m³/d was chosen for all systems. Influent total nitro-

gen is assumed to be entirely comprised of ammonium and organic nitrogen (TKN). The influent COD_u:TKN ratio is 6:1, which provides a stoichiometric excess to drive denitrification (Metcalf and Eddy Inc., 2003). Additional assumed influent characteristics include a water temperature of 20 °C, pH 7.5, relative humidity = 80%, and TDS = 500 mg/L. Energy demand for pretreatment is ignored for all systems. Only operational electrical energy requirements are considered in this analysis. There is no attempt at an embodied energy analysis.

A Modified Ludzack–Ettinger (MLE), activated-sludge treatment system is the conventional technology comparison (Fig. 1a). The EW technologies include a two-cell horizontal subsurface flow wetland system, a six-cell tidal flow wetland system, and a two-cell pulse-fed wetland system (Fig. 1b–d, respectively).

Treatment performance of the hypothetical systems is justified by literature values for the MLE process (Metcalf and Eddy Inc., 2003), aerated wetlands (Wallace et al., 2001; Wallace et al., 2006), tidal flow wetlands (Behrends et al., 2001; Austin, 2006), and pulse-fed wetlands (Molle et al., 2005). For the pulse-flow example, the French design of Molle et al. (2005) was chosen, although the Danish approach described in Brix and Arias (2005) would likely result in comparable process energy requirements.

Energy demand is based on the need to oxidize organic carbon and ammonia (Eq. (1)). Theoretical nitrogenous dissolved oxygen demand for combined ammonia and nitrite oxidation is 4.57 kg O₂/kg NO₃ formed. Denitrification has the equivalent mass value of 2.86 kg O₂/kg NO₃ converted to dinitrogen gas (N₂). Assuming sufficient carbon for heterotrophic denitrification, theoretical net dissolved oxygen demand is 1.74 kg O₂/kg of ammonia converted to N₂.

$$\text{DO demand [kg/d]} = \text{COD}_u + 4.57(\text{NO}_3)_f - 2.86(\text{NO}_3)_u \quad (1)$$

where COD_u is the chemical oxygen demand utilized (=bCOD_{in} – bCOD_{out}), kg/d; bCOD is the biologically available COD, kg/d; (NO₃)_f is the nitrate formed from ammonia, kg/d; (NO₃)_u is the nitrate utilized in denitrification, kg/d.

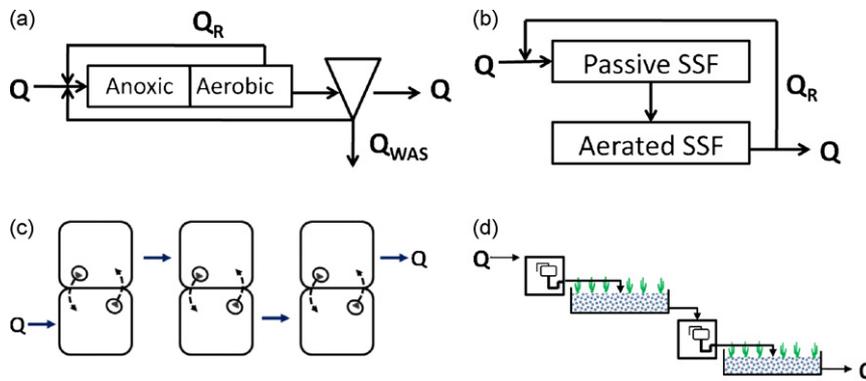


Fig. 1 – System schematics. (a) MLE: activated sludge with pre-anoxic denitrification from nitrified process recycle; (b) aerated wetland: organic carbon removal in passive first-stage horizontal SSF cell, with pre-anoxic denitrification from nitrifying (aerated) second-stage horizontal SSF cell; (c) tidal flow wetland: three paired cells with reciprocating pumped flow in each cell pair and overflow drain to next pair in series. The first stage rotates three paired cells in parallel to allow resting, but not pictured for simplicity; (d) pulse-fed wetland with siphon-dosed cells. Final polishing free water surface (FWS) wetland not depicted. All processes are assumed to include screened influent.

Theoretical dissolved oxygen demand in the tidal flow wetland is based on Eq. (2). Flood and drain (tidal flow) cycles transfer oxygen via cation exchange (Tanner et al., 1999; Austin et al., 2003). When reactors are flooded, NH_4^+ ions adsorb to negatively charged media surfaces. During draining, air is drawn down into media pore spaces. By Fick's law, the half-time of oxygen diffusion from the air–water interface across thin biofilms ($<100 \mu\text{m}$) is on the order of 1 s. Oxygen demand within the biofilm itself will determine time to oxygen saturation. Adsorbed NH_4^+ ions rapidly nitrify under these conditions (McBride and Tanner, 2000; Austin et al., 2006). Because nitrification occurs in drained phases, there is little apparent competition for oxygen within biofilms to limit nitrification. In the next flood phase, NO_3^- and NO_2^- ions desorb to bulk water. Nitrate and nitrite desorbing into a high organic carbon environment result in denitrification. The tidal flow wetland is assumed to have an aggregate with sufficient cation exchange capacity (CEC) to support this process. Theoretical dissolved oxygen demand in the pulse-fed wetland is assumed to be primarily governed from Eq. (2), and marginally from Eq. (1).

$$\text{DO demand}[\text{kg/d}] = \text{COD}_u - 2.86(\text{NO}_3)_u \quad (2)$$

where COD_u and $(\text{NO}_3)_u$ are as previously defined.

The treatment area for each wetland is determined using design criteria unique to each system. Area is included to compare the trade-off between energy and treatment area requirements. Only the wet treatment footprint is calculated to avoid the complications of berms, access, setbacks, etc., that are necessary in any final design.

2.1. MLE process design

Theoretical dissolved oxygen demands for the MLE process are presented in Eq. (3). For the small treatment system of this design exercise, aerobic stabilization of biosolids prior to reed-bed composting is assumed. With an assumed 60% biosolids digestion (Metcalf and Eddy Inc., 2003), terms for aerobic digestion oxygen demand and reduction of oxygen demand by wasting of biosolids balance to within 8% of each other. Therefore both biosolids terms are dropped from the calculation. Eq. (1) then becomes the method of calculating theoretical dissolved oxygen demand.

$$\text{DO demand}[\text{kg/d}] = \text{COD}_u + 4.6(\text{NO}_3)_f - 2.86(\text{NO}_3)_u - P_x(1.42) + 2.3P_d \quad (3)$$

where COD_u , $(\text{NO}_3)_f$, $(\text{NO}_3)_u$ as previously defined; P_x is the cells (VSS) produced that are wasted, kg/d; 1.42 is the conversion factor for cells to COD; P_d is the cells (VSS) destroyed in anaerobic digestion is the $0.4(1.42P_x)$.

Based on wastewater and site characteristics listed above, a preliminary design was developed to compute energy demands for the MLE Process. The design was based on extended-air plug flow activated-sludge process with a total HRT of 20 h, 2 h in the anoxic reactor (volume = 83 m^3) and 18 h in the aerated section ($V = 750 \text{ m}^3$). The aerated section was modelled using six theoretical complete mix reactors

in series in order to optimize the aeration requirement estimates.

The anoxic reactor (compartment) mixing was accomplished with two, low speed turbine (vertical shaft) mixers. Oxygen demand in the anoxic reactor was assumed to be met entirely by nitrate from process recycle by adjusting the apparent COD_u removal in the compartment so that no oxygen demand beyond that supplied by the process recycle was exerted.

Aeration demand for COD_u and TKN oxidation are computed on cell-by-cell basis and compared to aeration demands to insure mixing. A safety factor of 1.2 was applied to the COD_u to provide additional air to insure COD_u does not inhibit nitrification. Results of oxygen demand calculations are presented in Section 3. In the aerated reactors, total oxygen demand is converted to an air flow rate based on diffused air oxygenation efficiencies. Diffusers were fine bubble flexible membrane type with a standard oxygen transfer efficiency (SOTE) of 25% at submergence depth of 4.9 m. Correction factors result in a field oxygen transfer efficiency (FOTE) as presented in Section 3. Area of the MLE system was based on a treatment basin depth of 5 m at a hydraulic retention time (HRT) of 12 h, and redundant clarifiers.

2.2. Wetland process design

All three wetland types employ process pumps. Hydraulic and shaft power requirements are calculated by Eqs. (4) and (5). Electric power requirements are calculated by Eq. (6). Flow capacity and total dynamic head differ between systems. Because all pumping applications involve low heads, a pump efficiency of 75% and motor efficiency of 90% is held constant for all wetland systems:

$$P_h = \frac{q \rho g h}{3.6 \times 10^6} \quad (4)$$

where P_h is the hydraulic power, kW; q is the flow capacity, m^3/h , ρ is the density of fluid (1000 kg/m^3); g is the gravitational force (9.81 m/s^2); h is the total dynamic head, m.

$$P_s = \frac{P_h}{\eta} \quad (5)$$

where P_s is the Shaft power, kW; P_h is the hydraulic power, kW; η is the pump efficiency (0.75).

$$P_e = \frac{P_s}{\eta} \quad (6)$$

where P_e is the electrical power, kW; P_s is the Shaft power, kW; η is the motor efficiency (0.9).

2.3. Aerated wetland system

The aerated horizontal flow wetland process design includes two cells; a passive cell for initial COD_u removal followed by an aerated cell. Design treatment is based on first-order kinetics with a volumetric rate coefficient of 0.9 d^{-1} for COD_u removal and 3.3 d^{-1} for nitrification (unpublished data). A bed depth of 2.0 m has been chosen for this example (consistent with

recent designs for cold-climate applications), which results in a SOTE of 4.8%/m, or 9.6% total, based on literature values (Wallace et al., 2007). Power requirements for this system include the blower and the process recycle pump. Area of the treatment wetland is a function of the design depth and HRT extracted from the first-order design equation (Eqs. (7) and (8)). The required treatment volume to satisfy design HRT is calculated using a design porosity of 25% for the passive COD_u removal zone and 30% for the aerated nitrification zone (Eq. (9)):

$$\ln \left(\frac{C_0 - C^*}{C_i - C^*} \right) = k_v \tau \varepsilon \quad (7)$$

where C_0 is the initial concentration, g/m³; C is the effluent concentration, g/m³; C^* is the background concentration, g/m³; k_v is the volumetric rate coefficient, d⁻¹, τ is the detention time, d; ε is the porosity of wetland media, dimensionless.

$$\tau = \frac{hA}{Q} \quad (8)$$

where h is the water depth, m; A is the wetland area, m²; Q is the flow rate, m³/d; τ is the detention time, d.

$$A = \frac{Q}{k_v t \varepsilon} \ln \left(\frac{C_0 - C^*}{C_i - C^*} \right) \quad (9)$$

where A , C_0 , C_i , C^* , k_v , τ , ε as previously defined.

2.4. Tidal flow wetland system

In the tidal flow process, the organic carbon is primarily oxidized with nitrate, which is produced through flood and drain cycles. Therefore, calculation of the energy demand for a tidal flow wetland process does not require computing specific oxygen demands. Rather, energy demand is a function of the pump power and run times that support design flood and drain cycles.

The tidal flow wetland system design in this exercise is assumed to employ three reciprocating paired cells in series (Fig. 1c). Water is moved from cell to cell using low-head, high-volume pumps, typically axial flow or mixed flow propeller pumps. Total dynamic head is computed based on cell depth plus frictional losses in piping from cell to cell. Flood and drain cycles are eight per cell per day, a rate established by pilot system treatment performance (Austin et al., 2003). Tidal wetland pump selection and power requirements are presented in Section 3.

Sizing of the tidal flow cells was based on COD_u loading limitations to avoid clogging from growth of heterotrophic biofilms at an aggregate size d_{50} of 8 mm and uniformity coefficient (UC) less than 4 (Austin et al., 2007). The aggregate is specified as lightweight expanded shale, slate, or shale with a minimum CEC of 4.0 milliequivalents per 100g. The CEC specification is sufficient to support complete nitrification within the specified flood and drain cycles (Austin et al., 2003).

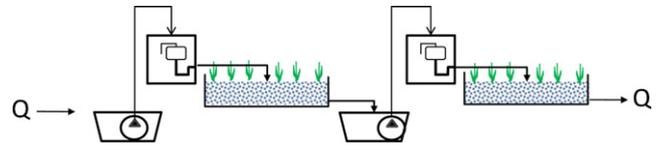


Fig. 2 – Pulse-flow wetland configuration at a flat site.

2.5. Pulsed flow wetland system

The pulsed flow wetland is based on French designs (Molle et al., 2005) which nitrify completely, but only partially denitrify. In the pulsed flow system siphons dose the surface of each wetland cell to a depth of 3–5 cm over a few minutes. Flow begins as saturated, but transitions to an unsaturated condition as the slug of water percolates downward and smears into gravitational water films exposed to air with pore spaces. Oxygen transfer occurs passively and rapidly across the water films. Although it is possible to model this process (Langergraber and Simunek, 2005) empirical dosing criteria (Molle et al., 2005) were used instead to ensure sufficient oxygen transfer.

The pulsed flow system uses three first-stage beds dosed in rotation and two second stage beds dosed in rotation (Molle et al., 2005). All beds are of equal area. Hydraulic loading is limited to 15 cm/d calculated over the entire first stage area. A higher hydraulic loading rate would interfere with oxygen transfer.

2.5.1. At a sloped site

The pulsed flow system is assumed to have site conditions favourable for gravity flow via siphons for two cells in series (Fig. 1(d)) to achieve complete nitrification. An elevation differential of 4 m is assumed from inlet to discharge. It should be noted that the pulsed flow technology described by Molle et al. (2005) is not designed to be used with recycle. Oxygen transfer is dependent on light hydraulic loading. A 2:1 recycle for denitrification would require expanding the bed sizes by a factor of 3. In keeping with the purely passive nature of flow on a sloped site, denitrification is achieved in a FWS wetland sized per Kadlec and Wallace (2008).

2.5.2. At a flat site

Additionally, a design configuration utilizing pulse flow at a flat site is explored using a combination of continuous flow pipes, siphons, and pump basins (small ponds) (Fig. 2). The authors are not aware of a system designed by this method. However, the design logic for it is strong as shall be seen in Section 3. Site elevation differentials from mean pump basin water surface to bed bottom drain surface are assumed to be 2 m and established by site excavation using common barrow and fill methods in the site-grading plan.

3. Results

3.1. MLE process energy requirements

Power requirements for the MLE process is the sum of following: power required to meet DO demands, anoxic mixing,

Table 2 – Power requirements for model MLE system

Component	Power draw (kW)	Number of units operating	Operational time (h/d)	Power consumption (kWh/d)
Anoxic mixers ^a	3.7	2	24	178
Aeration blowers	22.4	1	24	487
Clarifier drive	2.2	2	24	108
Process recycle pump ^b	4.2	1	24	75
RAS pump ^c	2.1	1	24	36
Total				884

^a Anoxic mixer design: impeller diameter 1 m, 1 rpm and k-value of 4.8.
^b Process recycle flow is 4000 m³/d, total dynamic head 5 m, 80% pump efficiency and 90% motor efficiency.
^c Based on Q_{RAS} (return activated-sludge flow) of 500 m³/d, total dynamic head 5 m, 80% pump efficiency and 90% motor efficiency.

clarifier drives and process and RAS pumping. The computation of power supply to meet DO demands is a multistep process and is presented below, followed by a table showing other power demands and the total system power demand.

Dissolved oxygen demand for the model MLE treatment process was computed using Eq. (10). From the dissolved oxygen demand, the air flow requirements were computed based on site conditions as shown in Eq. (11). A positive displacement blower was selected to meet the air flow requirements and the brake power requirements of the blower were multiplied by 1.2 to determine motor power draw.

$$\text{DO demand (kg/d)} = 1.2 \text{COD}_u + 4.6(\text{NO}_3)_f - 2.86(\text{NO}_3)_u \\ = (319 + 276 - 134) \text{ kg/d} = 460 \text{ kg/d} \quad (10)$$

where COD_u is the mass COD consumed (based on 290 mg/L consumed), kg/d; (NO₃)_f is the mass NH₃ nitrified (based on 58.8 mg/L nitrified), kg/d; (NO₃)_u is the nitrate utilized in anoxic reactor (based on 80% of (NO₃)_f is the 47 mg/L), kg/d.

$$\text{air flow (m}^3\text{/d)} \\ = \frac{\text{DO demand}}{\text{transfer efficiency} \times \text{air density} \times \text{percent O}_2} \quad (11)$$

where DO demand as defined in Eq. (3), kg/d; transfer efficiency is the diffused air system oxygen transfer efficiency under design condition; air density is the density of air at site conditions (30 m elevation, 30 °C, 80% relative humidity); percent O₂ is the percent oxygen in air = 23.2%.

Oxygen transfer efficiency of the diffused air system under design conditions (FOTE) was calculated to be 11.1% using the methods presented in Metcalf and Eddy Inc. (2003). The fol-

lowing correction factors were used: alpha = 0.65, beta = 0.9, theta = 1.024 and minimum DO of 2.0 mg/L.

Given the above methods and assumptions, the air flow requirements to supply diffused DO requirements was computed as approximately 14 m³/min at a gage outlet pressure of 55.1 kPa. Based on these criteria a positive displacement blower was selected based on Dresser Roots “Whispair” performance curve (Dresser Roots; Houston, TX). Interpolation between 2950 and 4000 rpm on frame size 406 J yielded a shaft power requirement of 18.7 kW. Motor draw is computed as 1.2 times shaft power or 22.4 kW. Other power requirements and assumptions along with total demand are listed in Table 2.

3.2. Wetland process energy requirements

Hydraulic and shaft power requirements and electric power requirements for each wetland technology are presented here. Differences in flow capacity and total dynamic head differ between systems and are also noted in this section. Additionally, process area requirements (footprint) for each wetland technology are calculated.

3.3. Aerated wetland

Process energy for the aerated wetland process (Fig. 1b) is comprised of the energy required for the blowers and the energy required to run the process recycle pump. The blower for the wetland in this design example operates 24 h a day at 8 psig to clear aeration orifices. The blower size of 18.7 kW (25 HP) results in a power consumption of 449 kWh/d. Process recycle pumps are capable of operating at 2000 m³/d (2Q_{AVG}) with 4.3 m of TDH, resulting in a power consumption of 36 kWh/d. Total power consumption for this scenario is 485 kWh/d (Table 3).

Table 3 – Power requirements for model aerated wetland system

Component	Power draw (kW)	Number of units operating	Operational time (h/d)	Power consumption (kWh/d)
Blowers	18.7	1	24	449
Process recycle pump	1.5	1	24	36
Total				485

Wetland area was calculated as the sum of the area required for total COD removal (Eq. (9), C_{COD}^{*} = 5 g/m³, k_v = 0.9 d⁻¹) and that required for complete nitrification (Eq. (9), C_{NH₄}^{*} = 0 g/m³, k_v = 3.3 d⁻¹).

Table 4 – Power requirements for model tidal flow wetland system

Component	Power draw (kW)	Number of units operating	Operational time (h/d)	Power consumption (kWh/d)
Process pumps	4.4	6	8 each 48 total	211
Total				211

(a) Six process pumps move cell pore volume to next cell and recycle from Cell 6 to Cell 1.

Table 5 – Power requirements for model pulse-fed wetland system (sloped site)

Component	Power draw (kW)	Number of units operating	Operational time (h/d)	Power consumption (kWh/d)
Process recycle pump	3.0	1	24	72
Total				72

3.4. Tidal flow wetland

The tidal flow wetland process energy requirement is the sum of the energy required for the pumps moving the wastewater from cell to cell. The tidal flow wetland system inlet cells were sized based on an inlet (first stage) COD_u loading rate of 0.10 kg COD_u/(m² d) to avoid clogging from formation of heterotrophic biofilms (Austin et al., 2007). The resulting first stage treatment media (lightweight expanded shale aggregate) has volume of 3000 m³ with a chosen treatment aggregate depth of 1.0 m. The underdrain media depth is 0.5 m. Each reciprocating cell pair in the tidal flow wetland cell has a surface area of 1000 m² divided into two equal cells of 500 m² each. Total first stage area is 3000 m² of which 1000 m² is active and 2000 m² is resting on rotation. The second and third stages are each comprised of one reciprocating pair. Total surface area is 5000 m² of which 3000 m² is active. Flow between stages is by gravity through an overflow drain. Pumps reciprocate flow within each cell pair, completely emptying the pumped cell and filling the receiving cell. Each pump must move the pore volume of one cell to the other in a reciprocating pair plus influent received during each cycle. Treatment aggregate has a long-term effective void fraction of 0.25. The net void fraction of the underdrain is 0.5 due to hollow conveyance structures through the cell length. Thus, the total pore volume is 250 m³/cell.

To achieve the requisite minimum of 8 cycles d⁻¹ and have “resting periods” during each cycle, the minimum cell drain time is 60 min. The pump flow rate is the cell pore volume divided by drain time plus the influent flow rate per cell. The pump flow rate is therefore 250 m³/h. Pump total dynamic head is computed from the total cell depth plus frictional losses in piping from cell to cell. Total dynamic head (TDH) is estimated as 4.3 m. For each sump, a 4.3 kW (6 HP) propeller pump was selected, resulting in a daily power consumption of 211 kW (Table 4).

3.5. Pulsed flow wetland

Pump power requirements for pulsed flow wetlands were calculated using Eqs. (4) and (5) for hydraulic and shaft horse power. Power requirements and area calculations are presented for both a sloped site and a flat site.

3.5.1. At a sloped site

If process recycle were used at a flat site power requirements for a pulsed flow wetland system at a sloped site would be small, as the process recycle pump would be the only component requiring a power input (Table 5). The sloped site is assumed to present a static head of 3.0 m for recycle pumping. The process recycle pump is nominally sized at 2.8 kW (3.7 HP) and is assumed to run continuously at a TDH of 7.3 m. The selected pump motor horsepower rating is 3.0 kW (4.0 HP), resulting in a daily power consumption of 72 kWh/d.

Process recycle, however, would impose unrealistically large area requirements on bed size. At a hydraulic loading rate 0.15 m/d for a forward and recycle flow total of 3000 m³/d, the first stage area is 20,000 m² of three equally sized cells. With two second-stage cells the total treatment area is approximately 33,330 m². In contrast, per Molle et al. (2005) the correct total area for a forward flow of 1000 m³/d is 11,110 m².

Denitrification in a surface flow polishing wetland assumes that half of the 60 kg/d TN load is denitrified in pulsed flows, leaving 20 kg/d to be denitrified at rate of 5 kg TN/ha d (Kadlec and Wallace, 2008). Denitrification to 10 mg/L TN requires an additional 40,000 m² of FWS wetland. Although this area is large, it is more economical to build than the pulsed flow beds because it does not require the use of aggregate.

3.5.2. At a flat site

Each first stage cell is 6667 m². The operating cell receives all flow. Dosing this cell to a depth of 4 cm over 5 min entails a dose volume of 267 m³ at an application rate of 3200 m³/h. A pump of that size is impracticably large and expensive

Table 6 – Power requirements for model pulse-fed wetland system (flat site)

Component	Power draw (kW)	Number of units operating	Operational time (h/d)	Power consumption (kWh/d)
Process pumps	1.5	2	24	72
Total				72

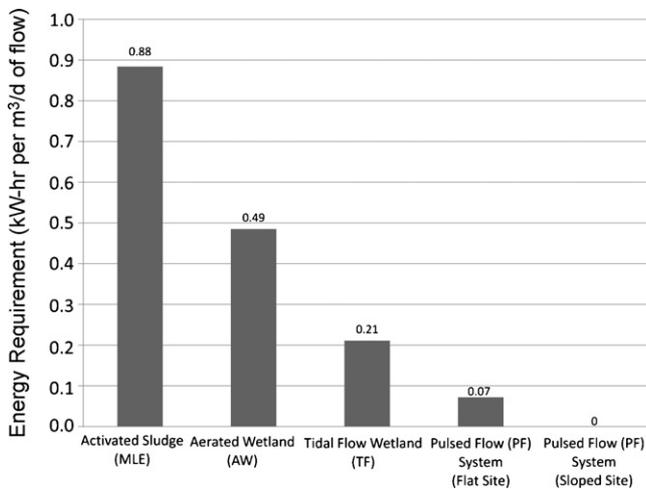


Fig. 3 – Summary of process energy requirements.

whereas a siphon (or slaved siphons in parallel) can deliver that flow at a fraction of the cost. Moreover, starting power demand for such a large pump would significantly increase the total power consumption and expense of electrical supply. A smaller, continuous-flow pump feeding siphon-dosing boxes (working volume of 267 m³ each) is simple and cost effective.

Pump power for the flat site is calculated at a TDH of 4.3 m. Flow is 42 m³/h for pumps feeding the siphon boxes. Sufficient flow equalization to avoid peak flow pumping requirements is assumed. Power requirements for a pulsed flow wetland system at a flat site are approximately the same for sloped site with recycle, and are summarized in Table 6. Pump sizes have been rounded up to the nearest commercially available pump sizes.

As in the sloped site, denitrification is achieved in a 4 ha FWS wetland.

4. Discussion

Along with technology-specific energy requirements, site conditions, design decisions, and total nitrogen removal requirements substantially affect the overall energy requirements for a given treatment system. Fig. 3 summarizes the process energy requirements for the MLE activated-sludge system along with the four wetland technologies of this design exercise (aerated, AW; tidal flow, TF; pulsed flow, PF; flat and sloped).

The calculated electrical power requirement for the wetland systems are 0–56% of that of MLE system. In the case of AW and TF wetlands, these low energy requirements are due to reduced dissolved oxygen demand (AW and TF) and the better electromechanical efficiency of pumps compared to diffused air supplied by blowers (TF). In the case of AW, the reduced energy requirement comes from passive removal of COD_u prior to meeting the dissolved oxygen demand of nitrification. In an AW system, aeration for COD_u would result in much higher energy requirements due to the lower SOTE of aeration within a packed bed compared to that of open water.

Nitrification in the MLE system is a major fraction of the energy demand due to the inefficiencies of meeting dissolved

oxygen demands. Recycle of nitrified process water to an anoxic first-stage recovers some of the oxygen used for nitrification but requires pumping and mixing energy. From Eq. (5), it is apparent that the theoretical DO demand due to nitrification alone is approximately 262 kg/d for the MLE system. This is 30% of total DO demand of 884 kg/d for the MLE system.

Because oxygen is sparingly soluble in water, especially warm wastewater, significant energy is required to overcome the thermodynamically unfavourable process of meeting the dissolved oxygen demand with diffused air. Field oxygen transfer efficiencies can be increased but significant increases usually involve sophisticated or expensive equipment with inherent operation and maintenance problems.

In contrast, there is no nitrification component to theoretical DO in the tidal wetland system. Per Eq. (2) and Table 1, approximately half of COD_u is met by nitrate produced in flood and drain cycles. Nitrate is highly soluble in water, thus there is no solubility limitation to transfer of oxygen equivalents to water via nitrate. Remaining COD_u is easily met by oxygen transferred in flood and drain cycles and anoxic mechanisms common to wetland treatment systems.

In pulsed flow wetlands, the empirical design criteria as described by Molle et al. (2005) meet theoretical DO by oxygen transfer across gravitational water films. It is unclear from the literature to what degree CEC of wetland aggregate assists nitrification. However, it is clear from the design experience of tidal flow wetlands that attention to aggregate CEC can only have a positive effect on treatment. It may even allow a higher hydraulic loading because desorbed nitrate would satisfy a substantial fraction of DO demand. If so, denitrifying recycle could be used without increasing the size of the pulsed flow beds, but would significantly reduce the size of the FWS polishing wetland.

For the tidal and pulsed flow wetland pumps are efficient means of oxygen transfer. Low head pumps can readily achieve hydraulic efficiencies of 75%. Assuming a shaft power efficiency of 75% and a motor efficiency of 90%, the overall energy-to-water efficiency is 51%. Tidal and pulsed flow wetland systems harness this efficiency by moving water to utilize a vast surface area of cation exchange sites to provide oxygen rather than force the oxygen into bulk water. This process is highly efficient compared to a FOTE of 11% for the diffused air system. By the stoichiometry and this design exercise, it has been shown that tidal flow systems are energy efficient for overall treatment and removing nitrogen from wastewater. The pulsed flow system is even more energy efficient, but at the cost of a substantially larger surface area.

The aerated wetland system is assumed to have the same field transfer factors as the MLE system. The AW FOTE is 4.3%. Despite a low FOTE, aerated wetland system that uses passive COD_u removal and aeration for nitrification ends up having a net process energy efficiency of approximately two times higher than the MLE system.

Process energy is not the only consideration a design engineer must take into account. Often, wastewater system designs are dictated by site constraints such as topography, available land, and/or climate. A summary of the footprint area for each technology evaluated is presented in Fig. 4. Generally, as required process energy increases, treatment footprint decreases. The MLE activated-sludge system has the

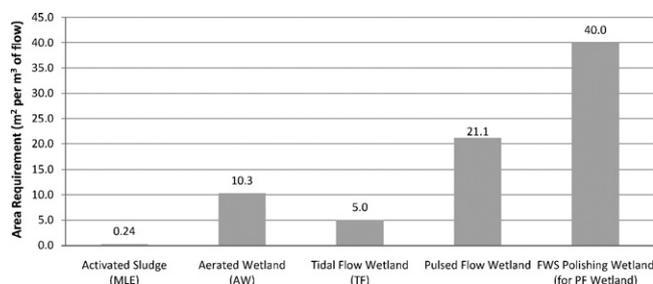


Fig. 4 – Summary of process area requirements.

highest process energy requirement ($0.88 \text{ kWh}/(\text{m}^3 \text{ d})$) and the lowest area requirement ($0.24 \text{ m}^2/(\text{m}^3 \text{ d})$). The aerated wetland system evaluated is estimated to use about half of the process energy ($0.49 \text{ kWh}/(\text{m}^3 \text{ d})$), and would require roughly forty times more area than what is required of the MLE system ($10.3 \text{ m}^2/(\text{m}^3 \text{ d})$). The tidal flow wetland system is efficient in both energy use ($0.21 \text{ kWh}/(\text{m}^3 \text{ d})$) and area requirement ($5.0 \text{ m}^2/(\text{m}^3 \text{ d})$). A pulsed flow wetland system followed by a FWS wetland for denitrification has the lowest energy requirement ($0.07 \text{ kWh}/(\text{m}^3 \text{ d})$) but requires the most land among the technologies evaluated (pulsed flow cells: $21.1 \text{ m}^2/(\text{m}^3 \text{ d})$), followed by FWS polishing cells of $40.0 \text{ m}^2/(\text{m}^3 \text{ d})$.

Pulsed flow wetlands developed in France operate on a forward flow basis without recycle (Molle et al., 2005). On a sloped site, the high level of treatment that can be achieved with zero electrical energy inputs is a remarkable technical achievement. For the purposes of this paper, a surface flow wetland was chosen per established design criteria to complete denitrification to a 10 mg/L TN effluent standard. The authors recognize that other unit process could denitrify in much less space than the 4 ha specified. However, these methods would require supplemental carbon feeds, which is an energy subsidy. A purely passive process without carbon feed was chosen to demonstrate that advanced nitrogen removal in wastewater treatment can be designed with zero anthropogenic energy operating inputs.

Consideration of temperature as a factor in energy efficiency is not addressed in this paper. It affects oxygen transfer efficiency in aerated systems and process kinetics overall. Kinetics, of course, influence energy efficiency. The most important effect of temperature is not on energy efficiency, however, but on design constraints. In cold climates where winter frost depth is greater than 1 m, such as in Canada and many northern and mountain states in the USA, wetland beds will freeze solid if not insulated. Freezing of a wetland bed can be avoided by insulating it with mulch (Wallace et al., 2001). Insulated, aerated wetlands are used successfully for cold-climate nitrification. Other wetland technologies cannot be used if nitrification is needed, making the energy comparison a moot point in severe northern climates. In hot climates where water temperature is greater than 25°C , oxygen transfer limitations make control of a small MLE system difficult because chronic low DO conditions tend to favor filamentous (sludge-bulking) bacteria (Wanner, 1994). In moderate to hot climates, tidal flow and pulsed flow wetlands are not constrained by freezing and are clearly the most appropriate technology choices for energy efficiency.

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