Constructed wetlands for wastewater treatment in Nepal

Konstruert våtmark for avløpsrensing i Nepal

Philosophiae Doctor (PhD) Thesis

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Preface/Acknowledgements

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The thesis contains five manuscripts (Paper I-V). Paper I deals with the comparative assessment of the existing centralized wastewater treatment system in Kathmandu with constructed wetland (CW) based decentralized wastewater treatment alternatives. Paper II deals with the performance study of horizontal and vertical flow planted beds. Paper III investigates the dewatering and stabilizing performance of the sludge in sludge drying reed beds. Paper IV deals with the hydraulic conditions and the reaction kinetics of the two horizontal flow (HF) subsurface constructed wetlands in Kathmandu. Paper V is a review paper and investigates the improvement possibilities of existing septic tank soakpit system to combination of CW and shallow infiltration systems.

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Abbreviations

${\sigma_t}^2$	Variance of the time of concentration curve, day^2
σ^2	Dimensionless variance
A	Surface area, m^2
a_s	Effective surface area, m^2/m^2
a_v	Specific area per unit volume; m^2/m^3
BOD	Biochemical oxygen demand (5 day, 20^0C)
C^*	Background concentration, g/m^3
C_e	Effluent concentration, g/m^3
C_i	Inlet concentration, g/m^3
C_n	Concentration of pollutant at fractional distance from inlet, g/m^3
COD	Chemical oxygen demand
CSTR	Completely stirred tank reactor
D	Dispersion number
DO	Dissolved oxygen
DO	Dissolved oxygen
D_p	Diameter of spherical media, m.
ET	Evapotranspiration, mm/day
e_v	Effective volume utilization, dimensionless
GSTP	Guheswori Sewage Treatment Plant
h	Water depth, m
HF	Horizontal flow
HPB	Horizontal flow planted bed
HRT	Hydraulic retention time, day
HUPB	Horizontal flow unplanted bed
k_a	Areal rate constant, m/day
k_T	Rate constant at temperature T ^0C
k_v	Volumetric rate constant, day^{-1}
MC	Moisture content
N	Number of CSTR
NH_4-N	Ammonium nitrogen
Q	Wastewater flows, m^3/day
q	Hydraulic loading rate, m/day
SLR	Sludge loading rate, kg TS/m^2/year
T	Temperature of wastewater, ^0C.
t_{actual}	Mean or actual hydraulic retention time, day
TKN	Total Kjeldahl nitrogen
t_n	Theoretical retention time, day
TP	Total phosphorus
TS	Total solids
TSS	Total suspended solids
V	Wetland volume
VF	Vertical flow
VPB	Vertical flow planted bed
VS	Volatile solids
VUPB	Vertical flow unplanted bed
η	Bed porosity
θ	Temperature correction factor (dimension less)

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Submitted to Ecological Engineering

Paper II

Manoj K. Pandey, Petter D. Jenssen, Tore Krogstad & Sven Jonasson. Comparison of vertical and horizontal flow planted and unplanted subsurface flow wetlands treating municipal wastewater.
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Manoj K. Pandey and Petter D. Jenssen. Wastewater infiltration for purification and groundwater recharge: International experience and potential in Nepal.
Submitted to Water Practice and Technology (WPT).

Abstract

As a result of rural to urban migration over the past five decades small towns are rapidly emerging particularly along the major road network in Nepal. These small towns lack resources and basic infrastructure to accommodate the rapid population growth. Decentralized wastewater management using constructed wetland (CW´s) can be a potential option for wastewater management in these small towns. There is growing interest in CW´s in Nepal but the requirement of a large area and the uncertainties in the design parameters have hindered wide spread application of the technology. There is also a lack of detailed studies regarding aspects related to the performance of constructed wetlands in sub-tropical climatic conditions. The overall objective of this research was to study constructed wetlands as part of a decentralized wastewater management scheme in Nepal and suggest some design criteria for wetland based systems.

To study the treatment performance of the horizontal flow (HF) and vertical flow (VF) wetlands pilot scale units were built. In the first phase of the experiment the hydraulic loading rate (HLR) in the beds was reduced in steps; 0.2, 0.08 and 0.04 m/d. The percent removal increased with decrease in the hydraulic loading rate for all beds and parameters except for total phosphorus. In the second phase the loading rate of 0.04 m/d was run for 7 months. In both parts of the experiment, the planted beds performed better than the unplanted beds and the VF better than the HF beds. To meet Nepalese discharge standards HF beds are sufficient, but to meet stricter requirements a combination of HF and VF is recommended.

Wetlands are robust and cheap treatment systems and thus attractive to Nepal, but how is their environmental impact compared to other treatment options? Life cycle analysis (LCA) along with cost analysis was used to investigate the environmental performance and economic sustainability of three CW based decentralized technologies and an exisiting centralized conventional secondary treatment system. The three decentralized wastewater treatment alternatives were; 1) CW, 2) CW combined with separation of urine 3) Greywater treatment in a CW combined with source separation of blackwater. The life cycle analysis does not point out one option as the best, but reveals bottlenecks and gives a basis for decision-making. CW contribute to greenhouse gas emissions, but also act as a sink for CO_2 and the net

greenhouse effect of CW´s is uncertain. The conventional system had the highest energy consumption. Transportation accounted for a significant contribution to the greenhouse gas emissions in the alternatives with separation of urine or blackwater. Sludge dewatering beds may be a poor treatment option for blackwater due to high greenhouse gas emissions. Combining constructed wetlands and urine diversion gives a net positive energy balance. The constructed wetland had the lowest total annual cost. The highest cost is for constructed wetlands combined with blackwater separation.

A short-term experiment with a pilot scale sludge drying reed bed was carried to investigate the sludge loading rate (SLR) and the drying period for sub-tropical climate as that of Kathmandu Nepal. An initial SLR of 100 kgTS/m^2/year is recommended with a gradual increase up to 250 kgTS/m^2/year. The study showed that the drying period can be substantially shorter if the beds are covered to divert precipitation.

The result of the study of hydraulic and kinetic behavior of a horizontal flow pilot- scale and a full scale constructed wetland show that larger wetlands or a longer retention time is needed to reach the maximum treatment capability. If a flow situation is achieved, where more wetland media is utilized, the wetland volume (and area) can be reduced. This should be focus of further development of horizontal flow subsurface wetlands in Nepal.

Shallow infiltration systems are suggested to upgrade soak-pit systems. Shallow infiltration utilizes the treatment capacity of the soil and maximizes distance to groundwater. Infiltration of wastewater can help mitigate groundwater depletion. Constructed wetlands and subsequent effluent infiltration will yield excellent purification as well as robust and flexible treatment systems. In order to successfully implement infiltration systems in Nepal local guidelines for site assessment and system sizing and design should be developed. There is substantial international experience regarding infiltration systems that can facilitate development of Nepalese guidelines and design criteria.

Sammendrag

Som et resultat av tiltagende flytting fra landsbygda og inn mot tettsteder de siste fem tiårene har mange småbyer vokst fram, spesielt langs hovedveinettet i Nepal. Disse små byene mangler ressurser og grunnleggende infrastruktur for å imøtekomme den raske befolkningsveksten. Desentralisert håndtering av avløpsvann ved hjelp konstruerte våtmarker (CW's) kan være en potensiell mulighet for bærekraftig håndtering av avløpsvann i disse små tettstedene. Det er økende interesse for CW's i Nepal, men fordi våtmarker krever relativt store areal, samt at gode retningslinjer for dimensjonering mangler for subtropiske områder, har utbredelse av denne tekonlogien gått langsomt. Det er også en mangel på detaljerte studier knyttet nedbrytningskinetikk og betydningen av planter i våtmarken sett opp mot hydrauliske forhold og dimensjonering som er nødvendig for å møte de nasjonale utslippskravene.. Det overordnede målet med denne oppgaven var å undersøke bruk av konstruerte våtmarker for desentralisert avløpshåndtering i byer og bymnære strøk i Nepal samt å se på kriterier for dimenjonering.

For å sammenligne rensing våtmarker med horisontal strømning (HF) under overflaten med våtmarker med vertikal strømning (VF) samt for å se på plantenes innvirkning på rensingen ble et pilotskala våtmarksanlegg bygget. Anlegget bestod av enheter med og uten planter. I den første fasen av eksperimentet ble den hydrauliske belastningen (HLR) redusert i tre trinn; 0,2, og 0,08 og 0,04 m/d. Reneevnen økte med nedgang i den hydrauliske belastningen for alle parametere unntatt fosfor. I den andre fasen av eksperimentet ble anlegget belastet med 0,04 m/d i 7 måneder. I begge deler av eksperimentet, ble det oppnådd bedre rensing i beplantete enn ubeplantete våtmarkssenger. For å møte nepalesiske utslippskrav er det tilstrekkelig tilstrekkelig, med en våtmark med horisontral strømning men for å møte strengere krav anbefales en kombinasjon av HF og VF anbefales.

Våtmark er en robust og billig rensemetode og dermed attraktiv for Nepal, men hvordan miljøpåvirkning fra en våtmark er sammenlignet med andre behandlingsalternativer.er undersøkt i en Livsløpsanalyse (LCA). Denne sammen med en kostnadsanalyse ble brukt til å undersøke miljøpåvirkning og økonomisk bærekraft av tre desentraliserte løninger, og en eksisterende sentral løsning med sekundærrensing. De tre desentraliserte alternativene var; 1) CW, 2) CW kombinert med separasjon av urin 3) gråvannsbehandling i CW kombinert med kildesortering av svartvann. Livsløpsanalysen peker ikke ut ett alternativ som best, men

avslører flaskehalser og gir et grunnlag for beslutninger. Konstruerte våtmarker (CW) bidrar til utslipp av klimagasser, men fungerer også som et sluk for CO_2 og netto drivhuseffekt av CW er usikker. Det konvensjonelle systemet hadde det høyeste energiforbruket. Transport står for et betydelig bidrag til klimagassutslippene i alternativene med separasjon av urin eller svartvann. Behandling av svartvann i plantebevokste tørkesenger kan være et dårlig alternativ behandling for svartvann på grunn av høye klimagassutslipp. CW kombinert med separasjon av urin hadde en netto positiv energibalanse. Den konstruerte våtmarken hadde den laveste årskostnaden. Den dyreste løsningen var konstruert våtmark for gråvann kombinert med kildesortering av svartvann.

En pilotstudie med avvanning av slam i plantebaserte tørkesenger er gjennomført for å studere hvilken slambelastning og hvike tørkeintervaller som kunne benyttes i sub-tropisk klima som iKathmandu. Forsøket sammen med litteraturstudier gi grunnlag for å anbefale en innledende slambelastning (SLR) av 100 kgTS / m^2 / år anbefales med en gradvis økning opp til 250 kgTS / m^2 / år. Studien viste at tørkeperioden kan reduseres vesentlig hvis anlegget er dekket for å avlede nedbør.

Kinetikken i renseprosesser og hydrauliske forhold er undersøkt i både et pilotskala og fullskala våtmarksanlegg med horisontal strømning under overflaten. Resultatene viser at kortslutningsstrømmer forekommer og at større våtmarker eller en lengre oppholdstid er nødvendig for å oppnå maksimal renseevne. Hvis en strømningforholdene kan forbedres slika at en større del av våtmarken deltar aktivt i renseprosessene kan både volum og eventuelt areal reduseres. Optimalisering av strømningsforhold bør derfor prioriters ved videre utvikling av våtmarker med horisontal strømning i Nepal.

Grunne infiltrasjonsanlegg er foreslått å oppgradere dagens mange synkekummer for avløpsvann. Grunn infiltrasjon utnytter rensekapasiteten i mer av jordprofilet, og maksimerer avstanden til grunnvannet. Infiltrasjon av avløpsvann kan bidra til å nydanne grunnvann. Rensing i konstruerte våtmarker og med sluttdisponering av utløpsvannet gjennom grunninfiltrasjon vil gi utmerket rensing og robuste anlegg. Nepal har mange områder der infiltrasjon kan benyttes, men mangler lokale retningslinjer for dimensjonering og utforming. Det finnes imidlertid en betydelig internasjonal kunnskapsbase som kan lette arbeidet med å lage dimensjoneringskriterier for Nepal.

1. Introduction

Providing safe drinking water and adequate sanitation is a major challenge for cities in the developing world. Rapid growth in urban population, unplanned and haphazard expansion of cities and rise in urban slums and squatter have increased the difficulty in providing an adequate level of urban water and sanitation. According to “global health observatory data” published in the website of the World Health Organization (WHO) global urban population is estimated to rise and the majority of the rise will occur in developing countries (http://www.who.int/gho/urban_health/situation_trends/ urban_population_growth_text/en/). A recent study on regional and global wastewater, generation, treatment and reuse shows that in developing countries only 8% of wastewater is treated (Sato et al. 2013). Up to 90 per cent of the current wastewater production flows untreated into waterways of densely populated areas or into coastal zones contributing to the growth of non-viable marine zones. Dead sea bottom already cover an area of approximately 245 000 km2 (Corcoran 2010).

As a result, the indiscriminate discharge of untreated sewage have turned urban rivers into sewers and also contaminate the ground water. There is a ongoing debate regarding centralized versus decentralized approaches to wastewater treatment among wastewater engineers and city planners (Balkema et al. 2002). Conventional or centralized systems consist of large sewage network for collection and transport of the sewage to a mechanized treatment plant often in the vicinity of the city. In a conventional system the sewer construction consumes 80% of the total project investment cost where as treatment only 20% (Grau 1996). In centralized systems high amounts of water, often potable, is necessary for transport of the waste. In sewered cities water used for flushing toilets alone consumes 20 to 40% of total the potable water supply (Gardner et al. 1997). The sludge from centralized treatment facilities can be polluted with heavy metals and other micropollutants due to discharge from industries and road runoff. At the same time the sludge is often low in elements like nitrogen, phosphorus and potassium that are valued for agricultural application (Otterpohl et al. 1997). Developing countries lack funding for construction and operation of centralized wastewater treatment system (Massoud et al. 2009). Even in the developing countries the conventional approach of wastewater management have not always satisfactorily improved the urban sanitation situation (Wright and Mundial 1997).

Cities represent a pool of nutrients where substantial amounts of plant nutrients and organic matter are present in the form of wastewater and organic household waste (Jenssen and Skjelhaugen 1994). The flow of plant nutrients into urban areas mostly come in the form of food supply from agricultural land (Borgestedt and Svanäng 2011). About 70-80% of the phosphorus exported from the agricultural sector as vegetables and animal products is passing through the sewerage systems (Swedish EPA, 1997a cited in (Hellström 1998). These nutrients are embedded in the faeces and urine that are excreted with minor contributions from the organic waste that comes from the kitchen. Currently very little of the nutrients in human excreta is recovered as exemplified by a nitrogen and phosphorus balance model study for Bangkok province. This study showed that only, 7% and 10% of N and P respectively, in the total food supply is recovered (Faerge et al. 2001).

Both nitrogen and phosphorus are fertilizers that are required for plant growth. Phosphorus is a part of the cells of all living organisms and there is no substitute or replacement. The phosphate rock from which the modern fertilizer is derived is a limited resource and is presumed to be depleted within 100 years (Barnard 2009). Without phosphorus we cannot grow plants to feed the world population. The price of phosphorus on the world market has tripled in last few years and will continue to rise in the future (Dockhorn 2009). The increasing market price is the indicator of scarcity. European fertilizer association predicts demand to exceed production in the year 2040, hence recycling of phosphorus may become crucial to sustain future high yields (Cordell et al. 2009). Annually 3 million tons of phosphorus is emitted in the form of faeces and urine and human emissions represent more than 10% of rock phosphorus production (Barnard 2009). The amount of phosphorus found in human excreta can cover approximately 28% of the worldwide phosphorus fertilizer consumption (Dockhorn 2009). In developed countries 10-20% of current fertilizer use can be supplied from wastewater and in developing countries up to 100% (FAO 2005). If all the nitrogen and phosphorous in Norwegian wastewater was reclaimed and recycled into agriculture, application of mineral fertilizer could be reduced 15% to 20% (Jenssen and Vatn 1991). The nutrient recycling will help to reduce the energy associated with the production of commercial nitrogen fertilizer. Production of the mineral fertilizers are energy intensive and is thus an important contributor to greenhouse gas emissions (Refsgaard et al. 1998).

There is an increasing demand for more sustainable wastewater management systems due to diminishing phosphorus resources, but also due to the fact that the water sector consume large amounts of energy and thus contribute substantial amounts of climate gases (www.parliament.uk/ parliamentary_offices/ post/pubs2007.cfm). There has been wide range of wastewater treatment technologies developed for nutrient removal and recovery from wastewater. Wastewater treatment can perform at very high efficiency in terms of nutrient removal and for phosphorus in particular (Ødegaard et al. 2002), but the cost, energy consumption and low degree of recycling of nutrients to agriculture have raised the question of the sustainability of traditional wastewater treatment systems.

In order to restore sustainable urban development the newly conceived green city concept is deemed to be a key for realizing sustainable urban city development (Yokohari et al. 2000). Sustainable urban wastewater management is to reduce the water consumption, increase recycling of nutrients and minimize the energy needed to do so (Kärrman 2001). Turning water challenges into opportunities for development by promoting reuse and recycling of wastewater and nutrients are elements of the emerging green city concept. A new approach of integrating urban water and sanitation with agriculture is gaining momentum (Larsen and Gujer 1997). Linkages between urban sanitation services and agriculture can close the nutrient and water loop and also give both economic and environmental benefits. Urban and rural linkage can be established through sanitation in which nutrients and organic residues from urban areas are transported to rural land areas and urban fringes. In return fresh food products are supplied to the cities. The agricultural lands in urban fringe areas often rely on the excessive use of mineral fertilizers (Raut et al. 2010), but much of this can be substituted by resources from the urban areas (Jenssen et al. 2014).

The question of carbon flow has to be considered in urban waste and wastewater management. Recycling of organic matter from wastewater and other household waste into the soil will increase the soil carbon pool (Rosso and Stenstrom 2008). Increasing the soil carbon pool through carbon sequestration will increase the binding capacity of the soil for nutrients, increase the water retention capacity and thus increases crop yields as well as contribute to offset fossil fuel (Otterpohl et al. 1997; Lal 2004).

Urban and peri-urban agriculture increases the possibility nutrient recycling because the haulage distance for urban waste products such as blackwater urine and compost is reduced (Lundin et al. 2000). For urban agriculture the decentralized wastewater management would be most appropriate because of locally available nutrients and soil amendment products (De

Bon et al. 2010).

According to Crites and Technobanoglous (1998) the definition "Decentralized wastewater management is: "the collection, treatment and disposal or reuse of wastewater at or near the point of waste generation". Because of water scarcity, energy requirements, urban growth pattern and financial and economic reasons a complete sewerage system in the growing urban cities in the developing countries may not be possible and most of these cities will have to rely on the onsite sanitation systems (Crites and Technobanoglous1998; Kone 2010).

There has been wide range of wastewater treatment technologies developed for nutrient removal and recovery from wastewater (Tchobanoglous 1991; Rose 1996; Crites and Technobanoglous 1998; Langergraber and Muellegger 2005).

Source separating technologies offer interesting possibilities for both reuse of resources and energy production (Jenssen et al. 2003). There are also emerging systems that precipitate both nitrogen and phosphorus as struvite, magnesium-ammonium-phosphate (MAP) (Le Corre et al. 2009) that are very interesting to consider if existing treatment systems are to be upgraded or new systems built. It is possible to avoid the centralized sewage in urban area through source separation of the wastewater (Otterphol et al. 1997). In cities nutrient recycling through urine separation is a promising solution that will not only prevent discharge of nitrogen and phosphorus to urban rivers it will also reduce the load on the treatment units (Kärrman 2001). It is also possible to scale down high tech centralized systems to a number of small decentralized systems so as to avoid large sewers, potential pumping and provide an opportunity for on-site water reuse and ground water recharge (Wilderer and Schreff 2000).

Small scale decentralized systems may not necessarily lead to an energy and resource saving alternative, it is the selection of appropriate technology depending on the local situation that is important (Lundin et al. 2000). There is no ideal system that is applicable in all conditions. Technology should be based on local social, economic and environmental conditions (Langergraber and Muellegger 2005).

In developing countries, because of decreasing external financial support, affordability of urban infrastructures has become an important factor when selecting technology (Sperling 1996). Constructed wetlands are gaining momentum in developing countries like China, Nepal, India (Gopal 1999; Shrestha et al. 2001a; Zhang et al. 2012). Because of its simplicity, low maintenance requirement and robust performance in both cold and warm climates it has been extensively used worldwide (Brix 1994; Haberl 1999; Jenssen et al. 2005). Pond systems has been the most common natural treatment system in developing countries (Hoffmann et al. 2011). However, there is a growing interest in constructed wetland in warm climate because subsurface flow CW´s, especially, have an advantage over pond systems as they do not encourage mosquito breeding and can be more easily integrated in urban landscape (Otterpohl et al. 1997; Hoffmann et al. 2011). The land area requirement for natural wastewater treatment systems like constructed wetland is larger than conventional systems. In big cities like Kathmandu the land price is extremely high and can hinder the application of CW´s. However, CW´s can be integrated in parks or landscaping of open green areas (Jenssen 2004). The land area and engineering required to establish the correct CW arrangement is largely related to the treatment objectives required for the system as well as to the climatic condition of the area. Social and aesthetic objectives and topography of the site available must also be considered. Constructed wetlands are widely researched and applied for as a tertiary treatment step for domestic wastewater and storm run-off in Europe, US and Australia (Brix 1994; Vymazal 1995; Reed 2001; Cooper 2009; Jenssen et al. 2010). On the other hand very little research has been done in developing countries where the technology may be most effective. The overall aim of this thesis is to study the suitability of constructed wetland based systems for use in Nepal (**Paper II**, **III** and **IV**) and comparing this to alternative systems in urban areas (**Paper I**) and also combination of wetlands with infiltration (**Paper V**). This study will also suggest design parameters for wetlands and vegetated sludge-drying beds for subtropical monsoon climate as in Kathmandu.

1.1 Current situation of wastewater managment in Nepalese cities

1.1.1 Urban river pollution

One of the major urban environmental problems in the developing countries like Nepal is the direct discharge of wastewater into the river system (Karn and Harada 2001). The rivers in Kathmandu and other urban areas have been seriously polluted by discharge of untreated industrial and domestic sewage (Shah et al. 2008). River system in Kathmandu valley is presented in Fig. 1. During the dry season, particularly when there is no rainfall (March, April), the flow in the Bagmati river passing through urban and semi-urban areas is mainly carrying wastewater. The water quality of the Bagmati river is presented in Table 1. The sampling location is at Sundarighat in the downstream part of the Kathmandu valley. The wastewater quality is virtually comparable to the domestic sewage.

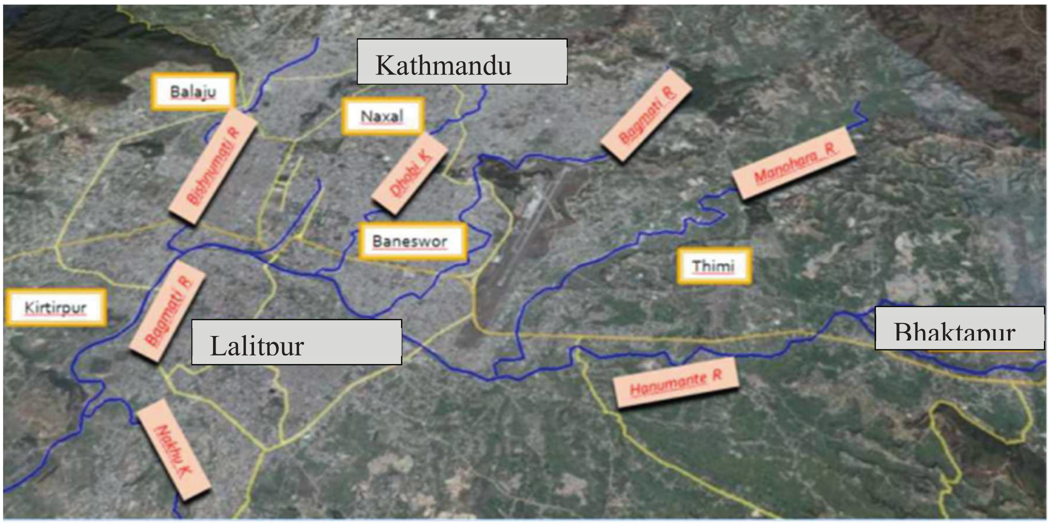


Figure 1: Bagmati and its tributaries (*Source ADB 2009*)

Table 1: Water Quality (mg/l) and flow rate of Bagmati river (m^3/s) at Sundarighat measured during the dry season of 1999 by three different consulting companies (PMC 2000).

	Flow rate m^3/s	Turbidity mg/l	TSS mg/l	Chloride mg/l	Total Phosphate mg/l	TKN mg/l	NH_3-N mg/l	BOD mg/l	COD mg/l	DO mg/l
Ness June, 1999	3.23	90	171	51	10	25	18	68	208	0
CEMAT Water Lab April, 1999	1.01	178	215	29	6	-	64	105	120	1.7
Soil T. Lab, March, 1999	0.935	113	-	84	-	55	-	140	143	0

The number of wastewater outfalls and corresponding flow of sewage into the river corridor are presented in Table 2. In the last decade the construction of new sewers in the urban Kathmandu has taken place at a rapid pace and therefore the current number of drains might be higher than presented in Table 2. When new sewers are constructed the septic tanks or septic tank/soakpit system that provided some treatment is bypassed.

Table 2: Number of outfalls from sanitary drains into the Bagmati river system and corresponding discharge volumes in million liters per day (MLD) (Pandey et al. 2006)

River Corridor	Length (Km)	Right Bank* Sewer outfalls (No.)	Discharge (MLD)	Left Bank* Sewer outfalls (No.)	Discharge (MLD)
Bagmati	35.0	22	11.6	19	7.5
Dhobi Khola	16.0	34	5.7	34	6.4
Bishnumati	14.3	11	1.7	9	5.9
Total	65.3	67	19.0	62	19.9

The majority of the buildings in urban Kathmandu are now connected to sewers that discharge the untreated wastewater directly to the Bagmati river or one of its tributaries.

One possible option to improve the environmental quality of the whole Bagmati river system, is to intercept and treat the incoming wastewater by constructing decentralized treatment facilities at the outfalls or other appropriate points. The other alternative is to construct large intercepting sewers collecting the outfalls to a few large treatment plants and this has ben done for some stretches along the Bagmati.

Such constructed treatment units built along the river corridor are expected to be cost effective due to low investment and operational cost, equal or better performance than the exsisting conventional secondary treatment system, and less need to invest in large collecting sewers.

1.1.2 Wastewater management

In the Kathmandu valley septic tanks followed by a soak pit is the most common method of wastewater treatment in urban and periurban areas where the sewer lines do not exist (HPCIDBC 2011 and Table 3).

Table 3: Number of households connected to different wastewater treatment and disposal options in urban and periurban areas of the Kathmandu valley (Metcalf and Eddy 2000).

Disposal of household wastewater to	Kathmandu		Lalitpur		Bhaktapur	
	No.	%	No.	%	No.	%
Septic Tank	24695	56.2	10799	58.4	4237	49.2
Community Sewer	8015	18.2	2863	15.5	2607	30.3
Open Sewer	862	2.0	217	1.2	83	0.9
River	908	2.1	58	0.3	42	0.5
Courtyard	915	2.1	46	0.2	358	4.2
Pit Latrine	560	1.3	266	1.4	52	0.6
Soak Pit	500	1.1	89	0.5	78	0.9
Road	719	1.6	22	0.1	23	0.3
No response	6752	15.4	4143	22.4	1134	13.1
Total	43926	100.0	18503	100.0	8614	100.0

A report published by the UN-Habitat (2008) estimate that about 30% of the houses have a septic system. Only 35% have a soak-pit associated with the septic tank. The remaining tanks presumably discharge septic tank effluent direct to surface flows.

Though the septic tank is commonly used, the cleaning of the tank is not carried out as frequently as is required. The functioning of the tanks are, thus, not efficient.

Metcalf and Eddy (2000), through a consumer survey, estimated the number of septic tanks in the municipalities of Kathmandu, Lalitpur and Bhaktapur to be 33,000, 8,400, and 2,300 respectively with a total of 43,700. It is estimated to increase to 77,700 by the year 2021.

The experience regarding operation and maintenance of the centralized treatment facilities Kathmandu valley are not satisfactory. Out of five-wastewater treatment plants only one is operating. The four others have been out of operation for decades. The reasons for the failure of these plants are high cost of operation and maintenance, lack of maintenance of sewers and failure of pumping stations (ADB 2009). Table 4 presents the operational status and the projected wastewater flow in 2021. The present design capacity of the existing wastewater treatment plants (Table 4) is far less than that required to serve the existing urban population within the Kathmandu Valley. ADB (2009) has estimated that if the existing wastewater treatment plants are rehabilitated only 22% of the wastewater collected could be treated. The current plants are all secondary systems and has low nutrient removal. In order to avoid eutrophication of the Bagmati and its tributaries nutrient removal is necessary (Ashley et al. 1999). This shows the urgent need of addressing the wastewater problem in Kathmandu.

Table 4: Operational status of existing treatment plants, main treatment process, current population and wastewater flow and projected population and the wastewater flow for the year 2021 (PMC 2000)

Name of WWTP	Year of Const.	Treatment Process	Design Capacity		Operational Status	2021 pop. in Catchment '000	2021 Flow in Catchment MLd
			MLd	Pop '000			
Kathmandu and Lalitpur Metropolitan Area						1470	335
Dhobighat	1978	Oxidation Pond	15.4	160	Not working	346	79
Kodku	1978	Oxidation Pond	1.1	40	Working at low efficient.	106	24
Guhyeshwori	1999	Activated Sludge Oxidation ditch	17.3	198	In operation	198	45
Bhaktapur Municipal Area						132	30
Sallaghari	1983	Aerated Lagoon	2.0	?	Yes	?	?
Hanumanghat	1977	Oxidation Pond	0.5	?	Yes	?	?

The first CW in Nepal was constructed at Dhulikhel Hospital in 1997 (Shrestha et al. 2001a). Since then more than dozen of CW´s have been constructed at different places most of them for institutions like hospitals, schools etc. The design flow in all these systems is below 40 m^3/day. Only a limited number of studies are published in international journals regarding the performance of wetlands in Nepal (Laber et al. 1999; Shrestha et al. 2001b; Singh et al. 2009). However, there are number of unpublished documents in the form of thesis and consultant reports.

1.1.3 Agricultural land and potential for resource recovery

Table 5 shows the population, agricultural land use and water demand in the Kathmandu Valley. About 50 % of land in the valley is arable. This shows that Kathmandu Valley, in spite of rapid urbanization and population growth, is still rural and green. About one third of the arable land is irrigated; the rest of the land depends on the monsoon rainfall.

Table 5. Population, water demand for human consumption, wastewater generation total land area, arable land, and park and green areas in the Kathmandu Valley (NTC 2009)

Zone/Physical setting	Population	Water demand (m^3/day)	Wastewater generation (m^3/day)	Total Land area (km^2)	Arable land (km^2)	Park and greeneries (km^2)
Rural	282056	12692	10788	473	235	213.
Peri-urban	283499	28349	24097	113	87	8.
Urban	995966	109556	93122	100	30	6
Total	1561521	150598	128008	686	353	228

Therefore, there is a huge potential of the nutrient recycling from urban wastewater in Kathmandu valley. Based on literature values (Vinnerås 2002; Otterpohl et al. 2003; Mattila 2003) of nutrient production per person from urine and faeces, in the form of nitrogen, phosphorus and potassium the annual nutrient production from the Kathmandu valley population is estimated (Table 6).

Table 6: Faeces, urine and corresponding nutrients (nitrogen and phosphorus) from the current population in Kathmandu valley.

Built up environment	Population	Faeces (m^3/year)	Urine (m^3/year)	Nitrogen ton/year	Phosphorous (ton/year)	Pottassium (ton/year)
Rural	282056	28826	164720	1128	211	507
Peri-urban	283499	28973	165563	1133	212	510
Urban	995966	101787	581644	3983	746	1792
Total	1561521	159587	911928	6246	1171	2810

Using a nitrogen application of 100 kg N/hectare the nutrients from the valley population can fertilize 624 km^2. This is nearly double the area of cultivated land in the valley. N-application in agriculture range from 50 – 150 kg/ha.

This calculation shows that if all excreta from the Kathmandu valley is reclaimed and recycled into agriculture, it could substitute all fertilizer application in the valley and in addition export fertilizer to other regions. Production of mineral fertilizer is energy intensive and contributes large amounts of greenhouse gases (Refsgaard et al. 1998). Nutrient recycling will help to reduce the energy associated with the production of commercial nitrogen fertilizer and, thus, improve the sustainability of the wastewater handling. Taking out nitrogen and phosphours from wastewater stream will also prevent the eutrophication of the rivers.

1.2 Sustainability analysis of wastewater treatment

Wastewater treatment systems are often selected based on simple cost benefit analysis. Cost benefit analysis emphasizes the technical and economic viability of the system but overlooks the long term sustainability. Over the last decades sustainable wastewater treatment has been an issue at several conferences (Kløwe et al. 1999; Werner et al. 2009). In order to develop sustainable wastewater treatment it is necessary to view the wastewater treatment systems using a holistic approach (Jenssen 1996). The technical solution has to match goals for treatment performance and resource recovery with a minimum of environmental impact. A holistic approach implies considering both the primary and secondary environmental effects and costs that the systems produce.

A number of sustainability indicators have been developed incorporating environmental, social and economic sustainability (Balkema et al. 2002; Lundin and Morrison 2002; Muga and Mihelcic 2008). Choice of the indicators depends upon the importance of the respective indicators in the local and regional context. Hence, when determining the sustainability of wastewater treatment systems the energy use is an important indicator in supplement to the pollutant load, the investment, operation and maintenance cost. The green-house gas emissions, often associated to energy use, and nutrient recycling are important issues regionally and globally. Thus a multidisciplinary approach is needed to determine the sustainability of a wastewater treatment system and both the primary and secondary effects and costs that the systems accrue should be accounted for. Examples of secondary effects are the pollution produced at the power plant generating electricity for wastewater treatment and the energy cost of producing treatment chemicals (Venkatesh and Brattebø 2011).

In this study life cycle analysis (LCA) was chosen because it has been found useful to determine the environmental impact of water supply and wastewater treatment facilities (Lundin et al. 2000; Machado 2006; Ortiz et al. 2007; Renou 2008). The components of a LCA framework is presented in Fig. 2.

The goal definition stage involves formulation of what should be investigated and how the investigation is to be carried out. The inventory analysis form the core of a LCA and is the most time consuming activity (Charlton et al. 1992). To build up the inventories (environmental inputs and outputs), the life cycle of product or a system is first divided into phases. The major life cycle phases examined are: construction, operation and demolition phases (UNEP/SETAC 2011). In this study only construction and operation phase have been included. In the impact assessment stage the results of the inventory analysis are interpreted in terms of the impact they have on the environment. In impact assessment, the analysed data is grouped or classified, according to the particular impact on the environment of each individual component in the inventory. Impact analysis in LCA includes impact classification and characterization, normalization and valuation of impacts (UNEP/SETAC 2011). In this study only impact classification and characterization has been carried out. The improvement assessment may result in changes in product design, raw material use, industrial processing, consumer use and waste management (Charlton et al. 1992).

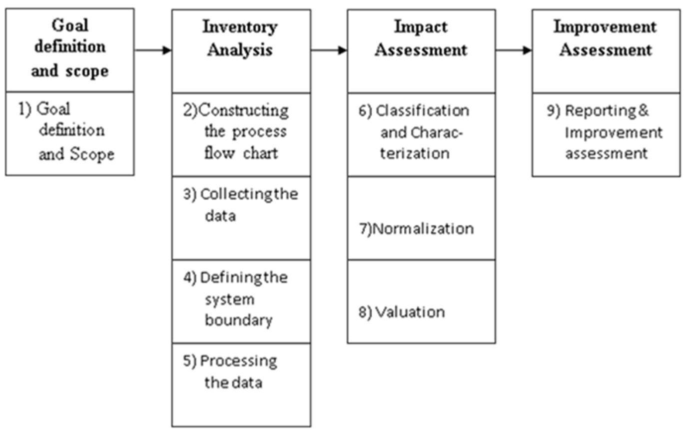


Figure 2: Components of a LCA framework

There are number of studies that have applied the LCA tool to wastewater treatment work

(Emmerson et al. 1995, Tillman et al. 1998; Lundin et al. 2000, Ortiz et al. 2007). However, there are only few studies that have applied LCA to compare centralized versus decentralized alternatives (Dixon et al. 2003; Unger et al. 2004; Benetto et al. 2009).

A number of software packages are developed for LCA analysis (Jonbrink 2000). In this study LCA analysis software SimaPro 7 is used. SimaPro 7 has been developed by Pre Consultants, Amersfoort, the Netherlands. The main features of the program are summarized below (Pré Consultants 2010):

- The program can be used for detailed life cycle analysis and for comparing two or more competitive products
- The program can determine the material or processes that have dominant influences on the product's total environmental impact.
- It can be used as a tool for decision making
- Allows results displayed in both graphical and tabular forms
- Contain a huge database on products and processes

1.3 Constructed wetland: An overview

Constructed wetlands are broadly classified as surface flow and subsurface flow (Brix 1994). In the surface flow CW the water level is above the surface of the bed. Subsurface flow CW´s are designed to maintain the water below the media surface. In warmer climates subsurface flow is preferred because of the possibility of less odor and vector problems. The subsurface flow CW's are further divided into horizontal flow subsurface wetlands and vertical flow subsurface wetlands depending on the direction of the wastewater moving through the bed (Brix 1994). HF wetlands and surface flow wetlands were the first types constructed in Europe and America (Kadlec 2009; Vymazal 2005; Cooper 2009). HF wetlands have high biochemical oxygen demand (BOD), chemical oxygen demand (COD) and total suspended solid (TSS) removal efficiency, but nitrification is low due to limited oxygen supply (Vymazal 2005; Vymazal 2009; Cooper 2009). HF beds have both aerobic and anaerobic zones therefore nitrification and denitrification occurs (Reed and Brown 1995). In HF wetlands the plants alone cannot provide sufficient oxygen to sustain both carboneous oxidation and nitrification therefore nitrification is limited (Kadllec and Wallace 2009). In order to enhance the phosphorus removal in the CW based treatment systems the HF beds are usually packed with high phosphrous sorption capacity porous media (Jenssen et al. 2005). HF wetlands are continuous or batch fed (Kadlec and Wallace 2009).

VF beds have good performance for the removal of BOD, COD, TSS and ammonium-nitrogen (NH_4-N), but have a low denitrification potential (Vymazal 2009). In VF wetlands intermittent dosing draws atmospheric air into the unsaturated pore system of the bed. Therefore HF beds have greater O_2 availability and a higher nitrification potential than HF beds (Hunter et al 2001; Vymazal 2009, Cooper 2009). In VF beds total nitrogen removal is limited (Vymazal 2009). VF constructed wetlands requires less area than HF, but require more operation and maintenance (Kadlec and Wallace 2009).

A combination of a HF constructed wetland and a VF constructed wetland, normally refered to as a hybrid wetland, is able to achieve substantial reduction of both organic matter as well as total nitrogen (Vymazal 2005; Saeed and Sun 2011). Different configuration and combination of hybrid wetlands have been used to enhance nutrient removal in CW systems (Hunter et al 2001; Vymazal 2009). The most common is VF bed followed by HF bed (Kadlec and Wallace 2009). In Nepal a HF bed followed by a VF bed is used (Laber et al. 1997). HF upfront removes organics and suspended solids and prevents clogging (Haberl 1999). Nitrified effluent from VF bed are recycled back to HF bed thus improving the total nitrogen removal in the system (Kadlec and Wallace 2009). A VF wetland at the front end of the system produce nitrified effluent which is subsequently denitrified in HF bed (Cooper 1999). Natural systems like ponds can be combined with constructed wetlands and enhance the overall removal including organic matter, nutrients and bacteria (Browne and Jenssen 2005).

1.3.1 Pollutant removal processes and the effect of climate

Pollutant removal in the constructed wetland occurs as a result of complex interactions between water, wetland media and wetland vegetation (Kadlec and Wallace 2009). The major processes involved in the removal of pollutants are given in Table 7. The plant and media together play a major role in the physical and microbial process of pollutant removal in the wetland (Brix 1997). Comparison of planted and unplanted beds (both full scale and pilot scale) have shown that plants play a major role in organic matter and nutrient removal (Yang et al. 2001; Hunter et al. 2001; Huett et al. 2005; Vymazal and Kropelova 2009; Kadlec and Wallace 2009). In planted beds the root network help to enhance the sedimentation and filtration process by slowing down the flow through wetland (Karathanasis et al. 2003).

Table 7: Pollutant removal processes for key pollutants in subsurface flow constructed wetlands.

Pollutant	Removal Processes	References
Suspended solids	Sedimentation and filtration	Kadlec and Wallace (2009)
Organic matter (BOD)	Biological degradation, sedimentation, microbial uptake	Kadlec and Wallace (2009)
Nitrogen	Ammonia volatilization, nitrification, denitrification, nitrogen fixation, plant and microbial uptake, mineralization, sorption and accretion.	Hunter et al. (2001); Huett et al. (2005); Vymazal (2007)
Phosphorus	Sorption, precipitation, plant uptake	Vymazal (2007); Adam et. al. (2007); Jenssen et al. (2010)
Pathogens	Settling, straining, sorption and predation by protozoa	Stottmeister et al. (2003); Jenssen et al. (2005); Heistad et al. (2006)

The transformation of organic matter and nitrogen in subsurface flow constructed wetlands are mainly due to the activity of bacteria and other microorganisms (Khatiwada and Polprasert 1999; Lim et al. 2001; Wynn 2001, Kadlec and Wallace 2009). Plant roots and the wetland porous media provide a large surface area for biofilm growth (Kadlec and Knight 1996; Brix 1997; Vymazal et al 1998; Person et al. 1999; Khatiwada and Polprasert 1999; Lim et al. 2001). The area of an operating CW that is occupied by biofilm is defined as the effective surface area (a_s) (m^2/m^2).

The effective surface area (a_s) of a bed can be rougly estimated (Eq. 1) (Kadlec and Knight 1996)

$a_s=6\,(1-\eta)/D_p$…………..(1)

Where, η = bed porosity and D_p= diameter of spherical media, m.

The specific area per unit volume (a_v) is "a_s/h" (m^2/m^3), where h = effective depth of bed or the water depth.

Khatiwada and Polprasert (1999) estimated a theoretical value of a_s and a_v 24000 m^2/m^2 and 16800 m^2/m^3 respectively for a planted beds.

The removal rate of the pollutants in the wetland is a function of the (a_s) (Kadlec and Knight 1996; Khatiwada and Polprasert 1999).

The availability of a large effective surface area is important in nitrogen and organic matter removal (Kadlec 1999). However, the contact between root zone and the wastewater should be maximized inorder to optimize the microbial mediated processes (Breen and Chick 1995). Poor hydraulic efficiency can cause the underutilization of the available biofilm surface area in the bed. Hydraulic efficiency can be improved for instance by proper inlet and outlet geometry and proper packing of the beds to avoid inhomogeneity (Suliman et al. 2005) and avoiding complex bed geometry to prevent the dead or stagnant zone (Kadlec and Wallace 2009). The microbial mediated processes are temperature sensitive (Kadlec 1999).

In warmer climates where suitable climatic conditions for plant growth prevail throughout the year treatment performance of the CW beds is expected to be higher (Breen and Chick 1995; Billore et al. 1999). In colder climate where plant growth is hindered by cold winters the contribution of plants to treatment performance were found not to be very positive (Mæhlum and Stålnacke 1999).

In cold climate with an extended period of colder months during which plants are dormant the role of plant to supply O_2 and initial biodegradation process is limited (Jenssen et al. 1993, Mæhlum and Stålnacke 1999). Unplanted constructed wetlands have been used in cold climate (Heistad et al. 2006, Jenssen et al. 2010). The organic matter and nutrient removal in such systems is enhanced by installing aerobic biofilters with sprinkle dosing followed by unplanted horizontal flow beds packed with commercially available media with high porosity, good hydraulic conductivity and high P-sorbing capacity (Heisted et al. 2006, Jenssen et al. 2010).

Plant uptake of nutrients may not be significant in cold climate (Jenssen et al. 1993), but in warmer climates plant growth is more rapid and frequent harvesting of plants will increase the rate of nutrient removal by plant uptake contributing to the overall removal of nutrients from bed (Brix 1997; Yang et al. 2001).

Evapotranspiration (ET) and rainfall have an effect on water mass balance in the wetland and thus influence the treatment efficiency (Kadlec 1999). ET increases the nominal retention time and concentrates the pollutants in the water (Kadlec and Wallace 2009). Increase in the hydraulic retention time may either provide modified removal rates, which can partially offset or enhance the concentration effects of ET (Kadlec and Wallace 2009). In moderate temperate climate ET lossess are on the order of 0.5 m/year, and will double in hot and arid climate (Kadlec and Wallace 2009). In cold temperate climate, the effect of ET is only in the growing season. In tropical and subtropical climates shorter retention time and shallow depth produced BOD reduction of 80 to 90% and effluent below 20 mg/l (Kantawanichkul and Wannasri 2013). At warmer temperature the surface area of the CW can be reduced (Langergraber et al. 2007).

In warmer climate the kinetic processes for organic matter and nutrient removal are faster and therefore both the kinetic and hydraulic parameters can be optimized. However, unlike in cold climate there are limited studies on constructed wetland in warm climatic conditions (Gopal 1999; Shrestha et al. 2001a; Kantawanichkul and Wannasri 2013).

1.3.2 Constructed wetland design models and uncertainties in design parameters

CW as an alternative treatment system in cities may not be feasible because of the requirement of a large area. Therefore design methods and desing parameter should be carefully selected. Different methods have been used for sizing of wetlands. Studies on a large number of CW's operating in cold and temperate climates have optimized the design parameters (Reed 1993; Brix & Arias 2005; Cooper 2009; Kadlec 2009; Kadlec and Wallace 2009; Vymazal 2011). However, unlike in temperate and cold climate the number of studies regarding constructed wetlands in warmer climate are limited (Gopal 1999; Shrestha et al. 2001; Kantawanichkul & Wannasri 2013) and there is a need for further assessment of design parameters for warm climates. Design guidelines in Europe and America (Reed 1993; Brix and Arias 2005), especially for single households, are based on "crude rules of thumb" (Rousseau et. al. 2004; Cooper 2009). These rules often express design in area per person and may give oversized systems particularly if these rules, derived in cold climate, are used in warmer areas. For urban settings where land prices are high and land availability is limited design criteria tailored to the local situation are required to produce an optimum system. In absence of prescriptive criteria, first order reaction kinetics and plug flow hydraulics is the common approach of designing wetland (Kadlec 1994; Kadlec and Knight 1996; Wynn and Liehr 2001).

Kadlec and Knight (1996) proposed a modified first order plug flow model; commonly referred to as k-C^* model, for the design of constructed wetlands. The k-C^* model can be expressed either in terms of volumetric rate coefficient (k_v), day^{-1} (Eq. 2) or in terms of areal rate coefficient (k_a), m/day (Eq. 3)

$(C_i-C^*)/(C_e-C^*)=e^{-k_v t_n}$ ……………….(2)

$(C_i-C^*)/(C_n-C^*)=e^{-k_a*y/q}$……… …….(3)

Where, C_i = inlet concentration, g/m^3; C_e = effluent concentration, g/m^3; C_n= concentration at a fractional distance "y" from inlet (at y =1, $C_n = C_e$), g/m^3; C^* = background concentration, g/m^3; t_n = nominal or theoretical hydraulic retention time (HRT), day; q = hydraulic loading rate (HLR), m/day. The theoretical hydraulic retention time, t_n, can be calculated by using equation (4).

$t_n=V\eta/Q$ ……………………….……(4)

Where, V= wetland volume, m^3, η = porosity of the bed (in fraction), Q = wastewater flow, m^3/day. The hydrualic loading rate q (m/day) can be calculated by Equation (5).

q=Q/A…………… ……………….(5)

Where, A = surface area of wetland, m^2. The relation between k_v and k_a are given by equation 6.

$k_a=k_v*h*\eta$ …… …………………(6)

Where, η = porosity, in percent. For model calibration (k_a) is used as it does not require the depth and porosity (Kadlec 2009). Both depth and porosity are difficult to know to a reasonable degree of accuracy.

For known (q), equation 3 can be used to predict the concentration profile along the horizontal flow wetland bed.

The (k-C*) model (Kadlec 2000) takes into consideration that a wetland, as a natural dynamic system, produces and discharges some organic matter and nutrients that is not directly derived from wastewater. Thus, a non-zero background effluent concentration (C*) is introduced. C* is normally unknown and therefore used as a free fitting parameter when calibrating the models.

The rate constant (k_a) is temperature sensitive and therefore, the temperature effect on (k_a) is expressed as (Kadlec & Wallace 2009):

$k_T = k_{20}\, \theta^{(T-20)}$ ....(7)

Where, k_T is areal rate constant at temperature T (^oC) and θ is the temperature correction factor (dimension less).

The rate coefficient is sensitive to θ (Kadlec 2009). A small change in value of θ will bring a large change in value of rate coefficient (Kadlec and Wallace 2009). Different values of θ have been suggested in the literature (Crites and Tchonoblagos 1998, Kadlec 2009). Based on the study of 30 wetlands Kadlec (2009) suggested a value of 1.06.

Tracer studies to provide hydrodynamic characteristics of wetlands have shown that the constructed wetlands are best represented by non-ideal flow pattern between plug flow and completely mixed reactor (Kadlec 2009). Several reasons are reported for the non-ideal flow pattern (Batchelor and Loots 1997; Person et al. 1999; Wynn et al. 2001; Whitney et al. 2003; Garcia et al 2004; Suliman et al. 2005; Headly and Kadlec 2007, Kadlec 2009). Irregular wetland shape is prone to develop stagnant pockets (dead zone) in the wetland (Headly and Kadlec 2007). Preferential flow paths may occur through the lower sections of gravel beds where the roots have not reached. Clogging of the wetland, inlet and outlet arrangements and inhomogeneities in the porous media may cause preferential flow (Suliman et al. 2005). Non-ideal flow tends to result in poorer pollutant reduction performance in comparison to the ideal plug flow situation (Headly and Kadlec 2007). Near plug flow and effective volume uitlization condtions are necessary to promote good hydraulic efficiency (Person et al. 1999). The effective volume (e_v) utilized by the constructed wetland can be determined by a tracer test using equation.

$$e_v = \frac{t_{actual}}{t_n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Where, t_{actual} = mean or actual hydraulic retention time, in days, obtained from tracer study. The degree of non-ideal flow conditions within is defined by the dispersion number (D). A dispersion number of zero indicates ideal plug flow conditions and as the dispersion number approaches infinity a completely mixed reactor is approached (Person et al 1999). The dispersion number can be calculated from the tracer study data using a closed-vessel equation suggested by Levenspiel (2012).

Although, first order models commonly assume plug flow, Kadlec (2000) found the non-ideal flow pattern, that is normally the case in a CW, could be better described by a tank-in-series model, commonly refered to as p-k-C*. The p-k-C* model is also based on the simple first order reaction kinetics, but this model includes a number of completely stirred tank reactors (CSTR) in series. The number of CSTR (N) can be determined from tracer studies (Kadlec and Knight 1996) using equation (9).

$$N = \frac{\sigma^2}{{t_{actual}}^2} \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Where, σ^2 is dimensionless variance. The p-k-C* model best represented the tracer response curve of several constructed wetland system examined (Kadlect and Wallace 2009). Low value of (N) indicates short-circuiting of flow in the wetland (Kadlec and Wallace 2009). Kadlec (2009) found the wetland best represented by three numbers of CSTR when p-k-C* model was fitted to the tracer curve of 30 examined constructed wetland. Batchelor and Loots (1997) found the p-k-C* model with 11 equal size tank to best fit the experimental tracer curve. The wetland outlet concentration in p-k-C* model can be predicted using equation (10).

$$\frac{C_e - C^*}{C_i - C^*} = \frac{1}{\left(1 + \frac{k_a}{Nq}\right)N} \ldots\ldots\ldots\ldots (10)$$

The modified plug flow model k-C* and the p-k-C* model consider CW as a black box and therefore the internal pollutant removal kinetics are unknown (Kadlec and Wallace 2009). The mechanistic compartmental model such as developed by Wynn et al. (2001) and

Langergraber et al. (2007) give insight to the intrinsic processes in the wetland system but several assumption and empirical relations make it difficult to use for general design purposes (Rousseau et al. 2004).

Mathematical modelling such as dynamic compartmental modelling may be helpful for comparing the different design alternatives prior to construction (Cooper 2009). Rousseau et al. (2004) has highlighted the model constraints and the uncertainty in different design models used in constructed wetlands. Because of the parameter uncertainty, the predicted result could vary within the same model category.

Prediction of effluent concentrations using plug flow may not be reliable because the slightest deviation from the ideal plug flow pattern will increase the effluent concentration (Kadlec and Wallace 2009). Non-ideal mixing can cause large errors in the rate constant estimation and performance prediction (Headly and Kadlec 2007). In adittion, the rate constant stochastically vary with time (Kadlec and Wallace 2009). As plants mature the root density increases and the plant-mediated processes will be more effective (Breen and Chick 1995). The rate constant has been found to vary with time. Short-term (months) observations by Bista et al. (2004) indicated an increase as the system matures. Brix et al. (1998) found an increase as the systems grew older, but Vymazal (2011) did not see this in long-term (10 years) study.

2. Study rationale and objectives

2.1 Study rationale

The conventional wastewater collection systems for water-borne sewerage and corresponding treatment systems needs huge capital investments, consume large amounts of energy and may not be the most feasible way to solve all water pollution problems (Gallego et al. 2008). Experience has shown that conventional centralized systems are expensive and difficult to operate and maintain in developing countries like Nepal (NTC, 2009). Therefore the sustainability of the centralized wastewater management approach is questioned (**Paper I**).

Sustainability of the wastewater treatment system should be examined through a multidisciplinary set of indicators that encompass environmental, technical, economic and social aspects (Jenssen 1996). In addition to cost and treatment performance energy aspects, recycling and social issues are important when evaluating the sustainability of a wastewater treatment system and selecting an appropriate system for a given condition (Jenssen et al. 2007). For developing countries, cost is an important indicator for the sustainability of wastewater treatment systems. The environmental impact of the technology from local, regional and even global point of view is important to investigate when the selecting technology. Life cycle analysis has proven to be a useful tool to assess the environmental impact of alternative wastewater treatment technologies (Balkema et al. 2002; Bisinella et al. 2014) **(Paper I).**

In recent years, there has been an increased interest in decentralized treatment of urban wastewater using natural treatment systems as constructed wetlands (Jenssen snd Vråle 2003; Parkinson and Tayler 2003). In Nepal it has been successfully applied to treat hospital, institutional, and community wastewater (Laber et al. 1997; Shrestha et al. 2001a).

The design of the CW´s in developing countries like Nepal is mostly based on the empirical findings from other countries (Shrestha et al. 2001b). There are few studies examining constructed wetlands in the subtropical climatic of Nepal (Laber et al.1997; Shrestha et al. 2001a; Singh et al. 2009) and design parameters suited to the climatic condition need further assessment. The current systems are often oversized because of an excessive factor of safety. It addition there is a need to investigate the role of plants in CW´s as input to develop an improved design rationale **(Paper II, III and IV).**

A major issue in adopting the constructed wetland technology is the choice of the wetland type. Performance studies of HF bed and VF beds have shown that while both are good in organic matter and TSS removal, nitrification was lower in HF beds (Vymazal 2013). However, comparative performance studies of HF bed and VF beds in subtropical monsoon climate under similar conditions have not yet been done. HF beds are more common in developing countries as they are easier to design and construct and normally do not require pump or dosing device to feed the wastewater (Gopal 1999). But, the horizontal bed alone cannot remove the ammonia unless a very large area is provided (Platzer 1999; Noorvee et al. 2005). The HF-VF combination gives excellent secondary treatment but it does not necessarily remove nutrients (phosphorus and nitrogen) and thus reduce the eutrophication potential. Therefore, the possibilities of using HF beds combined with the source separating options to improve nutrient removal are included in the analysis in **Paper I.** Eutrophication is also one impact category chosen for the comparative analysis of the alternative systems (**Paper I**). The performance of the HF and VF beds is compared in **Paper II.**

The rate constant (k) is an important parameter when CW systems are designed using kinetic models such as the commonly used first order model. Rate constants, however, vary and are dependent on ambient environment conditions (Kadlec 1999). An assessment of the removal kinetics and design models, with respect to the local climate in Nepal is therefore included (**Paper IV**).

One challenge of decentralized wastewater treatment systems is sludge handling and also the utilization of the accumulated sludge. In urban and peri-urban areas of developing countries onsite treatment using septic tanks is a common method of wastewater management. However, desludging is not performed when needed and the removed sludge is disposed off to drain or rivers or open spaces (HPCIDBC 2011). Despite the increase in septic tanks in the Kathmandu valley, the urban river pollution problem has not been solved but have increased. This is because of the lack of management and utilization of the sludge generated in the septic tanks (Shrestha et al. 2001a), but also the fact that a septic tank has a low treatment performance for BOD and nutrients in particular (Tchobanoglous 1991).

Constructed wetlands and sludge drying reed beds can be integrated to treat both wastewater and the sludge. Sludge drying reed beds require more area than most other sludge treatment options, but are more cost efficient due to low operational cost and no need to design the main treatment system for the return flow from the sludge treatment (Nielsen 2003). The loading rate and the operational strategy has been well established for temperate climate particularly based on the studies of Nielsen (2007). Koottatep et al. (2005) have given general design guidelines for tropical climate. In addition to large area requirements the challenges of using sludge drying reed beds are; long startup time due to conditioning of the reeds; sensitivity to the loading regime; wilting of plants, and limited experience and lack of design criteria for different sludge types and climate zones (Kim and Smith 1997; Koottatep et al.2005; Nielsen and Willoughby 2005). There are few studies that compare the planted and unplanted bed under similar operating conditions (Lienard et al. 1995; Edwards et al. 2001; Paing and Voisin 2005; Nassar et al. 2006). Sludge drying reed beds still requires large input and field research, development and testing before they may be propogated as a state of art option

(**Paper III**).

In Nepal septic tank-soakpit systems are the common method of wastewater treatment and disposal. Because the use of soak pits often overload the treatment capacity of the soil ground water contamination from the septic tank-soakpit systems is common. In addition the total annual abstraction of ground water in Kathmandu valley exceeds the recharge. Substituting the soakpits for properly designed wastewater infiltration systems can both reduce groundwater pollution and increase recharge of the groundwater reserve (**Paper V**).

2.2 Objectives

The overall objective of this research was to study constructed wetlands as part of a decentralized wastewater management scheme in Nepal and provide data to better understand performace of constructed wetlands in sub-tropical climate as basis for improved design.

The specific objectives were:

- Design and construction of pilot scale constructed wetland units comprising horizontal and vertical flow beds for the treatment of municipal wastewater.
- Evaluate and compare the performance of horizontal and vertical flow wetland units for treatment efficiency and the role of plants while operating under different hydraulic loading rates.
- Examine the oxidation/reduction potential (ORP) in the pilot constructed wetland
- Study the hydrodynamic behavior and removal kinetics of a horizontal flow constructed wetland.
- Compare environmental performance and cost of decentralized constructed wetland based treatment systems to conventional centralized sewerage and treatment.
- Evaluate sludge drying reed beds as a potential sludge treatment method in subtropical climate.
- Assess the potential for use of soil infiltration as a final treatment and disposal method for domestic wastewater in Nepal.

3. Material and methods

3.1 Site for experimental setup

The pilot systems were constructed at the premises of Guheswori Sewage Treatment Plant (GSTP), which is operated by the government. The existing wastewater treatment facility is an extended aerated lagoon system. The treatment plant was constructed to intercept and treat the wastewater discharged along a 11.5 km stretch of Bagmati River between Gokarna and Pashupati.

This site was selected due to the following reasons:

- Land available for the construction of experimental setup.
- Sand and gravel for the beds were available close to the site.
- Availability of sufficient raw wastewater required to feed the experimental units.
- GSTP laboratory facilities were available to perform the experiments.
- Wastewater could be drawn into the horizontal beds by gravity,

3.2 Experimental units

3.2.1 Pilot scale CW

The pilot scale subsurface flow CW system consists of two units of HF beds and two units of VF beds, each having surface area of (6 m x 2 m=12 m^2) (Fig. 3). The length of the HF unit is 3 times the unit width to promote plug flow conditions.

The depth of the horizontal HF and VF beds were both 0.6 m. The effective grain size (d_{10}) and uniformity coefficient (d_{60}/d_{10}) of the media was determined by sieve analysis (Table 8). The porosity of both the beds is 35%. The porosity of the media was determined by direct measurement by pouring 500 ml of representative sample into the cylinder containing 500 ml of water.

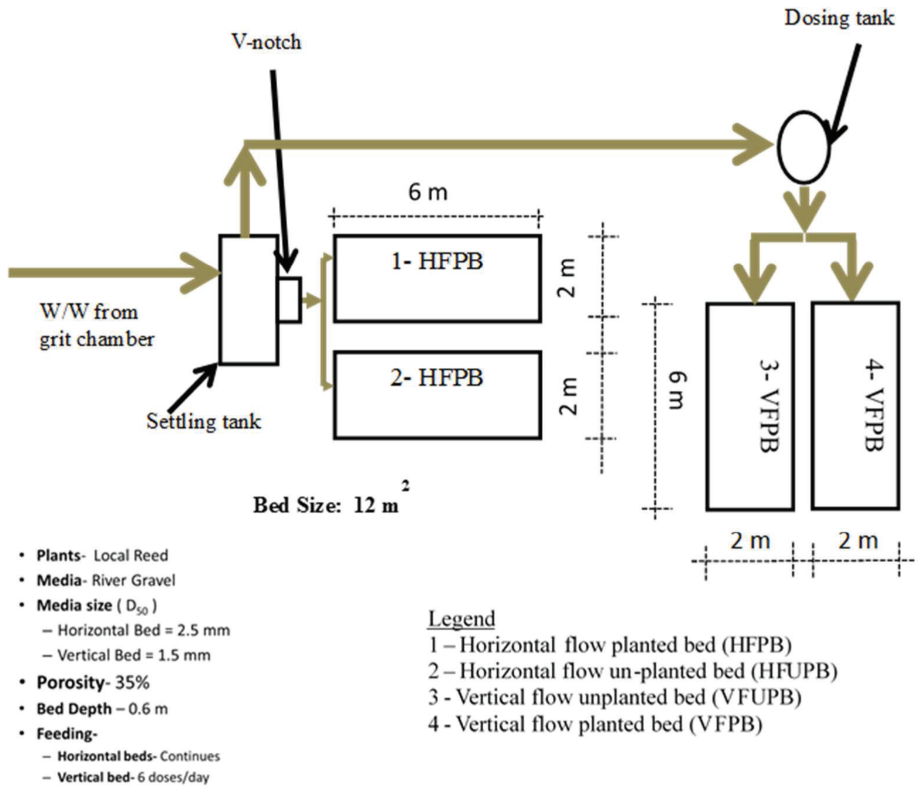


Figure 3: Plan of constructed wetland pilot system used in the experiment

The water level rising above the media was measured to calculate the pore volume.

Table 8: Grain-size characteristics of the media used in the HF and VF wetland beds.

Grain-size parameters	HF beds	VF beds	Recommended values from literature
Effective size (d_{10})	0.70	0.50	d10 > 0.3 mm (Vymazal et al. 1998)
Uniformity coefficient (d_{60}/d_{10})	4.5	4	d60/d10 < 5 (CPCB 2003)

One bed of each flow type was planted with *Phragmites Karkaa* (local reed) and the other is left unplanted. *Phragmites karka* was chosen because it is a common wetland species in Nepal (Shrestha et al. 2001a). Wastewater was drawn from the grit chamber. A v-notch weir was installed at the inlet side of the HF beds in order to measure the flow and ensure that flow is equally distributed in the beds. The HF beds were continuously fed whereas the VF bed received 6 doses/day using a pump and an overhead dosing/distribution tank. The daily total hydraulic load in both the beds were equal. In the HF beds the outlet pipes were adjusted so that the saturated depth was 45 cm (3/4 of the total bed depth). The VF beds had free drainage. The beds had an initial period of one year to stabilize the vegetation prior to running the experiments reported herein. The vegetation was not harvested during the experimental run.

Sampling, monitoring and analysis

The first phase of the experiment studied the effect of different hydraulic loading rates; 20 cm/day followed by 8 cm/day and 4 cm/day. Each loading rate was run for 21 days with sampling the last 7 days. Five inlet and outlet samples were collected for each loading rate. All water samples were 24-hour composite samples. In the second phase of the experiment the loading rate was adjusted to 4 cm/day and run for seven months to study the long term performance of the different beds. The average monthly temperature during the second phase experiment varied from 18°C to 23°C.

All samples were analyzed for TSS, BOD, COD, Total Kjeldahl Nitrogen (TKN), Total Phosphorus (TP), NH_4-N as per APHA (1985).

The horizontal beds had two sampling ports at a distance of 0.8 and 4.6 m from inlet. These ports were used to measure oxidation-reduction potential (ORP). The oxidation-reduction potential was measured in millivolts by using redox potential meter electrode.

Tracer studies were conducted in HF planted and unplanted beds to determine the hydrodynamic dispersion in the beds. Electric conductivity was monitored every half an hour. The conductivity was converted into NaCl concentration. From the concentration response data, the dispersion number was calculated according to closed-vessel equation of Levenspiel (2012) given below:

$t_{actual}=\Sigma tC/\Sigma C$…………………(11)

$\sigma_t^2=\frac{\Sigma t^2C}{\Sigma C}-t_{actual}{}^2$……………..….(12)

$\sigma^2=\sigma_t^2/t_{actual}{}^2$……………………(13)

$\sigma^2=2D-2D^2(1-e^{-D})$………..…....(14)

Where, t = time elapsed after the tracer injection, day, C = tracer concentration at time t, mg/l, σ_t^2 = variance (square of standard deviation) of the time-concentration curve (day^2), σ^2 = Variance in terms of dimensionless time, D = Dispersion number. Some photographs of the pilot scale units are presented in Fig.: 4, 5 and 6.

A B

Figure 4: A; horizontal flow (HF) planted and unplanted bed and B; vertical flow (VF) planted and unplanted beds.

A B

Figure 5: A; arrangement for intermediate dosing in the VF beds and B; flow distribution in the inlet zone of the HF beds

A

B

C

Figure 6: A; flow measurement in HF beds by V notch weir, B; outlets in HF beds and C; arrangement for sampling of effluents

3.2.2 Sludge drying reed beds

The pilot scale sludge drying beds are shown in Fig. 7. The units consisted of three identical beds with surface area 1.5m x 0.7m and a depth of 1m. Two beds were planted with Phragmites Karkaa (local reed) and one was left unplanted. The sequence and size of the filter media and the drainage layers were adopted from Koottatep et al. (1999) and is shown in Fig. 7.

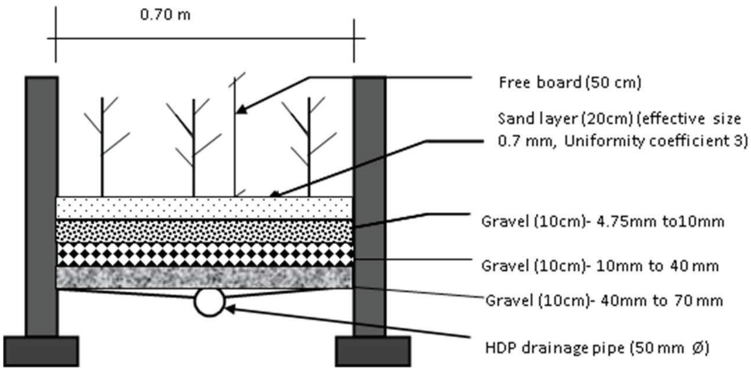


Figure 7: Cross sectional view of pilot scale sludge drying reed bed.

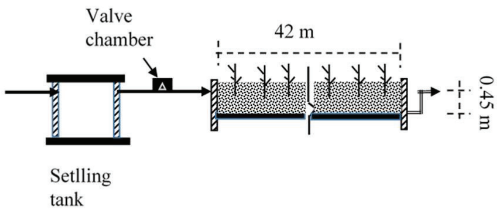
A 50 cm freeboard above the surface layer was provided for sludge accumulation. The bottom of the bed was sealed using a plastic membrane. The drainage pipe was connected to a vertical pipe at one end to assist aeration from below. The water percolating from the bed was collected and measured. Scales were placed on the beds to measure the sludge accumulation in the beds. The change in sludge depth was recorded at short time intervals for first 24 hours and then at the end of the each resting period of one week.

The beds had a plastic superstructure that allowed aeration, but prevented direct rainfall onto the beds. Prior to the actual experiment the planted beds were conditioned by planting reeds (4 plants/m^2) and loaded with wastewater for a period of two months. The reeds were then well established and had reached a height of 90 cm prior to the sludge application.

The beds were loaded with a sludge loading-rate (SLR) of 250 kgTS/m^2/yr (Planted 1) and 100 kgTS/m^2/yr (Planted 2).The sludge was obtained from a private company cleaning septic tanks in Kathmandu. The TS concentration of the septic tank sludge used for dewatering was different in each loading cycle. Therefore to maintain the constant SLR for each loading cycle the depth of application varied. The average depth of application for SLR of 100 kg TS/m^2 /year was 4.2 cm and for 250 kg TS/m^2/year. The sludge was fed every 7th day with 6 days resting between applications as suggested by Koottatep et al. (1999). The duration of sludge loading and monitoring was 2 months and was conducted from December through January. The average daily high and daily low temperature during the experimental period was 8^0C and 20^0C respectively. Composite samples of the stabilizing sludge were collected at the end of each loading cycle by mixing equal portions of sample from 4 quadrants of the bed. The sludge was analysed for moisture content (MC), total solids (TS), volatile solids (VS), total Kjeldahl nitrogen (TKN) and total phosphorus (TP) using standard method of analysis APHA (1985).

3.2.3 Full-scale horizontal flow wetland

The full-scale horizontal flow constructed wetland is shown in (Fig. 8). The horizontal flow bed is 42m long, 7m wide, and 0.45m deep. After primary treatment in a settling tank the wastewater is continuously fed into the bed. The media in the inlet and outlet zone consist of 40-80 mm crushed stone.



Horizontal flow planted bed

- Plants- Local Reed
- Media – River gravel
- Media Size
  - (d_{10}) – 21 mm
  - d_{60}/d_{10} = 1.2
- Porosity – 35%
- Bed depth – 0.45 m
- Width – 7 m
- Feeding- Continues

Figure 8. Cross sectional view of the full-scale horizontal subsurface flow bed

The porous media is 20-30 mm size river gravel. The bed is planted with *Phragmites Karkaa*. Sixteen sampling ports, equally spaced at 2.4 m, were installed along the middle longitudinal transect of the bed. The system was run at two loading rates, viz. 0.02 m/day and 0.05 m/day. Following initiation of a new loading rate, the bed was left to stabilize for three weeks before samples were taken. For each loading rate samples were collected from inlet, outlet and the sampling ports placed along the longitudinal transect. Daily composite samples were collected for 8 consecutive days for each loading rate.

3.3 Statistical analysis

Minitab statistical software was used to calculate arithmetic means and standard deviations of the variables. In Paper I to compare the performance of beds at different hydraulic loading rates a two away analysis of variance (ANOVA) test was used. The difference of the mean removal efficiency of the beds run at HLR of 4 cm/day a one–way ANOVA test at 95% confidence interval was used. A graphical comparison was expressed using box plots. In Paper III one-way ANOVA test at 95% confidence was used to compare the performance of the planted and unplanted beds and at different SLR. In Paper IV the least-square optimization procedure was used to estimate the model parameters. The fit of the model prediction data to the experimental data was evaluated by the coefficient of determination - R^2. The software tool Matlab was used for the fitting of the curves.

4. Main results

4.1 Comparative environmental and cost effectiveness of alternative decentralized system (Paper I)

Using life cycle analysis (LCA) the environmental performance of an existing centralized wastewater treatment system (alt. 1) was compared with three decentralized wastewater treatment alternatives: a) CW based system (alt. 2) b) CW system combined with separation of urine (alt. 3) c) Greywater treatment constructed wetland system combined with source separation of blackwater (alt. 4). Results of the comparison of the environmental performance of the alternative systems (**Fig. 4 in Paper I**) shows that with respect to the green house effect, the CW system combined with separation of urine has the best performance, whereas the CW system without source separation have highest impact on the greenhouse effect. Combining constructed wetlands and urine diversion (alt. 3) gives a net positive energy balance due to energy saved by substitution of mineral fertilizer. The source separating options (alt. 3 and 4) have the lowest eutrophication impact due to high nutrient removal.

The system with the lowest total annual cost is alternative 2 (2.5 USD/pe) and the highest cost is for alternative 4 (14.6 USD/pe). The investment cost is lowest for alternative 4 and highest for alternative 1. As seen from the annual operation and maintenance cost; which includes transportation of sludge/urine/blackwater, emptying the tanks, and regular harvesting of the sludge drying beds, the transportation of urine and faeces seems to be the determining factor for the economic sustainability of the decentralized with source separation (alt. 3 and 4).

4.2 Comparison of vertical and horizontal flow planted and unplanted subsurface flow wetlands treating municipal wastewater (Paper II)

The treatment performance regarding TSS, BOD, COD, TKN, NH_4-N and TP at different hydraulic loading rates (**Table 2 in Paper II**) shows that the percent removal increase with decrease in hydraulic loading rate for all beds and parameters, except for TP. The effluent discharge limit of 50 mg/l of BOD and 100 mg/l of TSS (Nepalese standard) was achieved at 0.08 m/day and 0.04 m/day loading rate for all beds except the unplanted horizontal flow HF bed. The Norwegian standard of 20 mg/l of BOD or 90% removal was met by the planted VF bed only. None of the beds met the Norwegian requirement of 90% P-removal. The EU-regulations require a minimum of 80% of P removal. The time dependent variation in the pollutant concentration and the statistical comparison of the different beds using box-plots (**Fig. 3 in Paper II**) showed that the VF planted bed had the best performance for all the tested parameters and for BOD, COD, TKN and NH_4-N this bed was also significantly better than the other beds. In general the planted beds showed better performance than the unplanted beds, but for the HF beds it is only for TSS, BOD and COD that the planted bed is significantly better than the unplanted bed. The HF planted bed is performing significantly better than the VF unplanted bed for BOD and COD. For TSS the HF planted bed, VF unplanted and planted beds have near equal performance and are all significantly better than the HF unplanted. For NH_4 both VF beds had an effluent concentration significantly lower than the other beds. This is due to high the oxygen availability in the VF-beds as these beds are operated under unsaturated flow conditions. This means that water is flowing in the smaller pores whereas the larger pores are filled with air. Unsaturated flow conditions are promoted by the intermittent dosing of the VF beds. The NH_4 in the effluent from the planted and unplanted HF beds was better than the influent on the average, but not statistically different from the influent showing a very limited NH_4-N removal in both HF beds. For the phosphorus there is no significant difference between the inflow concentration and the outflow of all the beds VF and HF. This is due to low phosphorus sorbing capacity of the porous media used in the wetlands, thus the phosphorus sorption capacity was saturated.

4.3 Sludge drying reed beds for decentralized sludge treatment (Paper III)

CW`s and sludge drying reed beds (SDRB´s) can be integrated to treat both wastewater and the sludge. The sludge loading rate, drying period and desludging frequency are the key design parameters. The liquid mass balance of the experiment (**Table 3 in Paper III**).showes that 25-33% of the water content of the sludge is lost through evaporation or evapotranspiration and 58 - 63% was lost by gravity drainage. The drying period required to achieve 30% TS content for SDRB varies from 13 days to 37 days for a sludge loading rate (SLR) of 250 kgTS/m^2/year and 6 days to 9 days for SLR of 100kgTS/m^2/year (**Fig. 3 in Paper III**). Longer drying time is required from June to September when precipitation is high and exceeds the evaporation. If the beds are covered the drying time in the months from June to September is significantly shortened. The rest of the year has more evaporation than precipitation and therefore the required drying times become shorter. In SDRB´s with matured plants the movement of the plants helps to make cracks on the surface of the residual sludge layer providing channels for the rainfall to pass through the beds. A final total solid content of 40-50% can be, theoretically, achieved in sludge drying reed beds. Correct loading and resting strategies based on local climatic conditions will maximize the dryness of the final residual sludge. The change in sludge depth was recorded in small time intervals for first 24 hours and then at the end of the each resting interval of one week. The sludge depth was rapidly reduced within first 24 hrs of sludge application indicating that initial loss of moisture content in the sludge is due to free drainage. After eight week of applications the depths recorded in Planted Bed 1 (250 TS kg/m^2/yr), Planted Bed 2 (100 kg TS/m2/yr) and unplanted bed 3 (100 kgTS/m^2/yr) was 0.9 cm, 0.3 cm and 0.4 cm respectively. For an assumed freeboard of 50 cm the sevice life of an SRDB until the accumulated sludge must be removed can be calculated. This gave 13 months for the planted bed loaded at 250 kg/m^2/yr, 38 months for planted bed loaded at 100 kg/m^2/yr and 29 months for the unplanted bed loaded at 100 kg/m^2/yr. This prediction is based on measurements over a short duration in the initial operation of the SRDB´s. The real life expectancy will probably exceed the above prediction, because mineralization can be expected to increase as the beds mature.

4.4 Assessing organic matter and nutrient removal in horizontal subsurface flow constructed wetlands using first order reaction rate models (Paper IV)

The tracer response curves for the HF planted and unplanted beds (**Fig. 3 in Paper IV**) for a hydraulic loading rate (q) of 0.27 m/day shows that the actual hydraulic retention time is 39-55 % shorter than the theoretical. The number of continuous tank in series reactor (N) calculated from tracer data was 4.

The effluent concentrations for TSS, BOD, NH_4-N and TP at different loading rates for the pilot- scale unit shows that (**Fig. 4 in Paper IV**) for BOD and TSS there is an exponential decrease in effluent concentration. The R^2 value show that both the k-C* and p-k-C* model represent the measured data well for the given ka and C* values (**Table 1 in Paper IV).** For NH_4 and for TP there is no apparent correlation. C* was used as a free fitting parameters and was adjusted to make the model fit the data. The areal rate constant (k_a) values for BOD were 17.8 m/yr for the k-C*model and 19.63 m/yr for the p-k-C* model. The measured longitudinal profile data from the full-scale wetland for COD (**Fig. 5 in Paper IV**) fitted with the model showed an exponential decline, but a plateau above zero was not reached. The longitudinal profile for both NH4-N and TP showed a steady and almost linear reduction along the wetland bed. The model fit was good ($R^2 < 0.88$). The rate constants increase with increasing (q) for COD and NH_4, but not for TP (**Table 3 in Paper IV**).

For the pilot- scale wetland (ka) values for BOD based on p-k-C* model was 19.63 m/yr. For the full-scale bed (ka) for COD was 26.9 m/yr. Kadlec (2009) found an average (ka) for 53 wetlands of 37 m/yr using the p-k-C* model. This is lower than all the values reported in temperate climate.

4.5 Potential of natural system for onsite treatment (Paper V)

Constructed wetlands (CW´s) are, as infiltration systems, technically simple and robust. CW´s produce a low BOD effluent and can reduce the number of indicator organisms substantially. However, CW´s normally do not reduce the nutrient content above secondary treatment standards so effluent from CW´s may cause eutrophication upon discharge. If the BOD in the incoming water is low infiltration is facilitated. The CW can be combined with the infiltration system to achieve high degree of pollutant removal and thus reduce the risk of ground water pollution. The northern part of Kathmandu valley and the areas along the Bagmati River that flows theroug the valley are composed of unconsolidated highly permeable materials that are up to 60 m thick and forms the main phreatic aquifer in the valley (**Fig. 3 in Paper V**). The central and southern part of valley are comprised of silty clay lake deposits, forming a clay aquitard protecting the deeper confined aquifer. The northern part of the valley has potential for large infiltration systems whereas in the central and southern part of the valley small household infiltration systems can be built with proper assessment of soil and groundwater conditions. However, the northern part has the highest risk for groundwater pollution.

Constructed wetlands are usually made with a sealed bottom. In fine-grained soils of low hydraulic conductivity as silt and clay soils, the wetlands are suggested to be constructed with an unlined bottom (**Fig. 4 in Paper V**)). This gives a cheaper wetland construction, and allows pretreated water to infiltrate whenever possible. In such fine-grained soils the purification capacity is generally excellent. How much that will infiltrate depends on the sizing of the system the hydraulic conductivity of the underlying soil and potential clogging of the wetland base. The overflow is treated water of secondary quality, and can be discharged to open waterways, but preferably dispersed in a shallow infiltration or drip irrigation systems. The effluent from a constructed wetland is rich in nitrogen and phosphorus but low in BOD and bacteria and, thus, well suited to irrigate green- or agricultural areas. An open bottom construction may give a fluctuating water level in the wetland and thus, plants that can tolerate varying moisture conditions (eg. Phragmites australis) should be selected.

5. Overall conclusions

The papers presented in this thesis represent an integrated approach to study the application of constructed wetland (CW) technology for wastewater management in Nepal. The overall conclusion drawn from this thesis is that the CW based treatment systems are suitable for Nepalese conditions. They can be constructed using local material and this makes them economically sustainable. In addition CW´s are technically robust and environmentally a good alternative to more technical and centralized systems. Constructed wetlands and sludge drying reed bed can be integrated to treat the sludge generated from on-site systems. CW´s can be combined with the infiltration systems for final effluent disposal as such contribute to groundwater recharge.

Comparative environmental and cost analysis of alternative decentralized wastewater treatment systems **(Paper I)**

1. Constructed wetlands contribute greenhouse gas emissions, but also act as a sink for CO_2 and the net effect the greenhouse effect is uncertain.
2. The conventional system had the highest energy consumption.
3. Transportation accounted for a significant contribution to the greenhouse gas emissions in the alternatives with separation of urine or blackwater.
4. Sludge dewatering beds may be a poor treatment option for blackwater due to high greenhouse gas emissions.
5. Combining constructed wetlands and urine diversion gives a net positive energy balance.
6. The constructed wetland had the lowest total annual cost.
7. The cost competitiveness of decentralized source separating systems depends on the transportation distance to the agricultural lands for recycling of nutrients.

***Comparision of horizontal and vertical subsurface flow wetlands treating municipal wastewater* (Paper II)**

1. The percent removal increase with the decrease in hydraulic loading rate for all beds and parameters except for total phosphorus.
2. For biogeochemical oxygen demand (BOD), chemical oxygen demand (COD), Kjeldahl-nitrogen and ammonium-nitrogen at loading rates 20, 8 and 4 cm/d the planted bed performed significantly better ($p < 0.05$) than unplanted beds, the vertical flow (VF) planted bed showed significant better removal than the VF unplanted bed, the VF planted bed performed significantly better than the horizontal flow (HF) planted bed. The superior performance of the VF beds, despite much shorter retention time than in the HF beds, can be explained by unsaturated flow conditions giving more air access as shown by the higher oxidation-reduction potential in the VF beds.
3. There was no significant difference ($p > 0.05$) in the removal of TP between any of the beds. This is due to low phosphorus sorption capacity of the porous media. This experiment shows that current Nepalese discharge standards are met by using HF wetlands alone. In order to meet stricter standards as in Norway or the EU a combination of horizontal and vertical beds are needed in addition to using a porous media with high phosphorus sorption capacity.

***Reed beds for sludge dewatering and stabilization* (Paper III)**

1. A short-term pilot-scale experiment can give valuable input to the design and operation of full-scale systems.
2. The overall dewatering efficiency of the planted bed was higher than the unplanted bed due to higher evaporation fraction in the planted bed. As the beds still were young during this study a larger effect of evapotranspiration can be expected when the roots are fully developed.
3. The planted beds had a higher VS reduction than unplanted beds indicating better conditions for degradation of organic matter and thus higher mineralization rate in the planted beds.
4. Based on the limited number of investigations currently available it is difficult to generate a model predicting loading rates versus climate based on sludge quality parameters, thus design of systems and selection of loading regime has to be based on

empirical data.

5. Based upon this short-term experiment and literature data an initial sludge loading (SLR) rate of 100 kgTS/m^2/year is suggested for the sub-tropical climate as that of Kathmandu. However, after one year of operation, when the plants are matured, the SLR can be gradually increased up to 250 kgTS/m^2/year.
6. Minimum resting period of one week between loadings and final resting phase of one year can ensure both adequate dewatering as well as a hygienized and stable end product. In warm climates with high annual precipitation, partially covered beds will reduce the drying time and consequently require less area than open beds.

Assessing Organic Matter and Nutrient Removal in Horizontal Subsurface Flow Constructed Wetlands using first order reaction rate models **(Paper IV)**

1. The model predictions for organic matter and ammonium show that larger wetlands or a longer retention time is needed to reach the maximum treatment capability of the wetlands. However, the wetlands were not hydraulically optimal and preferential flow paths occur. If a flow situation is achieved, where more wetland volume is utilized, the wetland volume (and area) can be reduced. This should be focus of further development of horizontal flow subsurface wetlands in Nepal.
2. Comparison of the tracer data from the planted and the unplanted bed, showed that the presence of plants increases dispersion and, hence, the retention time.
3. The modified first order model (k-C*) and the tank in series model (p-k-C*) are suited to describe biological removal reactions as for BOD, but not for phosphorus removal that is mainly dependent on sorption reactions.
4. The rate constants increased with hydraulic loading rate (q) for both models.
5. The p-k-C* model is regarded to describe the flow conditions in a horizontal flow wetland better than the k-C* model. However, in this study the models performed equally well (R^2) when fitted to the experimental data for total suspended solids (TSS), and organic matter measured as BOD and COD .
6. The rate constants (k_a) for organic matter removal determined in this study were low in comparison to rate constants determined for wetlands in colder climates. This indicates that the organic matter removal rate constants are not necessarily higher in warmer climates. Preferential flow and lack of established biofilms due to young system age may have contributed to the low k_a values.

Wastewater infiltration for purification and groundwater recharge - international experience and potential in Nepal (**Paper V*****)***

1. For sites with soil of low hydraulic conductivity, as in the southern part of the Kathmandu valley pretreating the wastewater in constructed wetland can enhance subsequent infiltration
2. Constructed wetlands are usually made with a sealed bottom. In fine-grained soils of low hydraulic conductivity as silt and clay soils, it is suggested that wetlands are constructed with an unlined bottom. This gives a cheaper wetland construction, and allows pretreated water to infiltrate whenever possible. In such fine-grained soils the purification capacity is generally excellent.
3. The existing soakpits can preferably to be upgraded to modern shallow infiltration systems. This will reduce the risk of polluting underlaying aquifers.
4. If properly sited and desgned infiltration gives excellent treatment and can help aquifer recharge. The risk of polluting the aquifers by infiltration systems is greatly reduced if greywater only is infiltrated.
5. In order to successfully implement infiltration systems in Nepal local guidelines for site assessment and system sizing and design should be developed. There is substantial international experience regarding infiltration systems that can facilitate development of Nepalese guidelines and design criteria.

6. Future studies

1. Sludge drying beds are a cheap and simple method for treatment of collected blackwater. The life cycle analysis (LCA) revealed that sludge drying beds may not be a good option for blackwater treatment due to production of climate gases and loss of more than 80% of the nitrogen to the atmosphere. This conlusion is based on results from fecal sludge. However, studies of sludge drying beds receiving raw blackwater are lacking and should be undertaken.
2. The competitiveness of source separating solutions, as described in **Paper 1,** are vulnerable to transportation distance from the source of the recycled resources to farmland. This is due to trucking of large amounts of liquid. If the urine or blackwater can be concentrated or solidified transportation costs and the environmental impacts from transportation can be greatly reduced. Production of struvite from urine or

blackwater is a promising method for extraction of both nitrogen and phosphorus from wastewater. Struvite is also an excellent slow release fertilizer. There are a few commercial processes for struvite productions. However, both simple and more sophisticated methods for struvite production, but also other models as vacuum distillation should be pursued in the case of Nepal if excreta from densely populated areas are to be recycled. Financial viability also needs to be studied.

3. Phosphorus sorption capacity of the media currently used in constructed wetlands in Nepal is low. Investigation is required to identify suitable industrial by- products or mineral products that can be use for phosphours adsorption in constructed wetlands.
4. Phragmitis karka is the commonly used wetland plant in Nepal. Further studies are required regarding the type of vegetation to be used under varying climatic conditions in Nepal. The plant selection is especially important if the wetlands are allowed to drain as suggested in P**aper V.**
5. Further experiments should be conducted in order to determine the optimal final resting period of sludge drying reed beds treating fecal sludge in sub tropical climate.
6. The constructed wetlands investigated in this thesis were not hydraulically optimal and preferential flow paths occurred. Technical design that facilitates a situation where more wetland volume is utilized and, hence, there is less preferential flow should be focus of further development of horizontal flow subsurface wetlands in Nepal.

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Paper I

Manoj K. Pandey, Petter D. Jenssen & John Morken. Comparison of a centralized and three decentralized wastewater treatment options using life cycle and cost analysis: a case of Kathmandu.

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Comparison of a centralized and three decentralized wastewater treatment options using life cycle and cost analysis: a case of Kathmandu

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Abstract

Using the life cycle analysis approach (LCA) and cost analysis, an existing centralized wastewater treatment system in Kathmandu was compared with three decentralized wastewater treatment alternatives: a) a constructed wetland (CW) b) constructed wetland combined with urine separation c) constructed wetland combined with source separation of blackwater. The LCA analysis focused on the construction, transportation, treatment processes of each of the alternatives, and eventually utilization of plant nutrients. The environmental performance of the alternative systems were compared for greenhouse effect, energy consumption and eutrophication. The LCA does not point out one option as the best, but it gives a basis for decision- making. The LCA reveals bottlenecks and strong and weak aspects of the systems compared. The LCA focus on environmental impacts and should be combined with other evaluation methods, as cost calculations, to obtain a more complete system analysis. Constructed wetlands (CW) contribute greenhouse gas emissions, but also act as a sink for CO_2 and the net effect the greenhouse effect is uncertain. The conventional system had the highest energy consumption. Transportation accounted for a significant contribution to the greenhouse gas emissions in the alternatives with separation of urine or blackwater. Sludge dewatering beds may be a poor treatment option for blackwater due to high greenhouse gas emissions. Combining constructed wetlands and urine diversion gives a net positive energy balance. The constructed wetland had the lowest total annual cost. The highest cost is for alternative wetlands combined with and blackwater separation.
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1. Introduction

Small scale and decentralized systems have proven to be effective in treatment and reuse of wastewater in urban areas of the developed and developing countries (Otterpohl et al. 1997). In Oslo, the capital of Norway, greywater from 33 apartments in a dense urban setting has been successfully treated for 14 years using a compact constructed wetland system (Sagen 2014). In Nepal and India constructed wetlands (CW's) have been successfully applied to treat hospital and municipal wastewater in a decentralized scheme (Billore et al. 1999; Shrestha et al. 2001a; Singh et al. 2009).

For cities having no or very poor sewer systems, as Kathmandu, it is important to plan for a future system that is environmentally and economically sustainable. The conventional treatment plants built with external funding have failed to operate satisfactorily because of the high energy demand and high operation and maintenance cost (Shrestha et al. 2001a, NTC 2009), therefore other options, as decentralized ones, need to be considered.

In the strive for sustainable systems the technical solutions have to match the goals for treatment performance and resource recovery with a minimum of environmental impact. Nutrient recycling can help to reduce the energy use associated with the production of commercial fertilizer as production of mineral fertilizers is energy intensive and an important contributor to production of climate change gases (Refsgaard et al. 1998). Wastewater treatment can perform at very high efficiency in terms of nutrient removal, for phosphorus in particular (Ødegaard et al. 2002). However, the cost, energy consumption and low degree of recycling of nutrients to agriculture have raised the question of the sustainability of traditional wastewater treatment systems. High performance wastewater treatment systems may reduce the pollution load in the receiving water body, but in doing so, the environmental burden might shift from water to air or soil because of consumption of more energy and chemicals (Emmersen et al. 1995).

The transportation used during construction and operation of a wastewater treatment plant is one main contributor to energy use and greenhouse gas production from the system (Dixon et al. 2003). Transportation of large volumes of urine/blackwater (urine and feces) from the households to agricultural land may not be sustainable from environmental point of view (Jenssen and Vatn 1991). Another energy-consuming factor is pumping of sewage that often is needed in a centralized wastewater collection system.

There are a number of tools available for analysis of sustainability of a given system such as: life cycle analysis (LCA), cost analysis and multi criteria analysis (Ashley et al. 1999). In this paper paper three decentralized wastewater treatment options are compared to an existing centralized system using LCA and cost analysis. The alternative wastewater management systems are:

a) Centralized conventional/oxidation ditch (Alternative 1)
b) Decentralized/wetland (Alternative 2)
c) Decentralized /wetland + Urine diversion (Alternative 3)
d) Decentralized/wetland + Blackwater diversion (Alternative 4)

2. Materials and methods

a. Treatment systems

i. The centralized treatment facility (Alternative 1)

The existing Guheswori sewage treatment plant (GSTP) is an oxidation ditch system. The features of the wastewater treatment plant are shown in Table 1. The treatment plant and collecting sewer was constructed to intercept and treat the wastewater discharged over an 11.5 km stretch of Bagmati River. The government currently operates the treatment plant.

ii. Decentralized Options

Alternative 2
In this alternative domestic wastewater generated from each house is pre-treated in septic tanks. Provision of a septic tank is mandatory by building construction bylaws in Kathmandu (NTC 2009). There are no guidelines for construction of septic tanks in Nepal and therefore the

geometry, depth and number of chambers varies from house to house. Average size of a one household septic tanks found from a survey was 5.31±0.39 m^3 (HPCIDBC 2011).

Table 1: Salient Features of the Guheswori sewage treatment plant (GSTP) (*from the record of GSTP)*

Type of system	Oxidation ditch system
Design flow, (m^3/day)	16416
Base year population (1996)	58000
Design year population (2021)	198000
Domestic wastewater, lpcd	100
Total sewer length	17 km
Interceptor sewer, km	11.5 km (24 inch dia)
Service area, ha	537 ha (Upper Bagmati Basin)
Primary unit	Mechanical bar screen Sump well Mechanical grit chamber
Biological unit	Carousel type oxidation ditches (2 units, volume 10400 m^3)
Secondary clarifier	Two units each of 27 m diameter
Sludge Treatment	Sludge drying beds (2 units)
By product of the plant (estimated)	Dry sludge 40 m^3/day (7% ds) Grit and sand – 3 m^3/day Screening – 2-3 m^3/day
Land area occupied the treatment system, ha	5

A liquid volume of 5m^3 has therefore been assumed in this study. An existing drain conveys the effluent from the septic tank to the decentralized treatment facility. Constructed wetlands with horizontal flow (HF) is proposed as the treatment unit. HF wetland beds are easy to construct and have less operational problems than vertical flow (VF) beds (Shrestha et al. 2001a; Vymazal 2009). However, nitrification is low in HF wetlands due to insufficient of oxygen supply by the wetland plants (Vymazal 2009, Cooper 2009).

The septic tank desludging frequency is set to be once per year. The sludge will be transported by truck to the sludge-drying reed beds located at the decentralized treatment facilities. Keeping in mind, the traffic and narrow streets a truck of 3m^3 liquid capacity of will be used (HPCIDBC 2011). These trucks are equipped with a diesel generator powering the suction pump. The sludge transported to decentralized facilities will be dewatered and stabilized in sludge drying reed beds. The total solid content of the residual sludge in the drying beds is assumed to be 30%. The residual sludge will be transported to agricultural land for application. The percolate from the sludge drying beds is discharged to water together with the wetland effluent for both alternative 2 and 3.

Alternative 3

This alternative is the same as alternative 2, but with urine diversion. Urine is collected separately through a PVC pipe of 50 mm diameter to a urine collection tank. For this the alternative existing toilets has to be replaced by a urine-diverting toilet (UDT). Each house will have a 500 liter polyethylene (PE) urine holding tank in addition to the septic tank. With an estimated production of 1.0 liter urine/person/day and additional 10%, increase for flush water the estimated emptying interval will be three months (4 times a year). The urine will be collected from each household by a 3 m^3 capacity truck and transported to the decentralized treatment facilities for storage and subsequent distribution to farmland. It is assumed that the truck of 3m^3 will empty 6 urine-holding tanks before returning to the urine storage facility where the urine will be stored for six months to achieve the necessary sanitation prior to agricultural application (WHO 2006).

Alternative 4

This alternative is the same as alternative 2, but with greywater separated from the flow stream. Greywater is collected in the septic tank and the wetland treats greywater only, hence a much smaller wetland than in alternative 2 and 3 is needed. The conventional toilets will be replaced by commercially available low flush toilets that uses 1.5 liter per flush of water. The blackwater (urine and faces) will be collected in a 1000 liter polyethylene (PE) tank. Blackwater needs treatment before application as fertilizer. Disposal of blackwater in a sludge drying reed bed will produce a clear non-smelling percolate containing the much of the nutrients in the blackwater. This nutrient solution is trucked to agricultural areas for application as fertilizer. The sludge retained on the bed will also be transported to agricultural land and applied as soil amendment.

b. Life cycle analysis

(i) Functional unit and system boundaries

The functional unit adopted for this study is the treatment of wastewater generated by one population equivalent (pe) in a year. The average flow generated by one population equivalent is set to 0.1 m^3/day (100 lpcd) as this is the assumed average wastewater generation in Kathmandu. Wastewater treatment systems are typically designed for a life expectancy of 20 to 30 years (Tchobanoglous 1991). For this study 30 years is chosen. LCA of a product or process consists of three phases: construction, operation and demolition (Curran 2006). For this study only the construction and operation phases have been considered. The system boundary chosen for the study is shown in Fig. 1.

In Kathmandu valley a septic tank followed by a soak pit is the most common method of wastewater treatment in urban and periurban areas where sewer lines do not exist (Metcalf and Eddy 2000). However, in the last decade the majority of the buildings in urban Kathmandu are connected to sewers which discharge the untreated wastewater directly to the Bagmati river or one of its tributaries, bypassing the existing septic tank. Pandey et al. (2006) investigated the avaibility of land along the river banks in the Kathmandu valley and found the area sufficient to built wetland systems at each sewer outfall except in the most heavily urbanized parts. The land along the rivers are government land and can be acquired for treatment purposes. The schematic layout of the sewer systems in the hypothetical study area in this analysis is shown in Fig. 2.

For the decentralized option a treatment plant is constructed at each outfall. For the centralized option a sewer intercepting the outfalls is constructed conveying the sewage to the treatment plant. The sewer collection system up to the river outfall is the same in both systems with the only difference that the septic tanks are bypassed in the conventional system. An intercepting gravity sewer has been shown to give little impact in an LCA analysis (Emmerson et al. 1995) and is therefore not included in the LCA, but is included in the cost anlaysis.

Transportation of products as sludge, urine and treated blackwater to agricultural land is included in the study. The fate of sludge and urine after farmland application is not considered. The sludge from the centralized treatment system is dewatered and stabilized in sludge drying beds. Since treatment in sludge drying beds are common to all four alternatives, the construction of the sludge drying beds are not part of the LCA analysis.

The distance from the centroid of the hypothetical study area to the nearest river is set to 2 km. The average distance from the treatment, facilities to the agricultural land have been assumed to be 10 km. This is a realistic distance if nutrients from wastewater resources are to be recycled to the farmers surrounding the urban Kathmandu. Since transportation have been reported to be one of the important factors influencing the sustainability of small decentralized system (Jenssen and Vatn 1991) transportation has been selected as one parameter in the sensitivity analysis.

(ii) Impact cateagories and Impact Assessment

The impact analysis in LCA includes impact characterization, normalization of impact and impact weighing (Curran 2006). For comparison in this study the contribution of the different treatment systems to the impact categories; climate change, energy use and eutrophication has been selected. The potential contribution of material, energy consumption and environmental emissions to each impact category was performed using SimaPro 7 software (Pré Consultants, 2010)

The impact of treating wastewater generated by one person annually has been modeled using three sub-components ; construction, wastewater processing and transportation of by-products of wastewater treatment (sludge/urine/blackwater).

The CML 2 method embedded in the SimaPro has been choosen for impact characterization. The CML2 is a problem oriented method where each process of the system under investigation is linked to an environmental effect such as greenhouse gas emissions or eutrophication (Renou et al. 2008).

In this study the normalization and the impact weighing processes have not been conducted.

(iii) Inventory of environmental inputs and outputs

The summary of material use, energy utilization and environmental releases over the construction and operational phases of the alternative wastewater management options are presented in Table 2.

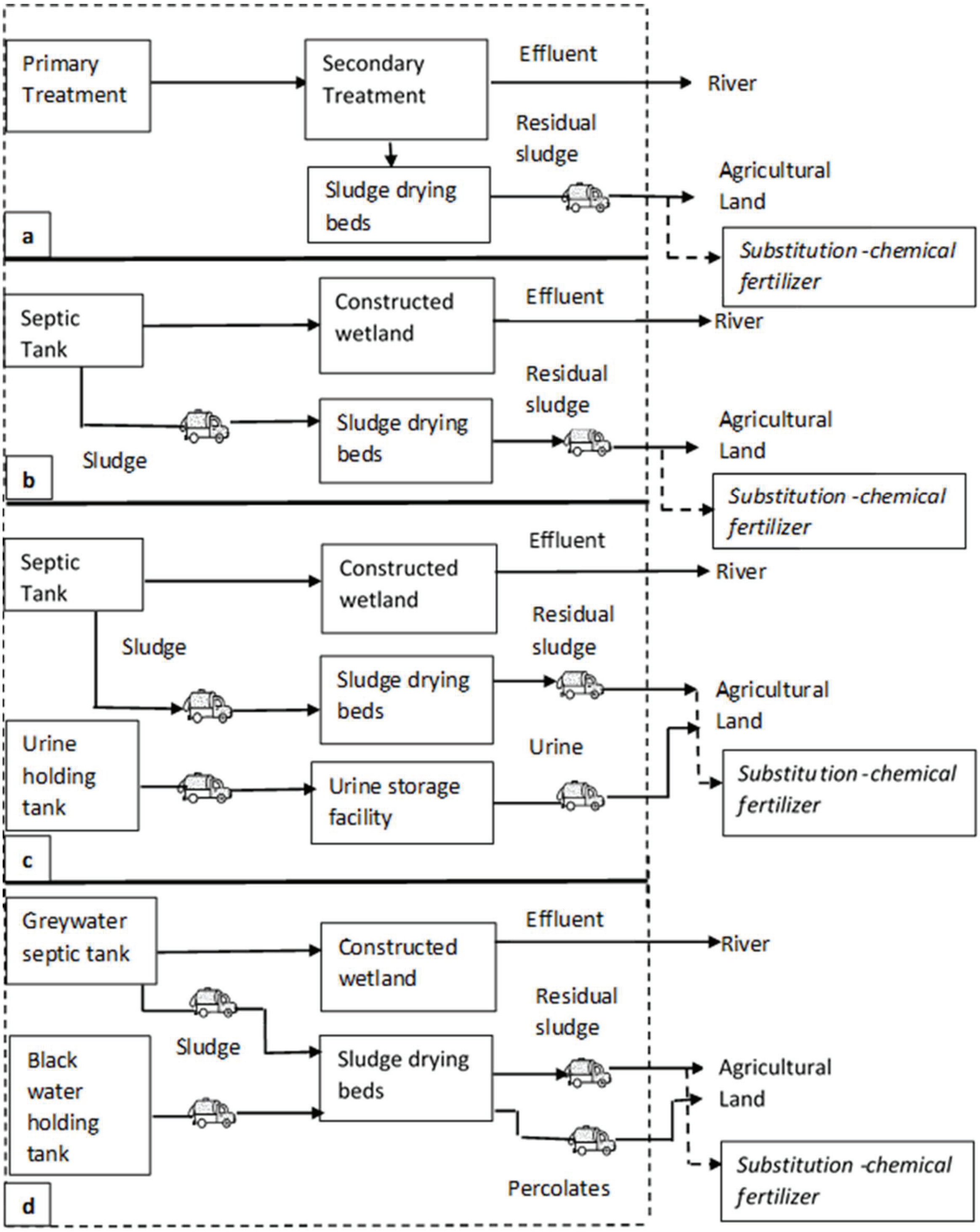

Figure 1. Systems and system boundaries: (a) Centralized/conventional (Alternative 1) (b) Decentralized/wetland (Alternative 2) (c) Decentralized /wetland + urine diversion (Alternative 3) (d) Decentralized/ wetland + blackwater diversion (Alternative 4).

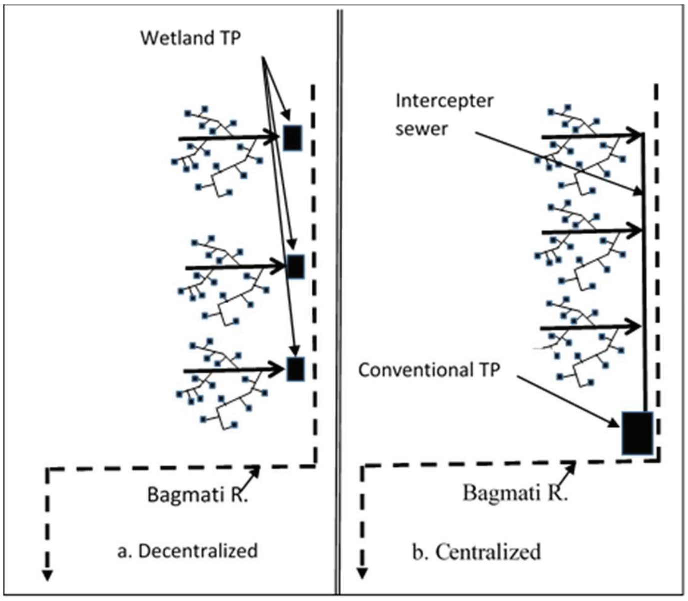

Figure 2. The schematic layout of the proposed decentralized systems (a) and the centralized system that are compared in this analysis.

The impact due to construction of infrastructure was distributed over a service life of the treatment system of 30 years. For each alternative the 1/30th of the construction impact was included in each year of operation. Only the major materials used for the construction are included in the inventories. Land area required in the inventory includes the foot print of treatment units and the land for office buildings and roads. The components of the domestic wastewater used in the inventory analysis are presented in Table 3. The treatment efficiency of GSTP and horizontal flow CW´s were adopted from Sah (2004) and Pandey et al. (2013) respectively.

The annual electricity consumption obtained from the record of GWWTP is 12.26 kWh/pe/yr. This is low in comparison to the values reported in other studies (Dixon et al. 2003; Ortiz et al. 2007). This may be because the aerators are powered by diesel generators used to run the aerators during power outage. In Nepal due to power shortage loadshedding is done up to 16 hours/day. There was no record of diesel consumed during the no electricity period. Therefore a gross energy consumption of 0.66 kWh/m^3 suggested by Tchobanoglous (1991) has been adopted for this study. An energy consumption of 3 kWh is assumed for each pumping of the septic tanks.

A yearly sludge accumulation of 0.3 l/pe/d has been used to estimate the sludge production in the septic tanks (Brandes 1978). For alternative 4 a toilet flush volume 1.5-liter is assumed which translates into 7liter/pe/day of blackwater to be collected in the blackwater holding tank (Jenssen et al. 2003).

Table 2: Summary of material use, energy utilization and environmental releases for treatment of wastewater from one functional unit (m^3/pe/y).

Item	Unit	Centralized/conventional Alternative 1	Decentralized Options		
			Wetland Alternative 2	Wetland+ Urine diversion Alternative 3	Wetland+Blackwater diversion Alternative 4
Construction of treatment units					
Input – Material and Resources					
Brick	kg	-	2.7	2.18	1.6
Concrete, normal, at plant	**m^3**	$2.0*10^{-2}$	$2*10^{-3}$	$1*10^{-3}$	$1.2*10^{-3}$
Reinforcement Steel	kg	2.04	$2.3*10^{-1}$	$1.8*10^{-1}$	$1.3*10^{-1}$
Steel, low/alloyed	kg	$7.7*10^{-2}$	-	-	-
Gravel	**kg**	$2*10^{-2}$	36.11	28.51	20.9
Sand, from ground	**kg**	$1*10^{-2}$	10.9	8.6	6.3
Polyethelene (HDPE)	kg	$8*10^{-2}$	-	$1*10^{-1}$	$2*10^{-1}$
Polyethelene (LDPE)	kg	$7.9*10^{-3}$	-	-	-
PVC pipes	kg	-	$1*10^{-2}$	$1*10^{-1}$	$8*10^{-2}$
Land requirement	m^2	$3.3*10^{-1}$	1.9	1.5	1.1
Wastewater processing					
Input-Material					
Wastewater	m^3/p.e/yr	36.4	36.4	36.4	28.56
Input- Energy					
Electricity /fuel	kWh	24.02	3	5	16.5
Output					
Dewatered sludge	kg	18.2	13.95	13.95	424.66
Emissions- In water					
BOD	kg	3.63	3	2.79	1.8
COD	kg	6	5.25	4.81	2.15
SS	kg	9.2	2.35	2.35	$5.9*10^{-1}$
TP	kg	$5.6*10^{-1}$	$6.3*10^{-1}$	$4.1*10^{-1}$	$1.2*10^{-1}$
TN	kg	3.03	2.8	1.27	$2.8*10^{-1}$
Emissions- In Air					
CO_2	kg	6.11	5.4	5.02	3.24
N_20	kg	$1.5*10^{-2}$	$2.23*10^{-2}$	$9.9*10^{-3}$	$2*10^{-2}$
Methane	kg	-	1.93	1.78	1.15
Transportation	t-km	$1.8*10^{-1}$	$1.3*10^{-1}$	1.3	22.98
Avoided Product					
Equivalent fertilizer value of sludge and percolate	kg NP	0.4	0.2	3.3	1

Direct emissions of greenhouse gases occur during wastewater and sludge processing. The indirect emissions are attributed to transportation and production of materials for construction. The compounds contributing to the green house effects from wastewater treatment and disposal are: carbon dioxide (CO_2), methane (CH_4) and nitrous oxide (N_2O) (Tchobanoglous 1991, IPCC 2006).

Table 3. Average composition of components of domestic wastewater, biochemical oxygen demand (BOD), chemical oxygen demand (COD), total nitrogen (TN) and total phosphorus (TP) (Vinnerås 2002; Otterpohl et al. 2003; Mattila 2003). The composition of blackwater is the sum of urine and feces.

Parameter	BOD		COD		TN		TP		Volume
	kg/pe/yr	%	kg/pe/yr	%	kg/pe/yr	%	kg/pe/yr	%	Lit./pe/yr
Feces	5.47	30	12.3	47	0.55	10.9	0.24	30	50
Urine	1.8	10	3.6	12	4	79.2	0.4	50	500
Grey water	10.9	60	14.1	41	0.5	9.9	0.16	20	29120
Total	18.19	100	30	100	5.05	100	0.8	100	

In an activated sludge system organic carbon (expressed as BOD) is partly removed in primary and secondary sludge and the remaining is emitted as CO_2. Based on transfer coefficients suggested by Doka (2009) 58% of the eliminated BOD is assumed to be transferred to the sludge and 48% is assumed to be emitted as CO_2.

The N_2O emissions are calculated using equation 1 suggested by IPCC (2006).

$$N_2O = N_{eff} * EF_{eff} * \frac{44}{28} \quad \text{..................................(1)}$$

Where, N_{eff} = nitrogen in the effluent discharged into water bodies, kg/N/yr; EF_{eff} = emissions factor for N_2O emissions from wastewater discharged into water bodies , kg N_2O-N/kg N; The factor 44/28 converts the kg N_2O-N into kg N_2O. The default emission factor suggested by IPCC (2006) is 0.005. In this study the IPCC model (equation 2) is used to estimate methane production in the septic tank.

$$CH_4 = \text{BOD removed in septic tank} * B_0 * MCF \text{......(2)}$$

Where, B_o = maximum CH_4 producing capacity, kg CH_4/kg BOD removed; MCF = methane correction factor, (in fraction). IPCC suggested a default value for B_o of 0.6 kg CH_4/kg BOD removed and for MCF 0.5.

In a wetland about 60% of incoming BOD is converted into CO_2 (Picek et al. 2007). Based on the study of Mander et al. (2005) and Søvik et al. (2006) the CH_4 emissions from the CW beds are set to 0.04 kg CH_4/m^2/yr. The CH_4 is emitted during sludge processing is beyond the study boundary therefore is not included in the analysis. The emission of N_2O from the CW systems has been calculated using equation 1.

The transport required to remove the sludge (all systems) and urine/blackwater (alt. 3 and 4 only) is expressed in ton-km. Data for transport related emissions refer to a 50% loading factor meaning that the delivery vehicle is full on its outward journey and empty on its return journey or opposite.

The nutrient contents of dewatered and stabilized sludge is calculated by performing a mass balance of inputs and outputs of the nutrients in the treatment systems. The nutrient content of source separated urine is based on the composition presented in Table 3. The equivalent amount of commercial fertilizer that can be substituted by the application of sludge and urine to agricultural land is based on the nutrient content of the sludge and urine transported to the agricultural land. The equivalent fertilizer that can be substituted by the nutrient recovery process in each alternative is calculated and presented in Table 2. The environmental burden that is avoided by the substitution of the fertilizer is included in the LCA. The commercial fertilizer that is substituted in this study is assumed as *ammonium nitrate phosphate*.

c. Cost analysis

The cost analysis includes the capital cost and the operation and maintenance cost for the period of 30 years. The investment and operational cost of alternative 1 has been obtained from the records of GSTP. The cost of alternative 1 includes the cost of the interceptor sewer, the pumping station and the primary and secondary treatment process. The investment and operational costs of the decentralised options have been quantified using power function of the form: $Y = a X^{b}$. Where, cost of constructed wetland (Y) is the function of X (e.g. flow rate, population equivalent, area of the wetland). Such cost functions has been used to quantify the capital and operational cost of wastewater treatment systems including constructed wetlands (Chamblee 1981; Gillot et al. 1999; Heaney et al. 1999; Tsagarakis et al. 2003; Kadlec and Wallace 2009).

The CW´s in Nepal are constructed using local material and local labor therefore the local cost figures are used for the cost estimations. The parameter "*a*" and "*b*" was derived by fitting the power function to the available data of constructed wetlands in Nepal (Shrestha et al. 2001b, UN-Habitat 2008). The resulting cost functions are:

Capital Cost (excluding cost of land) (USD)= 99.31 $X^{0.87}$ (R^2= 0.96)
Operation and maintenance cost (excluding vehicle operating cost) (USD) = 4.39$X^{0.822}$
(R^2= 0.73),

Where, X is the constructed wetland surface area in m^2.

The operational cost function above includes the cost of desludging the septic tank and regular maintenance of the wetland beds. The cost of transporting the faecal sludge and urine were calculated separately and added to get the total annualized operational cost. The total vehicle operating cost which includes the annual depreciation, average annual interest and yearly maintenance cost of the vehicle is USD 0.5 /ton-km. This value is from the records of Lalitpur metropolitan office, Kathmandu (Silwal 2011).

The net present value (NPV) of the capital and operating costs is used for economic comparison of the alternative wastewater management scenarios. All annual operating cost for each process is converted into their corresponding present value and added to the investment cost of each process to yield the net present value (Tsagarakis et al. 2003). The options with lowest NPV are regarded as more economically viable options. The cost of land was calculated based on prices of the recent land sale in the area.

3. Results and Discussions

a. **Impact characterization**

The relative contribution of transportation, wastewater treatment and construction for the impact categories selected is presented in Fig. 3. The impact categories are; green house gas emissions, eutrophicantion and energy resources. Fig. 3 shows values for the individual systems whereas in Fig. 4 the alternative systems are compared to each other by impact category.

The greenhouse house effect from the conventional system (alt.1) is mostly contributed by the material consumption during the construction phase (Fig. 3a). The environmental emissions are associated with the production of concrete and steel, which are used in large quantities when building a conventional system. The emissions during the biological wastewater treatment process (alt. 1) is mainly CO_2 which is part of a short-term ecological carbon cycle (Emmersen et al. 1995). N_2O emissions are low as indicated in Table 2.

In the wetland (alt. 2) the major contribution to the greenhouse effect is from the wastewater treatment process (Fig. 3b). This is due to emission of CO_2 and CH_4. The major source of CH_4 emissions are the septic tanks, but the CW beds also contribute to emissions of CH_4 (Mander et al. 2005). On the contrary CW´s act as a sink for CO_2 by photosynthetic assimilations from the atmosphere and sequestration of the organic matter produced in the wetland (Brix et al. 2001). This effect is not accounted for by this LCA analysis. Therefore the net effect of the CW beds regarding the greenhouse effect is uncertain, but more favorable than predicted by this analysis. This is important to consider when comparing CW based systems to other systems with respect to the greenhouse effect.

For the decentralized systems (alt. 2, 3 and 4), where a CW is the main treatment component, the resource consumption during construction has little impact on the greenhouse gas emissions. This is mainly because the decentralized alternatives use locally available raw materials for construction. However, the use of plastic materials in alternative 3 and 4 (Table 2) have contributed to the greenhouse effect for these two options.

In alternative 3 and 4 (Fig. 3: c and d) transportation accounts for a significant contribution to greenhouse effect.. The emissions from transportation is due to the trucking of urine (alt. 3) and blackwater (alt. 4). The blackwater volume generated, 7 liter/pe/d, is much larger than the urine volume of 1 liter/pe/day and thus the relative contribution to the greenhouse effect from transport is larger in alternative 4 than 3. The volume differences between the urine and blackwater is also expressed through the impact on energy resources where transportation is almost insignificant in alternative 3, but the main contributor in alternative 4.

The nitrogen and phosphorus discharged with the effluents are the main contributors to the eutrophication process for all four systems. The LCA analysis calculates the eutrophication effect of N and P in phosphate equivalents (Table 2). The eutrophication impact depends upon the nutrient removal efficiency of the treatment options and is better compared in Fig. 4.

In alternative 1 (Fig. 3a) most energy resources are used during wastewater treatment. This is because 70% of the total operational energy is consumed during running of the aerators in a system as that in GSTP. Similar results are reported in other studies (Zhang and Wilson 2000; Doka 2009).

Airborne emissions are also associated with energy consumption. In this study the electricity is assumed to be produced by hydropower. If the electricity was produced by using fossil fuels the resulting impacts predicted by the LCA on the greenhouse effect would be different because of the disturbance of a long-term geological carbon cycle (Zhang and Wilson 2000; Emmersen et al. 1995). The effect of converting to fossile resources for power production would have greatest impact on alternative 1, because the conventional system is consuming more electricity than the other options (Fig. 4 and Table 2).

The LCA analysis displays negative values in the impact category energy resources with respect to the wastewater treatment in the alternatives 2-4 (Fig. 3: b, c and d). This is due to the nutrients recovered in these systems that substitute commercial fertilizer (Table 2). The LCA analysis subtracts the energy saved by recovery from the energy used for wastewater treatment. A negative value is therefore to be viewed positive in an environmental accounting. The nutrient recovery, through return of sludge to agriculture, is about the same in alternative 1 and 2, but since the total energy use is much higher in alternative 1 (Fig. 4) the effect of nutrient recovery is insignificant with respect to the impact on energy resources for alternative1. The low total energy consumption for alternative 2 (Fig. 4) explains why construction has a relatively large impact on use of energy resources.

Comparison of the environmental performance of the alternative systems (Fig. 4) shows that with respect to the greenhouse effect alternative 3 has the best performance whereas the CW system without source separation have highest impact. The conventional system (alt. 1) has lower greenhouse emissions than the blackwater separation system (alt. 4). The low greenhouse gas impact of the alternative 3 can be explained by the fact that the N_2O emissions have been substantially reduced due to removal of large amounts of nitrogen from the wastewater stream through urine diversion. Alternative 2 has the largest relative impact on greenhouse gas emissions. This is because of the CO_2 and CH_4 emissions from the wetland bed, however the carbon sequestration, in the wetland, as discussed above is not accounted for by this LCA. The LCA calculates the emissions based on wetland area and nutrient input. The wetland area is largest in alternative 2 where all the wastewater is treated in the CW. The CW is smaller for alternative 3 that treats greywater and feces and smallest for alternative 4 where only greywater is treated.

Table 2 shows that the equivalent fertilizer value (kg NP) is highest for alternative 3 (3.3 kg/pe/yr) and only 1kg/pe/yr for alternative 4. Blackwater (alt. 4) contains substantially more nutrients than urine collected in alternative3 (Table 3). The reason for the low NP value of the blackwater is that Kottatep et al. (2004) found that in sludge drying beds loaded with fecal sludge, 82% of the nitrogen was lost through volatilization as NH_3 or denitrification, 13% was incorporated in the dried sludge, and only 5% was in the percolate. Faecal sludge is similar to blackwater and therefore we have used the Kottateps (2004) values to derive at the NP value for alternative 4. Because of the low NP value however, no studies of greenhouse gas emissions from sludge drying beds

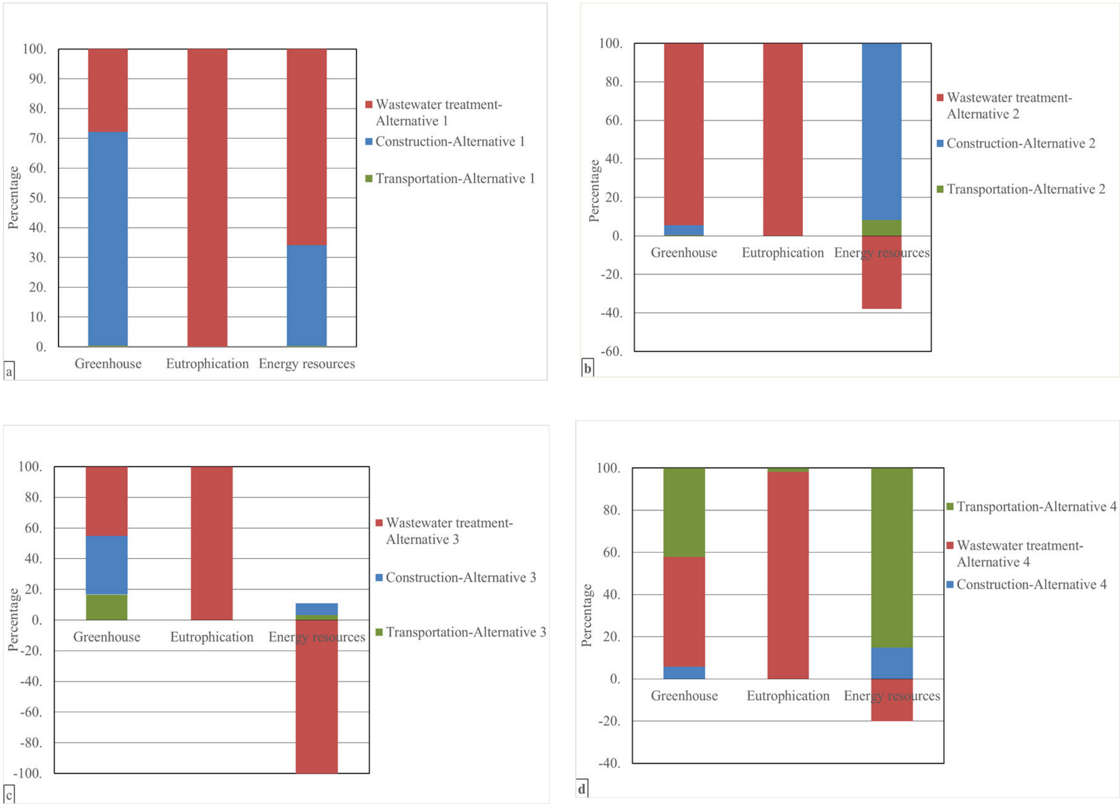

Figure 3: Relative contribution to greenhouse effect, eutrophication and use of energy resources from construction, wastewater treatment and transportation in the alternative systmes: (a) Centralized/conventional (Alternative 1) (b) Decentralized/wetland (Alternative 2) (c) Decentralized /wetland + urine diversion (Alternative 3) (d) Decentralized/ wetland + blackwater diversion (Alternative 4).

receiving raw blackwater have been reported. If the emissions predicted by Kottatep et al. (2004) are applicable to blackwater the greenhouse gas emissions for alternative 4 is underestimated since the emissions from the sludge drying beds are not accounted for. This analysis therefore points to sludge dewatering beds as a poor treatment option for blackwater due to high greenhouse gas emissions and thus low recovery of nutrients, N in particular. However, alternative 4 has the best performance with respect to eutrophication because of the large fraction of nutrients removed with the blackwater (Table 3).

Both the conventional systems (alt. 1) and CW system (alt. 2) provide secondary treatment only and, thus, have an almost equal and high impact regarding eutrophication (Fig 4 and Table 2) . This is as a result of poor nutrient removal efficiency of both of these systems. Horizontal flow CW beds have high removal of BOD, COD and TSS, but often low ammonia and phosphours removal efficiency (Vymazal 2005). In order to achive a higher degree of nitrogen removel a longer retention time and consequently larger area or combination with a vertical flow wetland is required (Luederitz et al. 2001; Vymazal 2005). This may not be feasible in urban areas where land aviability is limited.

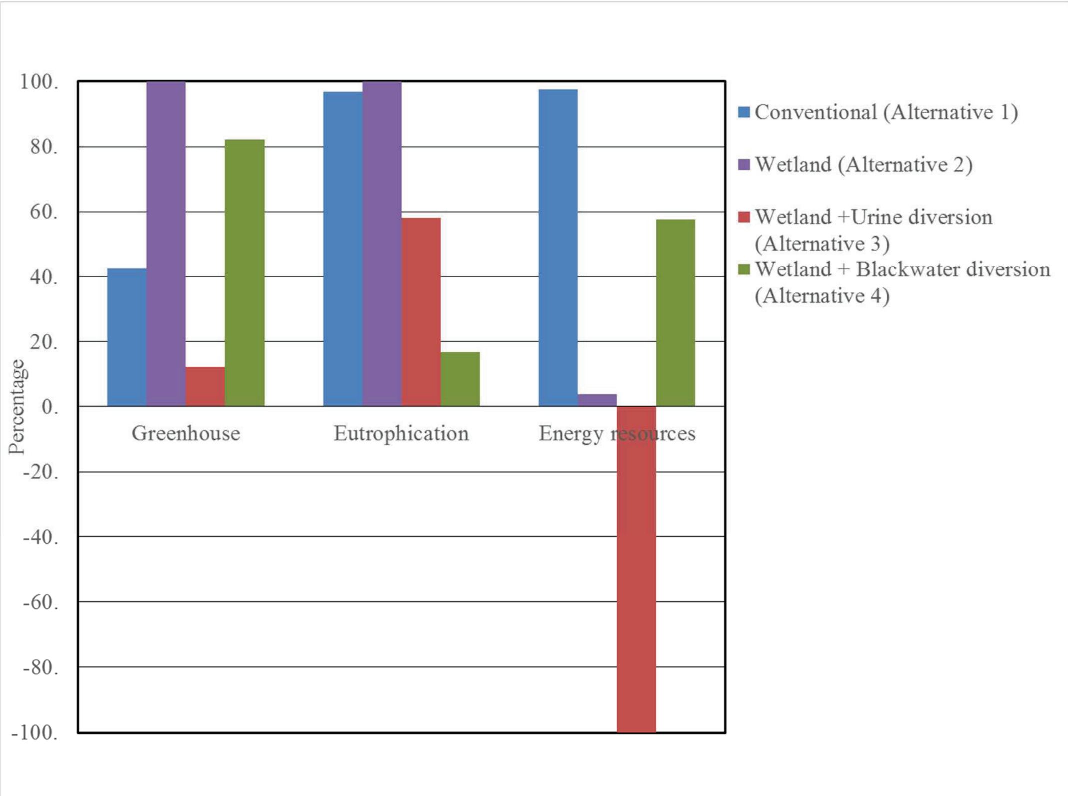

Figure 4: Comparision of environmental performance of the alternatives: (a) Centralized/conventional (Alternative 1) (b) Decentralized/wetland (Alternative 2) (c) Decentralized /wetland + urine diversion (Alternative 3) (d) Decentralized/ wetland + blackwater diversion (Alternative 4).

Phosphorus is considered the main element triggering eutrophication in fresh waters (Jenssen et al. 2010). Secondary treatment (alt. 1 and 2) will therefore not remove enough nutrients to prevent algae growth and eutrophication of the river. Removing the urine reduces the P-discharge by about 50% (Table 3), but this is still not enough to eliminate the risk of eutrophication of the rivers of Kathmandu because of the low flow compared to the effluents discharged. If the eutrophication of the Bagmati river is to be eliminated the nutrient input must be substantially reduced. Phosphorus discharge by alternative 1 can be reduced by chemical precipitation (Ødegaard et al. 2002). In wetlands high Premoval is achieved using specialized P-sorbing media like shell sand, light weight aggregate or crushed brick (Jenssen et al. 2010). However, the wetland P-filters need large volumes or frequent exchange of media. For a city like Kathmandu using traditional wetlands (alt. 2) with P-sorbing media is therefore not a feasible option. A better option is probably to use the nutrient rich effluent (alt. 2) for irrigation. In an urban setting irrigation of green areas like parks and river banks is an option (Pandey et al. 2006). If these areas are harvested the nutrients can be permanently removed.

If urine is collected and combined with a P-sorbing media in the wetlands, and/or irrigation of green areas, sufficient P-removal to avoid eutrophication should be possible to obtain. Alternative 4 removes all the blackwater and, hence, 80% of the phosphorus and 90% of the nitrogen (Table 3). This is higher removal than all the other alternatives, but may still not prevent eutrophication of the Bagmati river. However, for alternative 4 much smaller volumes of P-sorbing media or irrigation area is needed.

The LCA does not give a clear alternative, but predicts that alternative 3 is best with respect to energy resouces. Alternative 4 is best with respect to eutrophication, but it is uncertain how much nitrogen is lost to the atmosphere in the sludge dewatering process. The recovery of nutrients and substitution is highest in alternative 3. The energy used in alternative 4 is high because of the need for transport. The CW system without source separation (alt. 2) has very low energy requirements, because the wetland is gravity operated and only energy required is for emptying of septic tanks. But alternative 2 has the worst performance with respect to greenhouse gases. Alternative 1 has the highest energy consumption of all the options, this is due to energy used for both construction and operation of the system (Fig. 3a and Table 2). High energy consumption by conventional secondary treatment systems has been reported in other studies as well (Benetto et al. 2009).

b. Cost Analysis

The system with the lowest total annualized cost is alternative 2 (USD 2.5 /pe) and the highest cost is for alternative 4 (USD 14.6 /pe). The high annual cost for alternative 4 is due to the large operational costs, because alternative 4 has the lowest investment cost. In developed countries investment cost of natural systems (alt. 2) is often higher than conventional systems (alt. 1) because the design and production of conventional systems are highly standardized (Batchelor and Loots 1997; Vymazal 2007; Mannino et al. 2008).

In developing countries natural systems normally have a low investment as well as operational cost due to use of local resources. Table 4 show that the natural system (alt. 2) is cheaper than alternative 1 both for investment and operational cost as can be expected in a developing countries. However, the costs of alternative 3 and 4 are higher than the centralized alternative. This is because of the cost involved in trucking of urine (alt. 3) and blackwater (alt. 4).
The transportation of urine and blackwater seems to be the determining factor for the economic sustainability of the decentralized alternatives 3 and 4 as pointed out by Jenssen and Vatn (1991). If these systems are to be optimized the cost trucking must be reduced. The dry matter content in urine is 3.8 % (Vinnerås 2002) and blackwater collected with low flush toilets 0.5 – 1% (Jenssen and Skjelhaugen 1994), hence, a lot of liquid is transported. The liquid content can be reduced if the urine of blackwater is solidified as by production of struvite (Le Corre et al. 2009). Struvite is a magnesium-ammonium-phosphate mineral and a good slow release fertilizer. Etter et al. (2011) studied the feasibility of struvite recovery from urine in Nepal and found it was financially sustainable if high volumes were processed. If the volume of urine/blackwater is substantially reduced alternative 3 and 4 will be economically much more competitive.

Table: 5. Total investment cost, operation and maintenance cost, present value and annual cost for the alternatives 1-4.

	Alternative 1 conventional	Alternative 2 wetland	Alternative 3 wetland + urine diversion	Alternative 4 wetland + blackwater diversion
Construction cost (USD)	4 528 220	1 927 400	1 482 048	869 444
Land cost (USD)	28 220	1751 476	1 294 382	700 590
Total Investment cost (USD)	4 556 439	3 678 876	2 776 430	1 570 034
Annual operation and maintenance cost (USD)	143 750	63552	326700	2287782
Total Present Value (USD)	5 991 203	4325662	6101345	24853422
Annualized cost (USD)	699 950	505365	712 817	2,903 614
Total annual cost per person (USD/pe)	3.5	2.5	3.6	14.6

4. Conclusion

The LCA does not point out one option as the best, but it gives a basis for decision making. The LCA reveals bottlenecks and strong and weak aspects of the systems compared. The LCA focus on environmental impacts and should be combined with other evaluation methods, as cost calculations, to obtain a more complete system analysis;

- The conventional system (alt. 1) has the highest energy consumption of all the options. The greenhouse house effect for alternative1 is mostly contributed by the material consumption during the construction phase. For the decentralized systems (alternative 2, 3 and 4), where a constructed wetland is the main treatment component, the resource consumption during construction has little impact on the greenhouse gas emissions. This is mainly because the decentralized alternatives use locally available raw materials for construction.
- Constructed wetlands (CW) contribute to emissions of CO_2 and CH_4. However, constructed wetlands also act as a sink for CO_2. Therefore the net effect of the constructed wetlands regarding the greenhouse effect is uncertain.
- Transportation accounts for a significant contribution to greenhouse effect in the alternatives with separation of urine or blackwater. This is due to the trucking of urine and blackwater. If the urine of blackwater is solidified by production of struvite significant reductions in operational costs and a more positive environmental impact can be obtained.
- This analysis points to sludge dewatering beds as a poor treatment option for blackwater due to high greenhouse gas emissions and low recovery of nutrients for N in particular.
- The source separating solutions have the lowest eutrophication potential and the largest nutrient recovery.

- Combining constructed wetlands and urine diversion (alternative 3) gives a net positive energy balance due to energy saved by substitution of mineral fertilizer.
- The constructed wetland (alt. 2) has the lowest total annualized cost (USD 2.5/pe). The highest annualized cost (USD 14.6/pe) is for alternative 4 (wetland + blackwater separation). The high annual cost for alternative 4 is due to the large operational costs associated with transportation. Alternative 4 has the lowest investment cost and potential to become very cost effective if the transportation is reduced.

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Paper II

Manoj K. Pandey, Petter D. Jenssen, Tore Krogstad & Sven Jonasson. Comparison of vertical and horizontal flow planted and unplanted subsurface flow wetlands treating municipal wastewater. *Water Science and Technology*, 68 (1), pp. 117 – 123, 2013.

Comparison of vertical and horizontal flow planted and unplanted subsurface flow wetlands treating municipal wastewater

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ABSTRACT

In the search for design criteria for constructed wetlands (CWs) in Nepal a semi-scale experimental setup including horizontal flow (HF) and vertical flow (VF) CWs was developed. This paper compares the performance of HF and VF wetlands, and planted with unplanted beds. The experimental setup consists of two units of HF and VF beds of size 6 m × 2 m × 0.6 m and 6 m × 2 m × 0.8 m (length × width × depth) respectively. For both HF and VF systems, one unit was planted with *Phragmites karka* (local reed) and one was not planted. The systems were fed with wastewater drawn from the grit chamber of a municipal wastewater treatment plant. The media consisted of river gravel. In the first phase of the experiment the hydraulic loading rate (HLR) was varied in steps; 0.2, 0.08, 0.04 m^3/m^2/d and the percent removal increase with decrease in HLR for all beds and parameters except for total phosphorus. In the second phase the loading rate of 0.04 m^3/m^2/d was run for 7 months. In both parts of the experiment the planted beds performed better than the unplanted beds and the VF better than the HF beds. To meet Nepalese discharge standards HF beds are sufficient, but to meet stricter requirements a combination of HF and VF beds are recommended.

Key words | constructed wetland, decentralized wastewater management, horizontal flow systems, vertical flow systems

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INTRODUCTION

In recent years, there has been an increased interest in decentralized treatment of urban wastewater using natural treatment systems as constructed wetland (CW) (Jenssen & Vråle 2003; Parkinson & Tayler 2003). CWs are cost effective and easy to operate and thus suitable for the developing countries where cities grow without proper planning (Otterpohl *et al.* 1997; Shrestha *et al.* 2001). The requirement of a large area compared to the conventional systems has undermined the application of the CWs in peri-urban and urban areas where the land space is very often limited. In addition optimization of the design parameters facilitate treatment systems that are neither oversized nor fail to provide the desired water quality improvement (Buchberger & Shaw 1995). A major issue in adopting the CW technology is the choice of the wetland type. The two major types of CW are surface flow and subsurface flow wetlands (Kadlec & Knight 1996). Subsurface flow wetlands are preferred over surface flow wetlands in tropical and subtropical climates because the latter if not properly designed and operated, are potential breeding ground for mosquitoes (Kivaisi 2001). Depending upon the flow direction the subsurface wetlands are of two types: horizontal flow (HF) and vertical flow (VF). The HF wetlands have shown good performance in the removal of organic matter and were the first type of wetlands used in Europe to treat domestic wastewater (Vymazal 2005). In cold climates such as Norway, HF wetlands are preceded by a VF single pass biofilter (Jenssen *et al.* 2005) in order to remove Biochemical Oxygen Demand (BOD_5) during the winter. In Austria stringent discharge standards have led to the combination of a HF wetland followed by a VF wetland (Haberl *et al.* 1995) and this design is also used in several systems in Nepal (Laber *et al.* 1999). Many studies have been conducted on one or the other type of CW and most of these studies in Europe (Haberl *et al.* 1995; Cooper 1999), but there are very few studies comparing the performance of HF and VF wetlands

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under similar conditions (Breen & Chick 1995; Laber *et al.* 1999).

This paper compares the performance of HF and VF, planted and unplanted, wetlands in a pilot scale study. The suitability of HF and VF systems as single systems or in combination are evaluated against discharge standards for Nepal and Europe.

MATERIALS AND METHODS

The pilot scale subsurface flow CW system consists of two units of HF beds and two units of VF beds, each having a surface area of $6\ m \times 2\ m = 12\ m^2$ (Figure 1). The length of the HF unit is 3 times the unit width to promote plug flow conditions. The depth of the horizontal HF and VF beds are 0.6 m. The effective grain size (d_{10}) and uniformity coefficient (d_{60}/d_{10}) of the media were determined by sieve analysis (Table 1). The porosity of both the beds is 35%. The porosity of the media was determined by direct measurement by pouring 500 ml of representative sample into the cylinder containing 500 ml of water. The water level rising above the media was measured to calculate the pore volume.

One bed of each flow type was planted with *Phragmites karka* (local reed) and the other was left unplanted. *Phragmites karka* was chosen because it is very productive and a common wetland species in Nepal and has been used in all the CWs built in Nepal before 2001 (Shrestha *et al.* 2001). The pilot system was constructed at the premises of Guheswori Sewage Treatment Plant in Kathmandu and received wastewater from the grit chamber. The HF beds were continuously fed, whereas the VF bed received 6 doses/d using a pump and an overhead dosing/distribution tank. The daily total hydraulic load in both the beds were equal. In the HF beds the outlet pipes were adjusted so that saturated depth was 45 cm (3/4 of the total bed depth). The VF beds had free drainage. The beds had an initial period of 1 year to stabilize the vegetation prior to running the experiments reported herein. The vegetation was not harvested during the experimental run.

The first phase of the experiment studied the effect of different hydraulic loading rates (HLRs); 20 cm/d followed by 8 and 4 cm/d. Each loading rate was run for 21 d with sampling for the last 7 d. Five inlet and outlet samples were collected for each loading rate. All water samples were 24-hour composite samples. In the second phase of the experiment the loading rate was adjusted to 4 cm/d and run for 7 months to study the long term performance of the different beds. The average monthly temperature during the second phase experiment varied from 18 to 23 °C.

All samples were analyzed for Total Suspended Solids (TSS), BOD_5, Chemical Oxygen Demand (COD), Total Kjeldahl Nitrogen (TKN), Total Phosphorus (TP), Ammonia Nitrogen (NH_4-N) as per APHA (1985).

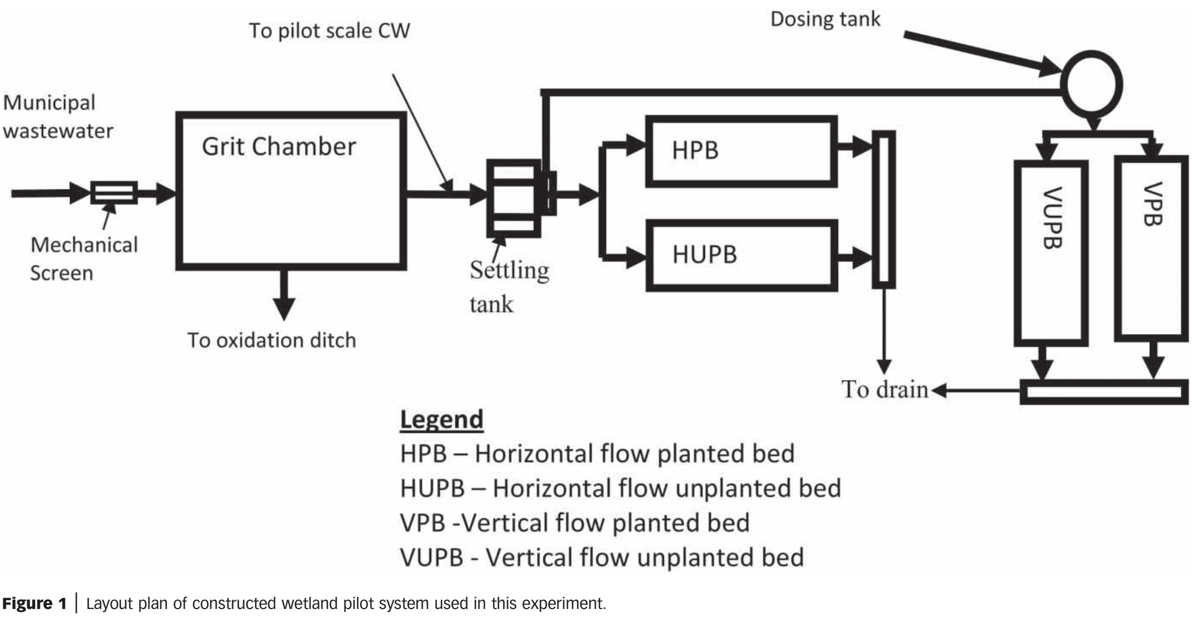

Figure 1 | Layout plan of constructed wetland pilot system used in this experiment.

Table 1 | The characteristics of the media

Parameter	HF beds	VF beds	Recommended values from literature
d_{10}	0.70	0.50	$d_{10} > 0.3$ mm (Vymazal *et al.* 1998)
d_{60}/d_{10}	4.5	4	$d_{60}/d_{10} < 5$ (CPCB 2003)

The horizontal beds had two sampling ports at a distance of 0.8 and 4.6 m from inlet (Figure 1). These ports are used to monitor Oxidation–Reduction Potential (ORP) and water level. ORP was measured in millivolts by using ORP electrode.

To compare the performance of beds at different HLRs a two-way analysis of variance (ANOVA) test was used. To compare the difference of the mean removal efficiency of the beds run at HLR of 4 cm/d a one-way ANOVA test at 95% confidence interval was used. A graphical comparison was expressed using box plots.

RESULTS

The effect of the hydraulic loading rate on removal efficiency

The treatment performance regarding TSS, BOD_5, COD, TKN, NH_4-N and TP at different HLRs is given in Table 2. The influent BOD_5 and COD varied during the experimental period ranging from 121 to 219 mg/l for BOD_5 and 240 to 395 mg/l for COD (Table 2). The percent removal increased with decrease in HLR for all beds and parameters, except for TP.

There was no significant difference ($p > 0.05$) in the removal of TSS and TP between any of the beds. Regarding removal of BOD_5, COD, TKN and NH_4-N and for all the tested loading rates the following results were obtained: (1) the HF planted bed performed significantly better ($p < 0.05$) than the HF unplanted bed and the VF planted bed showed significantly better removal than the VF unplanted bed, (2) the VF planted bed performed significantly better than the HF planted bed (Table 2).

The effluent discharge limit of 50 mg/l of BOD_5 and 100 mg/l of TSS (Nepalese standard) was achieved at 8 and 4 cm/d loading rate for all beds except the unplanted HF bed. The Norwegian standard of 20 mg/l of BOD_5 or 90% removal was met by the planted VF bed only. The P-removal was low for all beds.

ORP profile

The ORP profile along the HF and VF beds is shown in Figure 2. An increase of ORP from inlet to outlet is observed in all the beds at all loading rates. The effluent ORP in the

Table 2 | Average percentage removal of various parameters in the beds at three different hydraulic loading rates (mean, S.D, $n = 5$)

			% Removal			
Hydraulic load (cm/d)	**Parameter**	**Influent (mg/l)**	**HPB**	**HUPB**	**VPB**	**VUPB**
20	TSS	131.8 (±2)	64.1 (±2)	58.1 (±2.1)	76.3 (±2.8)	74.9 (±3.3)
	BOD_5	219.6 (±5)	50.1 (±9)	41.4 (±10.5)	67.1 (±8.4)	43.4 (±3.3)
	COD	375 (±43.7)	52.6 (±4.1)	39.4 (±22.6)	73.1 (±2.5)	58.3 (±6.8)
	TKN	59.5 (±9.5)	47.1 (±1.2)	21.7 (±1.98)	75.3 (±4.0)	44.9 (±2.9)
	NH_4-N	24 (±9.9)	37.9 (±0.9)	19.5 (±6.5)	67.9 (±4.6)	32.9 (±5.2)
	TP	3.9 (±0.4)	28.1 (±4.1)	21.7 (±2.03)	29.7 (±0.9)	18.3 (±2.5)
8	TSS	137.5 (±15)	77 (±2.6)	70.1 (±1.0)	89.2 (±1)	84.2 (±3.1)
	BOD_5	195.4 (±29)	67.6 (±4)	62 (±4.3)	81.2 (±5.4)	56.1 (±3.3)
	COD	395.5 (±66.6)	70 (±1.4)	58. 5 (±6.6)	84.8 (±4.7)	64.8 (±1.3)
	TKN	53.3 (±10.6)	49.9 (±0.5)	29.3 (±8.5)	73.5 (±6.5)	48.8 (±10.1)
	NH_4-N	26.8 (±4.4)	49.7 (±0)	19.6 (±4.3)	77.9 (±3.6)	40.1 (±7.8)
	TP	2.7 (±0.37)	32.7 (±2.8)	29.3 (±8.3)	30.1 (±7.6)	24.6 (±3.4)
4	TSS	148.3 (±19.4)	82.2 (±1.8)	77.4 (±0.97)	91.3 (±1)	86.8 (±0.7)
	BOD_5	121.8 (±33.4)	72.2 (±1.9)	61.1 (±8.4)	89.3 (±2.6)	60.2 (±3.3)
	COD	240.5 (±73.2)	72.1 (±1.9)	62.7 (±4.48)	89.8 (±1.9)	68.6 (±5.9)
	TKN	29.1 (±8)	51.3 (±1.8)	32.9 (±4.9)	78.8 (±4.1)	46.3 (±2.4)
	NH_4-N	19.4 (±2.1)	50.9 (±1)	22.4 (±1.1)	79.7 (±3.0)	36.2 (±9.2)
	TP	4.6 (±1.9)	33.8 (±2.7)	32.7 (±5)	47.7 (±2.4)	22.4 (±12.3)

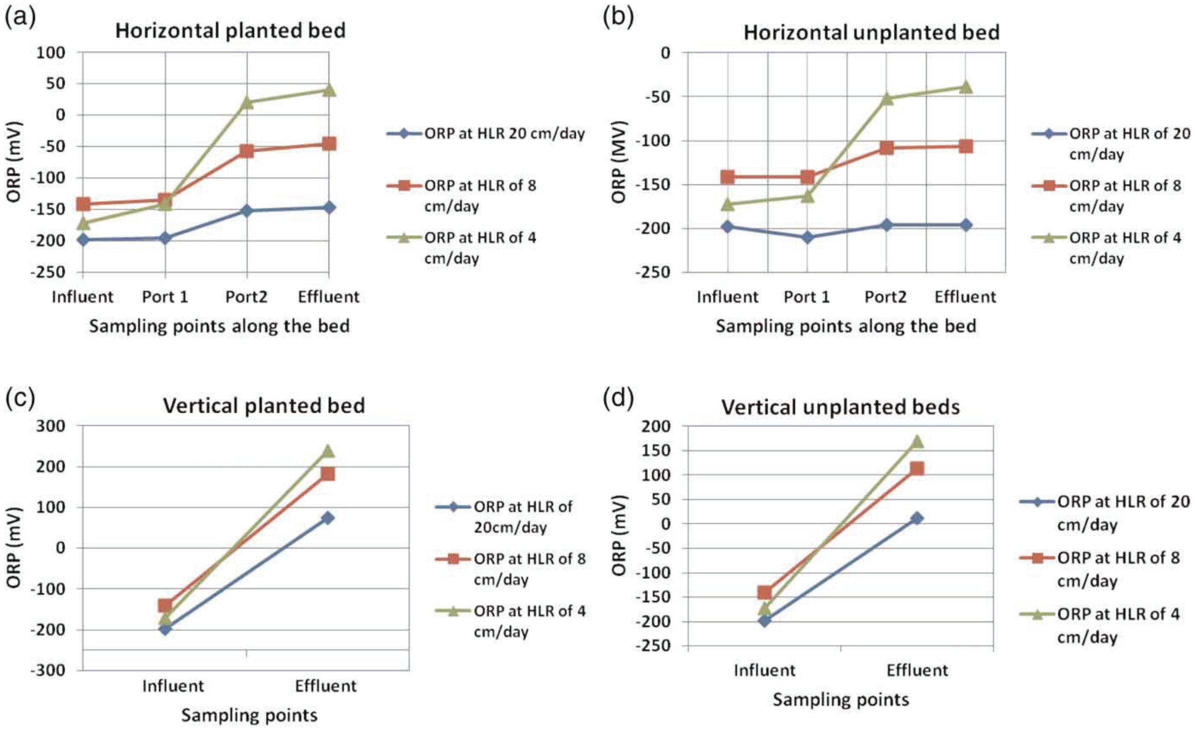

Figure 2 | Oxidation–reduction potential profile of horizontal and vertical planted and unplanted beds.

vertical beds are positive and higher than in the horizontal beds.

Long term performance at a hydraulic loading rate of 4 cm/d

Figure 3 shows the time dependent variation in the pollutant concentration and the statistical comparison of the different beds using box plots. The VF planted bed showed the best performance for all the tested parameters and for BOD_5, COD, TKN and NH_4 this bed was also significantly better than the other beds. In general the planted beds showed better performance than the unplanted beds, but for the HF beds it is only for TSS, BOD_5 and COD that the planted bed is significantly better than the unplanted bed. The HF planted bed is performing significantly better than the VF unplanted bed for BOD_5 and COD. For TSS the HF planted bed, VF unplanted and planted beds have near equal performance and are all significantly better than the HF unplanted bed. For NH_4 both VF beds had an effluent concentration significantly lower than the other beds. The NH_4 in the effluent from the planted and unplanted HF beds was better than the influent on the average, but not statistically different from influent showing a very limited NH_4 removal in both HF beds. For the phosphorus there is no significant difference between the inflow concentration and the outflow of all the beds VF and HF.

DISCUSSION

Performance of beds

The importance of plants in treatment wetlands has been subject to debate and in filter beds used in cold climate very high treatment performance is obtained without using plants in the HF beds (Jenssen *et al.* 2010). However, many studies point to plants enhancing the treatment performance (Brix 1994; Akratos & Tsihrintzis 2007). In this study the planted beds performed significantly better than the unplanted beds regarding BOD_5, COD, NH_4 and Kjeldahl-N removal. In the HF beds this can be understood because the plants provide oxygen, thus increasing the ORP (Figure 2). The plant roots will also provide more area for biofilm growth (Khatiwada & Polprasert 1999). The plants can also give support to a more diverse microbial community that can enhance treatment processes (Brix

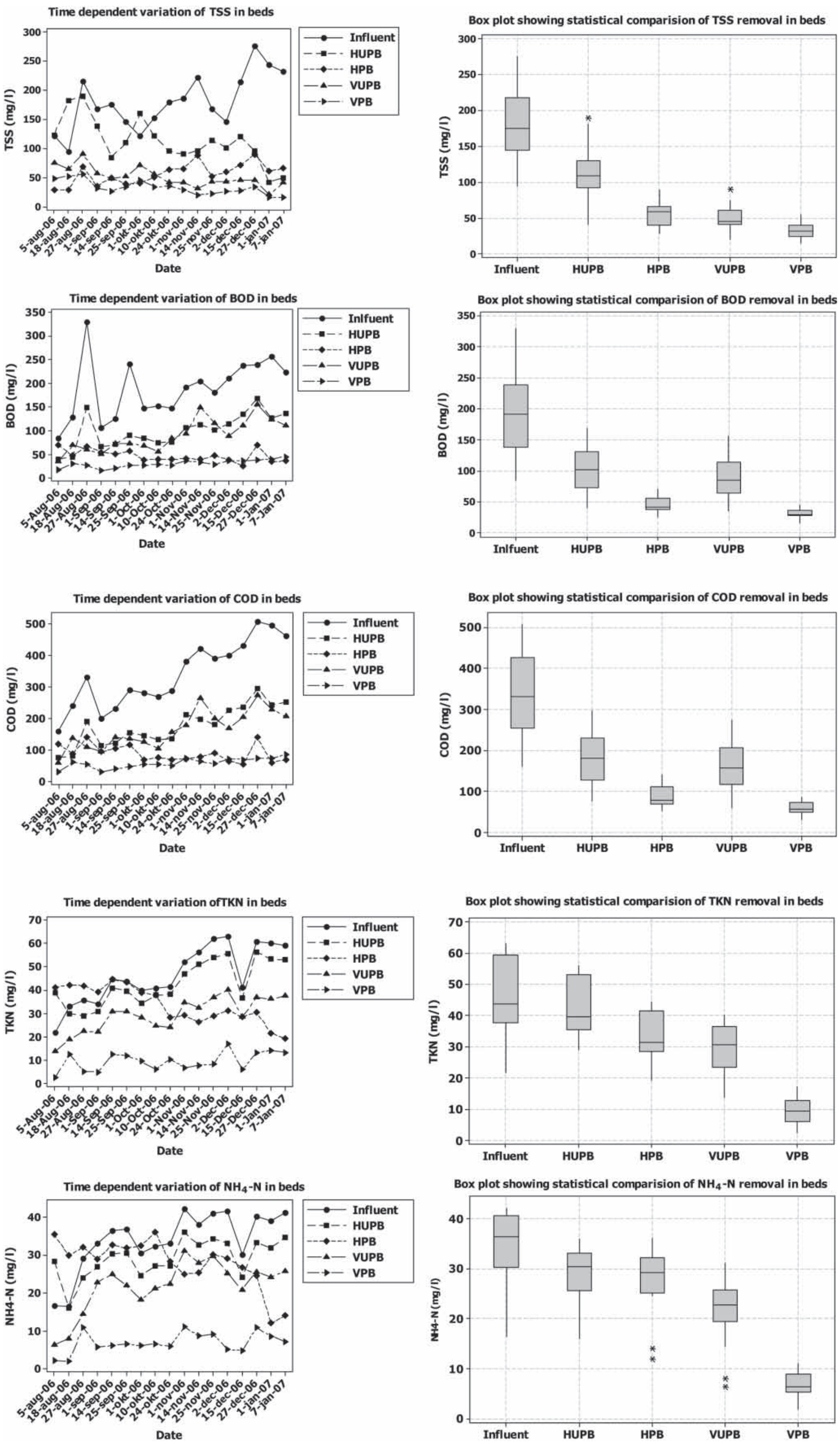

Figure 3 | Performance of pilot scale constructed wetland system.

1994). In HF beds the plants supply oxygen to the bed in the growing season, but not enough to completely remove all ammonia (Brix 1997; Okurut *et al.* 1999; Kuschk *et al.* 2003; Keffala & Ghrabi 2005). Zhu & Sikora (1995) found 30% higher nitrification in planted HF beds compared to unplanted. It could be argued that the uptake of nitrogen by plants can explain the larger nitrogen removal of the planted bed. However, nutrient uptake by plants in a wetland is low and not that sufficient to account for the better performance of the planted bed (Crites & Tchobanoglous 1992; Jenssen *et al.* 1993; Kantawanichkul *et al.* 2001).

The plants roots also alter the hydraulic conditions of a vegetated HF bed. A tracer study conducted for HF beds revealed that a HF planted bed had longer (average) HRT than a unplanted HF bed (Harne 2003). The longer retention time can be explained by a shift in pore size distribution towards smaller pores and thus a larger dispersion of the water molecules. Kadlec & Knight (1996) points out that greater dispersion can both prevent short-circuiting and increase the retention time. Suliman *et al.* (2007) who simulated flow in unplanted HF beds, pointed to preferential flow as a possible reason for reduced performance. Thus we can conclude that planting HF wetland beds increases removal of organic matter and enhances nitrogen transformations.

Planted sandfilters are termed VF wetlands. Due to the unsaturated VF the retention time in a VF wetland is much shorter than in a HF wetland provided the loading rate and porous media are similar. Despite a substantially shorter retention time the treatment performance of the VF beds is as good as or better than the HF beds, especially for the nitrogen transformations. Observing the ORP (Figure 2), this difference can be understood. The final effluent ORP value in both the planted VF bed (+239 mV) and unplanted VF bed (+169 mV) were positive and higher than the HF beds. This indicates that the VF beds are in a moderately oxidized state whereas the HF beds are in reduced state (Charpentier *et al.* 1998; Wiessner *et al.* 2005).

The intermittent dosing enhances the oxidative status of the filter as it promotes transfer of oxygen into the unsaturated zone (Bouma *et al.* 1983; Anderson *et al.* 1985; Brix 1994; Rousseau *et al.* 2005). In common terms this can be viewed as a pulse of liquid 'pushing' oxygen into the filter and at the same time 'sucking' air into the filter behind the pulse. Kadlec & Knight (1996) state that unsaturated flow enhances both organic degradation and nitrification. In this experiment 6 doses/d was used, but Emerick *et al.* (1997) points to 24–48 doses/d as more optimal for sandfilters indicating that the purification performance of the VF beds could be further enhanced.

The phosphorus removal was low in all beds both for the short term and long term experiments. However, P-removal is not emphasized by the Nepalese authorities and the media used in this experiment were not chosen for their ability to sorb P. The P-sorption capacity of the media used is documented as low by Laber *et al.* (1999) who used gravel from the same source. In order to really improve the river water in Nepal, lower discharge limits than the current ones (50 mg/l BOD and 100 mg/l of TSS) as well as restrictions on phosphorus discharge are probably necessary.

CONCLUSIONS

For BOD_5, COD, Kjeldahl-nitrogen and ammonia-nitrogen at loading rates 20, 8 and 4 cm/d, the HF planted bed performed significantly better ($p < 0.05$) than the HF unplanted bed, the VF planted bed showed significantly better removal than the VF unplanted bed, the VF planted bed performed significantly better than the HF planted bed. The superior performance of the VF beds, despite much shorter retention time than in the HF beds, can be explained by unsaturated flow conditions giving more air access as shown by the higher ORP in the VF beds. There was no significant difference ($p > 0.05$) in the removal of TSS and TP between any of the beds. This is due to low phosphorus sorption capacity of the porous media. As long as the current Nepalese discharge standard exists (50 mg/l BOD and 100 mg/l of TSS), the HF beds alone meet the Nepalese requirements. However, to really make an impact on river water quality in Nepal, stricter higher treatment goals are probably necessary and combined systems with both horizontal and VF beds as well as porous media with high phosphorus sorption capacity are recommended.

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Reed Beds for Sludge Dewatering and Stabilization

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Abstract

In urban and peri-urban areas of developing countries decentralized wastewater treatment using septic tanks as pretreatment is common. One challenge of decentralized wastewater treatment systems (DEWATS) is handling and utilization of the generated sludge. Sludge drying reed beds (SDRBs) are a robust method for dewatering and stabilization of sludge. Constructed wetlands (CWs) and SDRBs can be integrated to treat both wastewater and sludge. SDRBs require more area than most other sludge treatment options, but have low operational cost and energy requirements. The land area required for SDRB's can be optimized by the selection of an appropriate loading rate, sludge application frequency and resting phase. This paper gives a review regarding the use of SDRB's as well as presenting a pilot scale experiment comparing planted and unplanted sludge drying beds in Kathmandu. The planted beds showed a higher dewatering capability and higher reduction of volatile solids (VS). A short-term pilot-scale experiment can give valuable input to the design and operation of full-scale systems and for sub-tropical climate as that of Kathmandu Nepal, an initial sludge loading rate (SLR) of 100 kg total solids (TS)/m²/year is suggested with a gradual increase to up to 250 kg TS/m²/year.

Keywords

Sludge, Reeds, Dewatering, Evapotranspiration

1. Introduction

The management of sludge generated in decentralized wastewater treatment systems is a huge challenge in developing countries. In the Kathmandu valley there are about 68,000 septic tanks generating 75,000 m^3 of septic sludge (SS) annually [1]. Due to lack of sludge treatment facilities most of the sludge is disposed of untreated. A small fraction is used in agriculture, but the rest is illegally dumped into rivers, drains or open spaces. Sludge

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dewatering reed beds (SDRBs) present an exciting sludge treatment option for communities looking for alternatives to conventional sludge dewatering systems and can be used as a sludge treatment method at small as well as at large centralized treatment plants [2] [3]. When properly sized and constructed, SRDBs are effective for increasing the dry matter content of the sludge, thus reducing total sludge volume, while at the same time producing a safe, high quality end-product, which is often suitable for application to green areas or arable land [4] [5]. A properly constructed sludge dewatering reed bed system requires little maintenance, uses little to no electricity and can be loaded for 8 - 10 years before the sludge must be removed [6]. Although the evidence is limited planted sludge drying beds seem superior to unplanted and quicker dewatering, enhanced mineralization of residual solids, possibility of operating the beds at higher loading rates and longer life span of the bed is pointed out by several authors [7]-[9]. However, there are few studies that compare planted and unplanted beds under similar operating conditions [3] [10]-[12]. The challenges of using SRDB's are: long startup time due to conditioning of the reeds, sensitivity to the loading regime, wilting of plants and lack of design criteria for different sludge types and climate zones [2] [13]. The SLR is the main design parameter for sizing of the SDRBs, but the hydraulic loading rate (HLR) can also be used [6]. In the literature SLR vary from 17 - 28 kg TS/m^2/yr for cold climate up to 250 TS/m^2/yr in warm climate [13] [14]. For cold temperate climate and sludge from activated sludge plants with co-precipitation using iron or aluminum coagulants a SLR rate of 50 - 60 kg TS/m^2/yr and a corresponding per capita area requirement of 0.3 - 0.6 m^2 has been used with good results [6]. In Norway and central Sweden, because of the cold climate, short growing season and freezing during winter a minimum area of 0.6 m^2/person equivalent and conservative SLR of 17 - 28 kg TS/m^2/yr, has been suggested [14]. In cold climates natural freezing and thawing processes aid the dewatering process [15] [16], but the systems have to be designed to accommodate the accumulating frozen sludge during the winter [14]. Freezing separates the solid and liquid fraction by the process of ice crystal formation. During the summer the ice crystals melt away leaving the consolidated and dewatered sludge [17]. Short dosing times and long resting periods have shown best results in cold climate [2] [18]. A preliminary recommendation for the design and mode of operation for tropical climate has been suggested by Koottatep [19]. A SLR of 250 kg TS/m^2/yr and a loading frequency of once a week produced residual solids with TS content of 30% to 60% [20]. The equivalent per capita land requirement was 0.03 m^2/p.e., an order of magnitude lower than in colder climate. In Ghana a SLR of 100 - 200 kg TS/m^2/year has been used to treat fecal sludge in unplanted sludge drying beds. These beds were able to remove the Helminths egg by 100% and the organic matter and solids concentration reduction was more than 80% [21]. In Yemen average dry solid content of 25% was achieved at a SLR of 178 - 283 kg TS/m^2/year and drying time of 7 - 12 days [22]. Well documented design criteria exists for unplanted sludge drying beds [23] [24]. SRDBs follow this design regarding the construction of the bed with the exception that sometimes a more fine-grained layer, suited to nurse plants, is added at the top [25]. General design and operational criteria for SDRBs are given for cool temperate climate and tropical climate from the study of Nielsen [8] and Koottatep [19] respectively. Design and operating guidelines are not established for sub-tropical climate as that of Kathmandu. Dewatering efficiency of the beds depends on the sludge type, sludge quality and local climatic condition [26]. A short-term de-watering study can be helpful to provide rational information regarding parameters such as loading frequency, resting period and life expectancy of the bed [27]. The aim of the short-term study presented herein is to examine the dewatering performance of planted and unplanted drying beds treating septic tank sludge. Together with a literature review this study serves as input for suggesting design and operational parameters for SRDBs for sub-tropical climate as that of Kathmandu Nepal.

2. Materials and Method

2.1. Pilot scale Sludge Drying Beds

The pilot scale sludge drying beds are shown in **Figure 1**. The units consist of three identical beds with surface area 1.5 m × 0.7 m and a depth of 1m. Two beds were planted with *Phragmites Karkaa* (local reed) and one was left unplanted. The sequence and size of the filter media and the drainage layer were adopted from Koottatep [19] and is shown in **Figure 1**. A 50 cm freeboard above the surface layer was provided for sludge accumulation. The bottom of the bed was sealed using a plastic membrane. The drainage pipe was connected to a vertical pipe at one end to assist aeration from below. The water percolating from the bed was collected and measured. Scales were placed on the beds to measure the sludge accumulation in the beds. The change in sludge depth was recorded at short time intervals for first 24 hours and then at the end of the each resting period of one week.

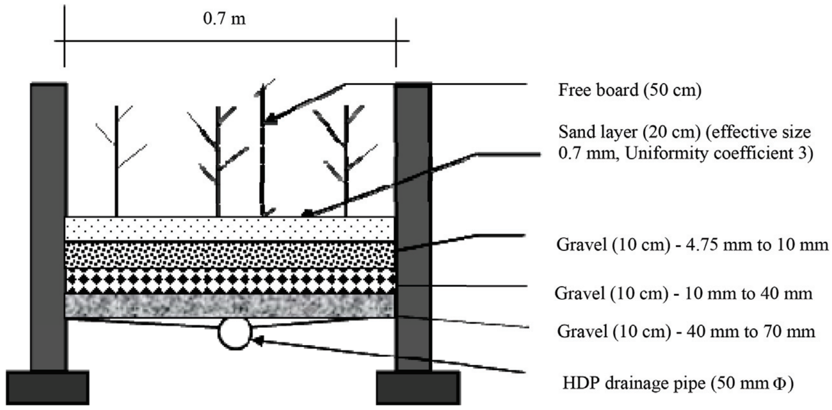

Figure 1. Cross sectional view of pilot scale sludge drying reed bed.

The beds had a plastic superstructure that allowed aeration, but prevented direct rainfall onto the beds. Prior to the actual experiment, the planted beds were conditioned by planting reeds (4 plants/m^2) and loaded with wastewater for a period of two months. The reeds were then well established and had reached a height of 90 cm prior to the sludge application.

The beds were loaded with a SLR of 250 kg TS/m^2/yr (Planted 1) and 100 kg TS/m^2/yr (Planted 2). The sludge was obtained from a private company cleaning septic tanks in Kathmandu. The sample from raw sludge was analysed before each loading cycle. The TS concentration of the septic tank sludge used for dewatering was different in each loading cycle. Therefore, to maintain the constant SLR for each loading cycle the depth of application varied. The average depth of application for SLR of 100 kg/TS/m^2 was 4.2 cm and for 250 kg TS /m^2/yr 12 cm. The sludge was fed every 7th day with 6 days resting between applications as suggested by Koottatep [28]. The duration of sludge loading and monitoring was 2 months and was conducted from December through January. The average daily high and daily low temperature during the experimental period was 8˚C and 20˚C respectively. Composite samples of the stabilizing sludge were collected at the end of each loading cycle by mixing equal portions of sample from 4 quadrants of the beds. The sludge was analysed for moisture content (MC), total solids (TS), VS, total Kjeldahl nitrogen (TKN) and total phosphorus (TP) using standard method of analysis [29].

Descriptive statistics (means, standard deviation) of the variables were examined. One-way Analysis of Variance (ANOVA) test at 95% confidence was used to compare the performance of the planted and unplanted beds and at different SLR.

2.2. Model for Estimation of Drying Period

The mass balances of all incoming and outgoing moisture can be used to compute the drying time [24] (**Figure 2** and Equation (1)). The drying time is a key operational parameter, because it gives the time between the loading cycles of the beds. The drying time required to achieve the desired dry solid content can be computed using Equation (1)

$$t = (1 - f_i) q_i + (1 - f_r) q_r - q_d / (f_e E_w) \qquad (1)$$

where, t = required drying time, days; q_i = initial water content of sludge, kg/m^2; q_r = moisture received by precipitation, kg/m^2; q_d = moisture remaining in the dried sludge, kg/m^2, f_i and f_r are fraction q_i and q_r that is drained by gravity; f_e = reduction factor to account for reduced evaporation rate from a sludge surface; E_w = pan evaporation rate from a free water surface in kg/m^2/day. The monthly average of rainfall, temperature and class A pan evaporation in Kathmandu valley was obtained from Nayava [30].

3. Result and Discussion

3.1. Sludge Characteristics

Sludge may vary in composition depending on treatment system. When designing SRDBs it can be assumed

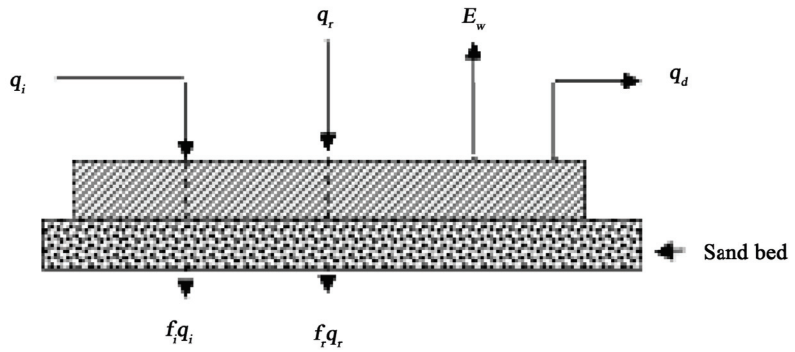

Figure 2. Conceptual illustration for computation of drying time in sludge drying reed bed [24].

that the characteristics of the sludge is important. In **Table 1** the septic tank sludge used in this study, is compared to other studies. Compared to sludge from activated sludge treatment plants the septic tank sludge is highly variable and in general has a higher TS content. The quality of septic sludge is affected by several factors as emptying intervals, emptying technology and design of the septic tank [31]. High concentration of solids indicates that the sludge has had a long storage time before being pumped out [32]. The emptying interval of septic tanks in Kathmandu is 3 to 3.5 years [1]. In a decentralized sludge treatment facility where the incoming sludge is expected to vary considerably, homogeneous mixing of sludge in a buffer tank is recommended, as this will equalize the TS concentration of the sludge [28].

Although it could be expected that the sludge from septic tanks also have a higher content of VS due to the long retention time, the content of VS is generally a bit lower, but not significantly different from the activated sludge systems. However, since the activated sludge is younger, the readily degradable part of the VS can be expected to be higher in the activated sludge, thus for the same amount of TS load the mineralization rate of organic matter can be expected to be higher for the plants receiving activated sludge. This implies that for the same climatic conditions higher organic and HLR should be possible for activated sludge than for septic tank sludge.

The TKN is higher in the septic sludge than for the activated sludge (**Table 1**). This difference is not easy to explain, but may be because up to 90% of the nitrogen in raw wastewater is in the ammonia state [34] and thus escapes with the liquid phase. Septic tank sludge has a higher TS content and more nitrogen is potentially held back as organically bound nitrogen. The phosphorus concentration is much higher in the activated sludge. The latter can be explained by chemical co-precipitation of phosphorus using aluminum or iron coagulants in the activated sludge plants (**Table 1**). Based on the parameters, TS, VS and TKN, and the limited number of investigations currently available it is difficult to generate a model predicting loading rates versus climate based on sludge quality, thus design of systems and selection of loading regime has to be based on empirical data.

3.2. Dewatering Rate and Drying Period

In order to operate SRDBs, the best possible dosing and drying cycles has to be determined as this can greatly influence the long-term capacity of the beds [2] [6] [13]. The fraction of the initial water content of the sludge that has percolated and evapotranspired in our pilot study is shown in **Table 2**. In all the beds, the percolation started between 10 to 20 minutes after the sludge loading and about 50% of the water content in the applied sludge drained from the filter within the first five hours. More than 97% of the percolation fraction emerged within the two first days. There was no significant difference ($P = 0.017$) in the fraction of water drained by gravity (f_i) between the planted and unplanted beds (**Table 2**).

The liquid mass balance of the experiment showed that 25% - 33% of the water content of the sludge is lost through evaporation or evapotranspiration and 58% - 63% was lost by gravity drainage. This is similar to results reported from a one-year long mass balance study of SDRBs in which around 35% of the water was accounted for by evapotranspiration and about 65% for gravity drainage [28]. Another short-term study [27] found that 60% to 70% of the water content in the sludge is typically free water that drains out by gravity. Comparing the planted and unplanted beds at SLR of 100 kg $TS/m^2/yr$ the overall dewatering efficiency of the planted bed is

Table 1. Characteristics of raw sludge (Range/Average).

Parameter	Septic tank sludge (This study)	Septic tank sludge Kathmandu valley[1]	Septic tank sludge Bangkok[2]	Activated sludge with co-precipitation[3]
TS, mg/l	24,365 - 48,200 Avg. 30,160 (3.01%)	27,000 (mean of 42 samples) (2.7%)	2200 - 67,200 Avg. 19,000 (1.9%)	5000 - 13,000
VS (% of TS)	49 - 65 Avg. 58	65 (mean of 28 samples)	40 - 78 Avg.71	60 - 70
TKN mg/l (avg.)	1273 (1021 - 1500)	-	1000 (300 - 5000)	160
TP, mg/l	Avg. 23.4	-	-	71
SLR kg TS/m^2/year	100 and 250	250	140 - 360	50 - 70

[1][1], [2][28], [3][33].

Table 2. Fraction of water lost by percolation and evaporation/evapotranspiration and remaining TS content in the dewatered sludge (Average of 8 weekly samples).

	Fraction drained by gravity	Evaporated/Evapotranspired fraction
Planted (100 kg/m^2/year)	0.60 (±0.05)	0.33 (±0.05)
Planted 250 kg/m^2/year	0.58 (±0.01)	0.29 (±0.02)
Unplanted 100 kg/m^2/year	0.63 (±0.02)	0.25 (±0.02)

higher than the unplanted bed (see also **Table 3**) due to higher evaporation fraction in the planted bed. As the beds still were young during this study a larger effect of evapotranspiration can be expected when the roots are fully developed [32]. In SDRBs with matured plants, evapotranspiration of up to 64% is reported in a *Phragmites* stand [15]. In cold climate, where it takes a couple of years for the plants to mature, the SLR is gradually increased over the first years [8].

Drying time to achieve a final TS content of 40% for the planted beds have been estimated using the "f_i" value in **Table 2** and monthly climatic data for Kathmandu. Wetland evapotranspiration (ET) is approximated as about 0.7 - 0.85 times the class A pan evaporation [15]. Initial TS content of the raw septic tank sludge is assumed to be 4% (**Table 3**). The typical values of the coefficients f_r = 0.43 and f_e = 0.78 for anaerobically digested sludge have been adopted from [24]. The drying time estimated for each month is presented in **Figure 3**.

In this experiment the beds were covered, but the model (Equation (1)) allows prediction of the response in beds that are uncovered (open) and receives precipitation. For open beds, the estimated drying period required to achieve 30% TS content for SDRB varies from 13 days to 37 days for SLR of 250 kg TS/m^2/year and 6 days to 9 days for SLR of 100 kg TS/m^2/year. Longer drying time is required from June to September when precipitation is high and exceeds the evaporation. If the beds are covered the drying time in the months from June to September is significantly shortened. The rest of the year has more evaporation than precipitation and therefore the required drying times become shorter. In SDRBs with matured plants the movement of the plants helps to make cracks on the surface of the residual sludge layer providing channels for the rainfall to pass through the beds [27]. This will also help oxygen diffusion into the residual sludge promoting aerobic mineralization of the sludge [6]. Longer drying period will increase the TS concentration of the sludge, reduce the accumulated residual sludge volume, increase the life expectancy of the bed and, hence, reduce the restoration cost of bed [27]. However, in warm climates longer resting time could lead to plants suffering from aridity and consequently wilting [28]. When the drying periods are longer additional beds will be required to treat same amount of sludge, this will consequently increase the required bed surface area. Therefore, appropriate selection of the drying period is important for optimal performance as well as size of the system. A final total solid content of 40% - 50% can be theoretically achieved in sludge drying reed beds [35]. Correct loading and resting strategies based on local climatic conditions will maximize the dryness of the final residual sludge [6] [8] [14] [32]. In cold climate where evaporation and evapotranspiration is low it is recommended that beds are rapidly loaded within a few days and then allowed to rest for 30 to 50 days [8] [12] [26] [36].

3.3. Drying Bed Performance for TS, TP and TKN Removal

The TS, VS, TKN and TP contents of the dewatered sludge from the experimental beds after eight feeding and

Table 3. Average TS, VS, TKN and TP content measured at the end of each drying cycle (total 8 cycles).

Parameter	Raw Sludge*	Sampled after one week of resting period		
		Planted 1* (250 kg TS/m²/year)	Planted 2* (100 kg TS/m²/year)	Unplanted* (100 kg TS/m²/year)
TS (%)	4 (±0.48)	20.94 (±2.6)	35 (±6.32)	24 (±2.00)
VS (% of TS)	64 (±5.37)	34 (±1)	27 (±5.57)	43 (±1)
TKN (% of TS)	3.2 (±0.971)	2.83 (±0.304)	2.24 (±0.60)	2.82 (±1.315)
TP (% TS)	0.05 (±0.00)	0.06 (±0.01)	0.05 (±0.01)	0.05 (±0.005)
Initial volume (lit)		122	52	52
Final volume (lit)**	-	10	3	6
Volume reduction (%)		92	96	89

*Mean based on analysis of 8 set of composite samples. **Final volume calculated using expression $V_1/V_2 = P_2/P_1$ [23]; where V_1 and V_2 are the initial and final volume of the sludge and P_1 and P_2 are the initial and final solid concentration of the sludge.

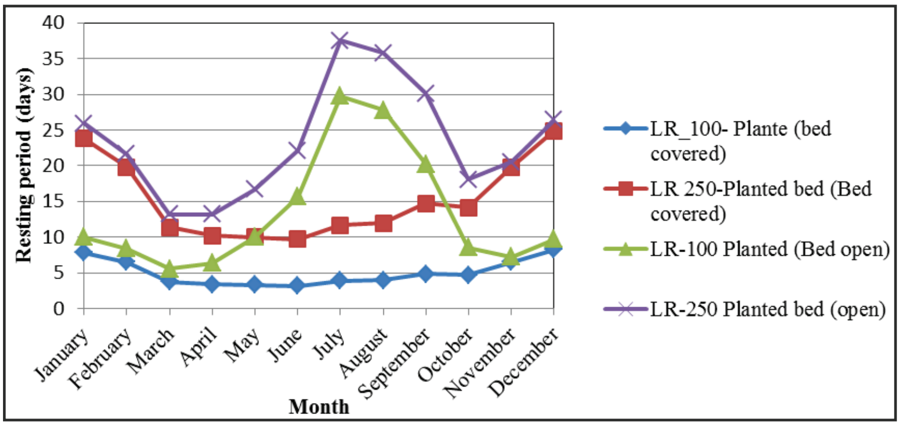

Figure 3. Estimated drying time for sludge treatment beds in Kathmandu for two different loading rates and covered/uncovered (open to precipitation) beds.

resting cycles are presented in **Table 3**. Comparison of planted and unplanted beds at SLR of 100 kg TS/m²/y shows that planted beds produce sludge with a higher TS concentration than the unplanted beds. The planted bed loaded at 250 kg TS/m²/y only had slightly lower TS than the unplanted bed loaded at 100 kg TS/m²/y. Both planted beds show a higher VS reduction something that indicates better conditions for degradation of organic matter and thus higher mineralization rate in the planted beds.

There was slight reduction in TKN in the sludge after 2 months of operation. There was no significant difference in nitrogen mineralization in the planted and unplanted beds, but the planted bed loaded at 100 kg TS/m²/year had the lowest TKN content and the planted bed loaded at 250 kg TS/m²/year only had a slightly higher content than the unplanted bed. This indicates that nitrogen mineralization is more efficient in the planted beds. The reduction in TKN-content in the sludge is due to ammonia volatilization, plant uptake and nitrification reactions [2]. The planted beds also show a higher volume reduction, which indicates a both a higher mineralization rate and better dewatering capabilities.

The sludge depth is rapidly reduced within first 24 hrs. of sludge application due to rapid initial free drainage. After eight weeks of applications the depths recorded in Planted Bed 1 (250 kg TS/m²/yr), Planted Bed 2 (100 kg TS/m²/yr) and unplanted bed 3 (100 kg TS/m²/yr) was 0.9 cm, 0.3 cm and 0.4 cm respectively. If sludge thickness (h_t) and resting time between each loading is known the life expectancy (T) of the bed can be calculated by using following expression [27].

$$T = \left(H/h_t\right) * t \tag{2}$$

where, H is freeboard for long-term sludge storage. Assuming a freeboard of 50 cm the life expectancy calculated are: 13 months for Planted Bed 1 (SLR 250 kg TS/m^2/yr), 38 months for Planted Bed 2 (SLR 100 kg TS/m^2/yr) and 29 months for Unplanted Bed (SLR 100 kg TS/m^2/yr). The simple model (2) does not account for mineralization and, thus underestimates the life expectancy. The drying efficiency of the bed affects the life expectancy of the system. In Denmark the operational cycle of the SDRB's in an average is 10 years with a final dry solid content of about 30% [2].

3.4. Comparison with Previous Studies and General Design Considerations

In **Table 4**, this study is compared with other case studies of SDRB's with respect to loading regime (loading rate and dosing/resting cycles) and final TS content. It is apparent from the table that the loading rates used in warmer climate are higher than in colder. Especially when considering that the data from the warmer climate is for systems receiving septic tank sludge that has a higher TS content than activated sludge. However, the empirical database is limited (**Table 4**) and does not provide data by which the loading regime can be modeled based on climate and sludge type. Due to lower evapotranspiration in cold climate rapid loading and prolong resting have been suggested [8]. In warmer climates a shorter resting period of a week or less seems to significantly increase the TS content in the sludge. In places of high precipitation partially covering of the bed would shorten the drying period. A prolonged resting period can be counterproductive in warm climate because of the possibility of wilting of plants. In both cold and warm climates a final sludge residue of at least 25% to 40% (**Table 4**) is achievable through proper operation of the bed. The experience in cold climate has shown that for the same TS load the rate of accumulation of activated sludge is higher than for septic sludge [37]. Therefore the desludging frequency of SDRB's treating activated will be higher.

The majority of the water content in the sludge is lost during normal operation period (loading and resting sessions) and the final resting phase does not significantly increase the dry matter content in the residual sludge [37]. However, the final resting period improves stability and the hygienic quality of sludge that is necessary for safe application in agricultural land. In Denmark 6 - 9 months of rest after the final application produce biosolids that meet the hygienic quality set by Danish guidelines [2]. The WHO guidelines [38] suggests more than 1 year resting in climate as in Kathmandu for fecal sludge. Due to limited data further experiment should be conducted in order to determine the optimal resting period for sub tropical climate.

With regard to composition of the filter design all the systems (**Table 4**) have top layer of sand from 15 cm to 25 cm thick over a gravel layer of 20 - 35 cm. The effective size of sand range from 0.3 mm to 1 mm and the gravel size range from 1cm to 4 cm. The systems in Denmark [26] use an additional 15 cm layer of silty loam above the sand. This layer promotes the initial growth of the reeds due to a large water holding capacity, but may induce more rapid clogging.

3.5. Conclusions

Sludge drying reed beds (SRDBs) are a technically simple method providing dewatering and sludge treatment

Table 4. Comparison of design and operation of sludge drying reed beds in different climatic zones.

Climatic Zone	Sludge type	SLR kg /m^2/year	Feeding and resting strategy	TS content of dewatered sludge (%)	Reference
Temperate	AS	50 - 60	3 days loading and 30 to 50 days rest period	≥30	[26]
	SS	25 - 30	20 days rest period in between loading	70%	[39]
	SS	46	1 week loading and 5 weeks rest period	38%	[12]
Continental	AS	85 - 90	1 week loading and 3 weeks resting in winter and 1 to 2 weeks resting in summer	50% - 64%	[35]
	AS	50 - 60	2 days loading and 10 days rest period	26% - 30%	[40]
Tropical	SS	178 - 283	7 - 12 days resting period between each dosing	40%	[20]
	SS	100 - 200	7 days of resting between each loading	≥30%	[4]
Humid sub-tropical climate	SS	100	7 days of resting between each loading	35 (±6.32)	This study

capabilities comparable or exceeding most other sludge handling methods. The method can handle any type of sludge, but if the sludge quality varies, mixing of the sludge prior to application in SRDBs is recommended. This experiment showed that:

- The overall dewatering efficiency of the planted bed was higher than the unplanted bed due to higher evaporation fraction in the planted bed. As the beds still were young during this study a larger effect of evapotranspiration can be expected when the roots are fully developed.
- The planted beds had a higher VS reduction than unplanted beds indicating better conditions for degradation of organic matter and thus higher mineralization rate in the planted beds.
- A short-term pilot-scale experiment can give valuable input to the design and operation of full-scale systems.

Based on the limited number of investigations currently available it is difficult to generate a model predicting loading rates versus climate based on sludge quality parameters, thus design of systems and selection of loading regime has to be based on empirical data.

Based upon this short-term experiment and literature data, an initial sludge loading (SLR) rate of 100 kg TS/m^2/year is suggested for the sub-tropical climate as that of Kathmandu. However, after one year of operation, when the plants are matured, the SLR can be gradually increased up to 250 kg TS/m^2/year. A minimum resting period of one week between loadings and final resting phase of one year can ensure both adequate dewatering as well as a hygienized and stable end products. This study as well as other studies has shown that in warm climates with high annual precipitation partially covered beds will reduce the drying time and consequently require less area than open beds.

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Paper IV

Manoj K. Pandey, Sushil K. Shrestha, Petter D. Jenssen & Jan Mulder. Assessing organic matter and nutrient removal in horizontal subsurface flow constructed wetlands using first order reaction rate models.

Manuscript

Assessing Organic Matter and Nutrient Removal in Horizontal Subsurface Flow Constructed Wetlands using first order reaction rate models

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Abstract

In this study hydraulic conditions and reaction kinetics for organic matter and nutrient removal of a pilot-scale and full-scale horizontal subsurface flow constructed wetland in Nepal were evaluated. The pilot- scale wetland, consisted of two identical beds one planted with *Phragmitis Karka* and the other without plants, and was operated at three hydraulic loading rates; 0.2 m/day, 0.08 m/day and 0.04 m/day. The influent and effluent concentrations were analyzed for biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), ammonium (NH_4-N), and total phosphorus (TP). The full-scale, planted horizontal subsurface flow wetland was run at two hydraulic loading rates (0.02 and 0.05 m/day) and the longitudinal concentration profiles of COD, NH_4-N and TP were measured from sixteen sampling ports. A tracer study was performed in the pilot scale wetland. The experimental data were fitted with the modified first order plug flow model (k-C*) and tank in series model (p-k-C*) to obtain the areal rate constant (k_a) and the background concentration C*. The model predictions showed that the treatment was suboptimal probably due to preferential flow paths. The presence of plants increased the dispersion. For BOD and TSS the models gave a good fit to the experimental data, however, the models were not suited to describe the long term phosphorus removal in wetlands. The rate constants (k_a) determined for organic matter (BOD, COD) removal from this study were low compared to other studies and indicate that the higher temperature of Kathmandu is not reflected by (k_a).

Introduction

Horizontal subsurface flow wetlands were the first type of constructed wetlands (CW's) treating wastewater in Europe and America (Kadlec 2009; Vymazal 2005; Cooper 2009). The pollutant removal in the constructed wetlands occurs as a result of a complex interaction between the water column, wetland media and wetland vegetation (Kadlec and Wallace 2009). Wetlands are heavily influenced by environmental factors. Although wetlands are technically simple the internal reactions and removal processes are complex and this makes them more complicated to model than many other biological reactors like activated sludge systems (Kadlec and Wallace 2009). The pollutant removal efficiencies of constructed wetland are temperature dependent (Akratos and Tsihrintzis 2007). In warmer climate, the process kinetics for organic matter degradation and nutrient removal is faster and therefore the design parameters of CW's should deviate from those in colder climate. Studies on a large number of CW's operating in cold and temperate climates have optimized the design parameters (Reed 1993; Brix and Arias 2005; Cooper 2009; Kadlec 2009; Kadlec and Wallace 2009; Vymazal 2011). However, unlike in temperate and cold climate the number of studies regarding constructed wetlands in warmer climate are limited (Gopal 1999; Haberl 1999; Shrestha et al. 2001; Kantawanichkul and Wannasri 2013;) and there is a need for further assessment of design parameters for warm climates. Design guidelines in Europe and America (Reed 1993; Brix and Arias 2005), especially for single households, are based on "crude rules of thumb" (Rousseau et al. 2004; Cooper 2009). These rules often express design in area per person and may give oversized systems particularly if these rules, derived in cold climate, are used in warmer areas. For urban settings where land prices are high and land availability is limited design criteria tailored to the local situation are required to produce an optimum system.

The most common approach for design of CW´s, is based on first order kinetics to predict removal of organic matter, expressed as biochemical oxygen demand (BOD) or chemical oxygen demand (COD), and nutrients as ammonium (NH_4-N) and phosphorus (Wynn and Lieher 2001; Kadlec and Knight 1996; Kadlec 1994). However, the first order reaction kinetics is not necessarily able to account for the complex reactions or mechanisms prevailing within the wetland and, thus, describes the wetland as a "black box". Since the reaction kinetics are temperature dependent it has not been possible to define a global rate constant for organic matter and nutrient removal to be used with this simple model (Tchobanoglous et al. 2000; Stein et al. 2006). The reported values of the areal rate constant (k_a in m/year) from a large number of horizontal subsurface flow wetlands in cold and temperate climates vary from 30.17 m/yr (Sun and Cooper 2008) to 50.96 m/yr (Vymazal 2011).

The modified first order model (k-C*) (Kadlec 2000) takes into consideration that a wetland, as a natural dynamic system, produces and discharges some organic matter and nutrients that is not directly derived from wastewater. Thus, a non-zero background effluent concentration (C*) is introduced. C* is normally unknown and therefore used as a free fitting parameter when calibrating the models.

Although, first order models commonly assume plug flow, Kadlec (2000) found the non-ideal flow pattern, that is normally the case in a CW, could be better described by a tank-in-series model (p-k-C* model). The p-k-C* model is also based on the simple first order reaction kinetics, but this model includes a number of completely stirred tank reactors (CSTR) in series. The number of CSTR can be determined from tracer studies (Kadlec and Knight 1996). The reported literature value of CSTR obtained from tracer studies of operating wetland ranges from 2.5 to 34 with a mean value of 11 (Kadlec and Wallace 2009). However, both the k-C* and p-k-C* models consider the CW as a black box and therefore the internal pollutant removal kinetics are unknown (Rousseau et al. 2004). Insight in the internal processes in CW's may be obtained through mechanistic compartmental models (Wynn and Liehr 2001, Langergraber 2007). These models have numerous empirical parameters that make them difficult to use for general design purposes (Rousseau et al. 2004).

In this study the hydraulic and kinetic behavior of a pilot- scale and a full scale constructed wetland has been evaluated with the objective to determine design parameters for organic matter (BOD) and nutrient removal applicable for subtropical climate conditions as in Kathmandu. The estimation of model parameters have been carried out using two model approaches; 1) modified first order plug flow (k-C*) and, 2) the tank in series model (p-k-C*). The k-C* model is commonly used and most of the rate constants found in the literature are derived using this model. The p-k-C* model was included because it may better represents the non-ideal flow conditions in a wetland.

Materials and Methods

The pilot- scale unit

The pilot-scale subsurface flow constructed wetland (CW) system consists of two units of horizontal flow (HF) beds. One bed was planted with *Phragmites Karkaa* (local reed) and the other is left unplanted. Each bed has a surface area of (6 m x 2 m=12 m^2) (Figure 1). The length was made 3 times the unit width to promote plug flow conditions. The depth of the media packed in the beds are 0.6 m. The effective grain size (d_{10}) of 0.70 mm and uniformity coefficient (d_{60}/d_{10}) of 4.5 (dimensionless) of the media was determined by sieve analysis. The porosity of both beds was 35%. A v-notch weir, made of acrylic sheet, was installed at the inlet side of the beds to measure the inlet flow. The beds were continuously fed. The outlet pipes were adjusted so that the saturated depth was 45 cm (3/4 of the total bed depth). The beds had an initial period of one year to stabilize the vegetation prior to running the experiments reported herein. The vegetation was not harvested during the experimental period.

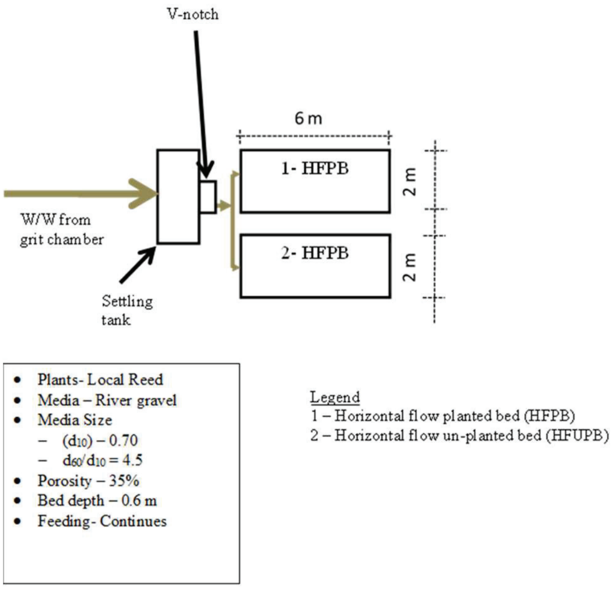


Figure 1: The constructed wetland pilot system used in this experiment.

The hydraulic loading rate (q) of the beds were set to 0.2 m/day, 0.04 m/day and 0.08 m/day and. The corresponding hydraulic retention time (HRT) was 0.78 day, 1.9 day and 3.9 day respectively. Each loading rate was run for 21 days with sampling the last 7 days. Five inlet and outlet samples were collected for each loading rate. All water samples were 24-hour composite samples. The 0.04

m/day was run for seven months to study the long-term performance of the planted and unplanted beds. The average monthly temperature during the experiment varied from 18ºC to 24ºC.

Tracer studies

Tracer studies were performed in both beds using sodium chloride (NaCl). The tracer test was performed only at a hydraulic loading rate 0.27 m/day. After the injection of the tracer solution, the conductivity of the effluent was measured every half hour. From the tracer data the method suggested by Levenspiel (2012) was used to calculate the mean or actual retention time (t_{actual}) hr; variance (square of standard deviation) of the time-concentration curve (σ_t^2), day^2; and dimensionless variance (σ^2). The dispersion number (D) that defines the degree of non-ideal flow conditions was obtained by iterations using equation (1) (Levenspiel 2012).

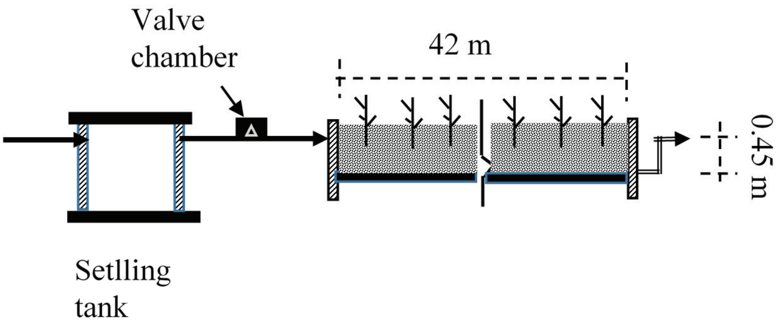
$\sigma^2 = 2D - 2D^2(1 - e^{-D})$……………..(1)

The full-scale constructed wetland

The full-scale horizontal flow constructed wetland is shown in (Fig. 2). The horizontal flow bed is 42m long, 7m wide, and 0.45m deep. After primary treatment in a settling tank the wastewater is continuously fed into the bed. The media in the inlet and outlet zone consist of 40-80 mm crushed stone. The filter bed has 20-30 mm river gravel. The bed is planted with *Phragmites Karkaa*. Sixteen sampling ports, equally spaced at 2.4 m, were installed along the middle longitudinal transect of the bed. The system was run at two loading rates, viz. 0.02 m/day and 0.05 m/day. Following initiation of a new loading rate, the bed was left to stabilize for three weeks before samples were taken. For each loading rate samples were collected from inlet, outlet and the sampling ports placed along the longitudinal transect. Daily composite samples were collected for 8 consecutive days for each loading rate.

Analysis

All samples were analyzed for total suspended solids (TSS), biochemical oxygen demand (BOD_5)*, chemical oxygen demand (COD), total phosphorus (TP), and ammonia nitrogen (NH_4-N) as per APHA (1985). *(*Not analyzed for the full scale wetland)*



<u>Horizontal flow planted bed</u>

- Plants- Local Reed
- Media – River gravel
- Media Size
  - (d_{10}) – 21 mm
  - $d_{60}/d_{10} = 1.2$
- Porosity – 35%
- Bed depth – 0.45 m
- Width – 7 m
- Feeding- Continues

Figure 2. Cross sectional view of the full-scale horizontal subsurface flow bed

Design models

The modified first order plug flow model, k-C* model, (Kadlec and Knight 1996) is expressed in (Eq. 2). The k-C* model can be expressed either in terms of volumetric rate constant (k_v), day^{-1} or in terms of areal rate constant (k_a), m/day. In this study the areal rate constant is used (Eq. 2). The areal rate constant was chosen because it is independent of system depth and media porosity (Kadlec 2009) and thus simplifies the model calibration.

$(C_i\text{-}C^*)/(C_n\text{-}C^*)=e^{-k_a*y/q}$……………………….(2)

Where, C_i = inlet concentration, g/m^3; C_n= concentration (g/m^3) at a fractional distance "y" from the inlet ; C^* = background concentration, g/m^3; q = the hydraulic loading rate, m/day. In equation (2) at y = 1, C_n is equal to effluent concentration C_e (g/m^3). Equation (3) is used to calculate q .

$q=Q/A$………………………………..…………(3)

Where, Q = wastewater flow, m^3/day and A= the surface area of the wetland, m^2. For known q, equation 2 can be used to predict the concentration profile from inlet to outlet along the horizontal flow wetland bed assuming plug flow conditions.

The rate constant (k_a) is temperature sensitive and therefore, the temperature effect on (k_a) is expressed as (Kadlec and Wallace 2009):

$k_T = k_{20}\,\theta^{(T-20)}$ ………………..……………….….(4)

Where, k_T is areal rate constant at temperature T= T °C and θ is the temperature correction factor (dimension less).

The p-k-C^* model in terms of the areal rate constant (k_a) is given in equation 5. (Kadlec and Wallace 2009).

$(C_i-C^*)/(C_n-C^*)=1/(1+k_a y/Nq)^N$……………(5)

Where, N = apparent number of CSTR (dimensionless). The number of CSTR (N) is calculated from tracer data using the equations (6) and (7) (Kadlec and Wallace 2009).

$\sigma^2=\frac{\sigma_t{}^2}{t_{actual}{}^2}$…………..(6)

$\sigma^2=\frac{1}{N}$……………….(7)

The k-C* model and the p-k-C* models were fitted to the experimental data from the pilot- scale and full scale HF beds through optimization of the areal removal rate constants (k_a) after the free fitting parameter C^* was obtained as explained in the results section. For the pilot- scale wetland (k_a) and C* were determined for BOD, TSS, NH_4-N and TP and for the full-scale wetland this was done for COD, NH_4-N and TP.

Statistical Analysis

The mean and standard deviation of the variables were obtained using Minitab. The least-square optimization procedure was used to estimate the model parameters. The fit of the model prediction data to the experimental data was evaluated by the coefficient of determination - R^2. The software tool Matlab was used for the fitting of the curves.

Results

Retention time

The tracer response cures for the HF planted and unplanted beds are presented in Fig. 3. For a q of 0.27 m/day, at which the tracer test was conducted, the theoretical hydraulic retention time (HRT) is 16.8 hrs. The actual HRT calculated from the tracer data are 10.12 hrs and 7.39 hrs for HF planted and unplanted beds, respectively. The actual retention time is shorter than the theoretical for both beds. This implies preferential flow where not the whole flowbed volume is used.

The planted beds have 27% longer retention time than the unplanted beds. The shape of the tracer response curve (Fig. 3) shows that the HF beds do not behave as ideal plug flow reactors because the curves show dispersion. The dispersion numbers calculated using equation 1 were 0.13 and 0.1 for HF planted and unplanted beds, respectively. A dispersion number of zero suggests plug flow conditions and when the dispersion number approaches infinity it suggests a completely mixed reactor (Person et al. 1999).

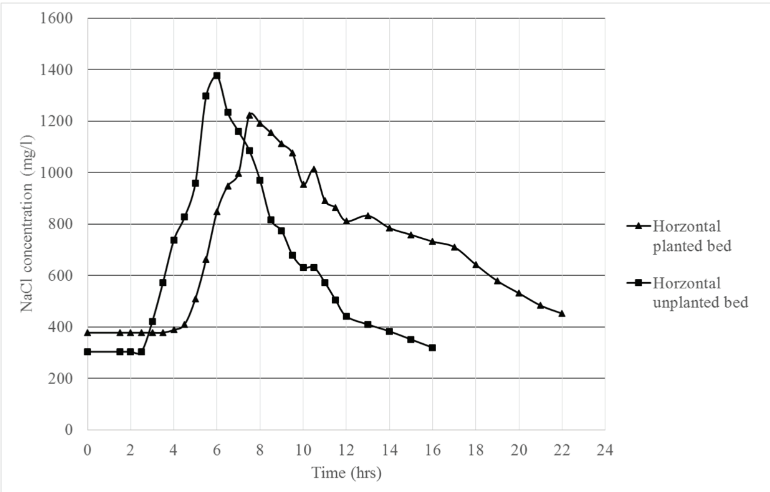


Figure 3. Tracer response curves for the pilot- scale horizontal flow constructed wetlands, including planted and unplanted beds.

The number of CSTR (N) calculated from tracer data was 4. This means that the pilot-scale HF bed unit is hydraulically best represented by conceptually dividing the bed into 4 equal volume completely mixed reactors. Tracer studies were not performed for the full-scale wetland and the number of CSTR of 11 was therefore used (Kadlec and Wallace 2009).

Determination of model parameters

Pilot- scale unit

The effluent concentrations for TSS, BOD, NH_4-N and TP against the inverse of the loading rate (1/q) for the pilot-scale unit are shown in Fig. 4. For BOD and TSS there is an exponential decrease in effluent concentration at increasing values of 1/q (decreasing loading rate q). Both first order models (the k-C* and p-k-C* model) reproduce the measured data fairly well (R^2 0.7-0.81, Table 1). Optimized values for k_a and C* values are presented in Table 1.

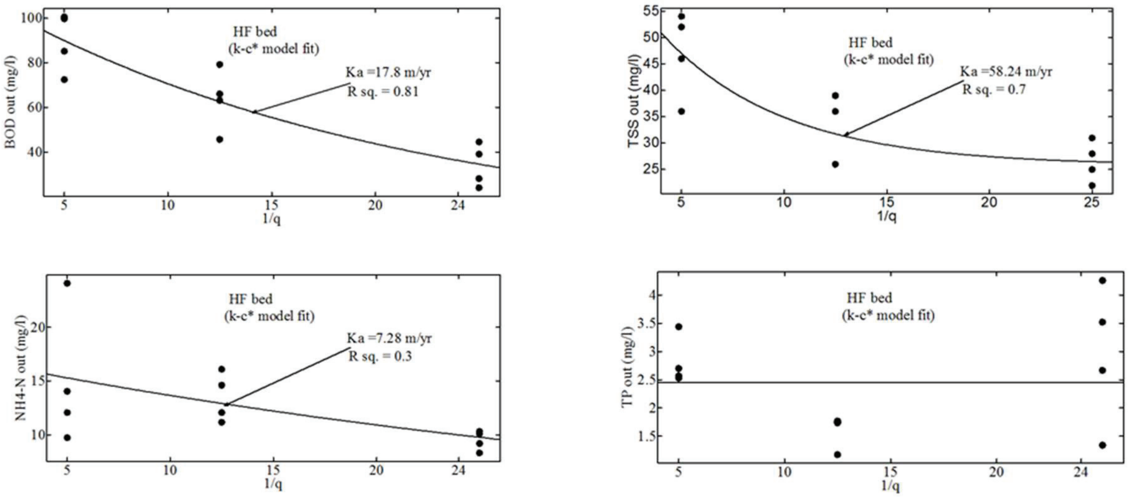


Figure 4. Fitting of data for effluent BOD, COD, NH_4-N, and TP at decreasing loading rate to the k-C^* model for the pilot- scale unit.

Table 1: Estimated values of (k_a) and C^* obtained from k-C^* and p-k-C^* models fitted to the effluent date for loading rates of 0.2, 0.08 and 0.04 m/d for the HF planted bed.

	k-C^* model		C* (mg/l)	p-k-C^* model		C* (mg/l)
	k_a (m/yr)	R^2		k_a (m/yr)	R^2	
TSS	58.24	0.70	25	72.8	0.70	24
BOD	17.80	0.81	0	19.63	0.77	0
NH_4	7.28	0.31	0	9.12	0.30	1
TP			No fit			

The initial value of C* was manually selected and then processed to find optimized values for both k_a and C* (highest possible R^2). The (k_a) values for BOD were 17.8 m/yr for the k-C*model and 19.6 m/yr for the p-k-C* model. For NH_4-N and TP there is no significant decline in concentration with decreasing loading rate.

Full-scale unit

The measured longitudinal profile data from the full-scale wetland is shown in Fig. 5 together with the model prediction.

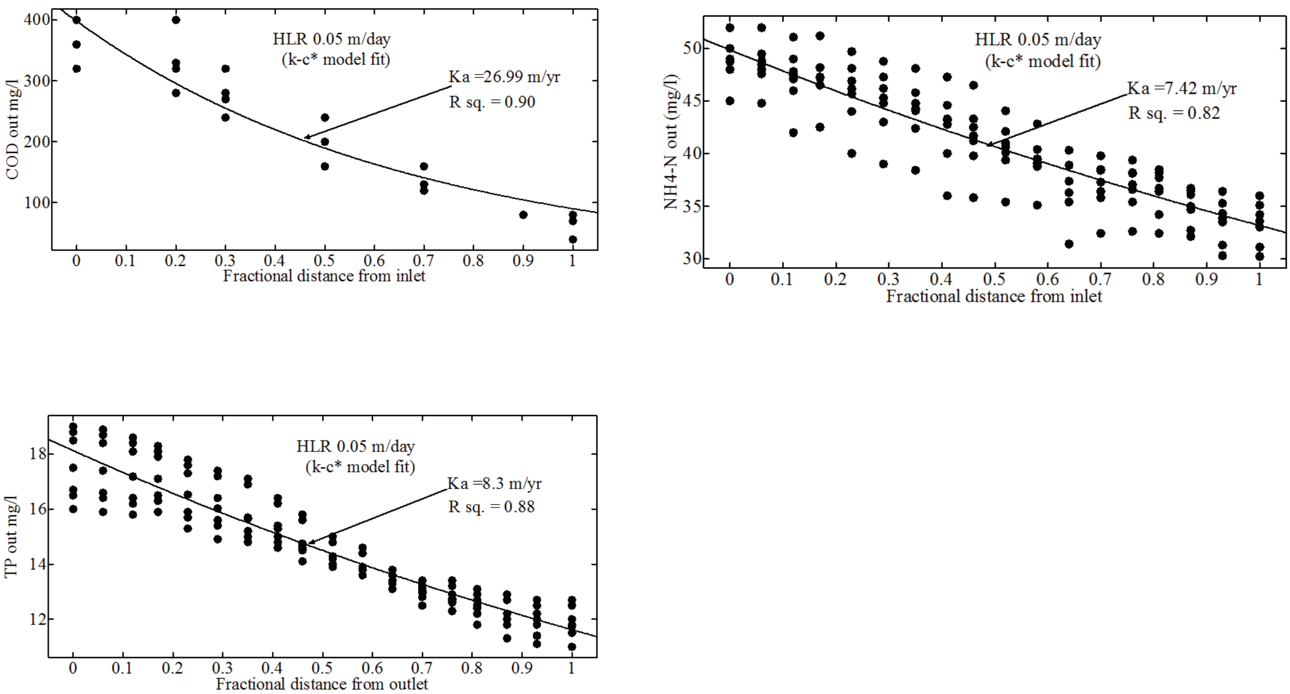


Figure 5. Measured and predicted values by the k-C* model for BOD, NH_4-N, and TP for the full scale wetland at a hydraulic loading rate (q) of 0,05 m/d. The model parameters (k_a) and C* obtained from the curve fitting process for loading rates of 0.05 and 0.02 m/day are summarized in Table 2 and 3.

Table 2: Estimated value of (k_a) and C^* obtained from k-C* and p-k-C^* models fitted to data from the full scale wetland at (q) = 0.02 m/day .

	Influent (mg/l)	k-C* model		C* (mg/l)	p-k-C* model		C* (mg/l)
		k_a (m/yr)	R^2		k_a (m/yr)	R^2	
COD	360±28	13.92	0.88	0	14.54	0.87	0
NH_4-N	43.56±2.4	5.67	0.92	0.2	5.80	0.92	0
TP	17.2±0.52	8.14	0.91	0	8.24	0.91	0

Table 3. Estimated value of (k_a) and C^* obtained from k-C* and p-k-C^* models fitted to data from the full scale wetland at (q) = 0.05 m/day.

	Influent (mg/l)	k-C* model		C*	p-k-C* model		C* (mg/l)
		k_a (m/yr)	R^2		k_a (m/yr)	R^2	
COD	365±24	26.99	0.90	0	28.31	0.89	1
NH_4-N	48.69±1.8	7.42	0.82	0	7.53	0.81	0
TP	17.56±1.04	8.3	0.88	0.5	8.25	0.88	0

For COD the longitudinal profile (Fig. 5) showed an exponential decline, but a plateau above 0 was not reached. The longitudinal profile for both NH_4-N and TP showed a steady and almost linear reduction along the wetland bed. The model fit was fairly good ($R^2 > 0.81$). Table 2 and 3 show that the rate constants increase with increasing (q) for COD and NH_4-N, but not for TP.

Discussion

The actual hydraulic retention time is 39-55 % shorter than the theoretical. This shows that the system has a significant degree of preferential flow. Suliman et al. (2007) pointed to inlet and outlet arrangements and inhomogeneity's in the porous media causing preferential flow. The pilot-scale system, in which the tracer studies were performed, contained a well-sorted coarse sand (d_{10}

= 0.7mm, d_{60}/d_{10} = 4.5). In this material inhomogeneity's in the pore size and distribution resulting from the filling of the porous media into the wetland bed may occur. Inhomogeneity's is less likely the coarser and better sorted the material is. In the coarse very well sorted gravel (d_{10} = 21mm, d_{60}/d_{10} = 1.2) in the full-scale system, such inhomogeneity's are unlikely. In the planted bed of the pilot study the retention time and dispersion was higher. This is most likely because the plant roots decrease the hydraulic conductivity by taking up pore space and roots can also increase the tortuosity of flow thereby increasing the dispersion number and the retention time (Kadlec and Knight 1996).

Evapotranspiration from the wetlands were not measured. However, it can be calculated from standard pan evaporation data (WPCF 1990) and constituted a loss of approximately 15% at the flowrate used during the tracer study. A significant loss due to ET should increase the retention time compared to the theoretical. However, the tracer study showed a retention time lower than the theoretical. This points to a significant preferential flow through the pilot scale wetland beds.

Our calculations for the p-k-C* model based on the tracer curve (Fig. 3) gave 4 CSTR for the pilot-scale wetland. Kadlec and Wallace (2009) found, based on a study of 30 wetlands that the number of CSTR ranged from 2.5 to 34.4 with a mean value of 11.0. A low number of CSTR indicates preferential flow in the wetland (Kadlec and Wallace 2009). The number of CSTR (N) of the full scale wetland selected to 11. The aspect ratio (Reed 1993) of the pilot scale and full-scale wetlands were 3 and 6 respectively. With an aspect ratio of 6 less preferential flow can be expected and thus a higher number of CSTR can be justified. This indicates that the wetlands modeled herein, and the pilot scale wetland, especially, are not hydraulically optimal and that preferential flow paths occur. If a flow situation was established, where all the wetland volume was incorporated, the wetland area could be reduced. Optimizing the flow conditions should therefore be an important focus of further development of CW´s in Nepal.

The distinct exponential behavior for TSS in Fig. 4 shows that for the loading rate of 0.04 m/d the effluent is almost down to the background concentration C* of 25 mg/l also expressed as the non-zero plateau (Kadlec and Knight 1996). The fitted curves for BOD in pilot-scale and COD in full-scale unit show an exponential decay, but the concentrations do not reach an asymptote (plateau).

This indicates that longer retention times (i.e. lower hydraulic loading rate) or larger wetlands are needed to reach maximum treatment capability. The NH_4-N profiles (Fig. 4 and 5) show a declining concentration for longer retention times, however, the fit of the model in the pilot-system was poor due to large variations in effluent values at the highest loading rate (q = 0.2 m/d). The results (Fig. 5) also indicate that longer retention time is required to achieve very low NH_4-N effluent concentrations. In a horizontal flow bed the nitrification is affected by oxygen availability. The measured oxygen concentration in in the pilot scale beds were less than 1.5 mg/l and this is too low to meet both the demand for degradation of carbonaceous organic matter and nitrification (Nivala et al. 2012). The plant uptake of nutrients is low and insignificant when the beds are not regularly harvested (Brix 1997).

For total phosphorus (TP) the fit was poor for both models in the pilot-scale wetland, but good for both models in the full-scale wetland. The phosphorus removal mechanisms in HF wetlands are sorption, precipitation and plant uptake (Vymazal 2007) where sorption is the dominant mechanism if the porous media has a high sorption capacity (Drizo et al. 1999). The sorption capacity of the material commonly used in wetlands in Nepal is low (Laber et al. 1997) and any significant phosphorus removal can only be expected to occur in the juvenile stage of the wetland bed. The pilot-scale unit has been in operation longer than the full-scale unit. It is therefore likely that Fig. 4 shows results from a wetland bed where the TP sorption capacity is saturated whereas in the full-scale wetland (Fig. 5) only the inlet zone is saturated. The model fit for TP reduction the full-scale wetland is therefore still good , but can be expected to decrease rapidly.

Removal of organic matter is mainly achieved by biological processes and the k-C* and p-k-C* models, as the results herein show, are suited to describe that. However, phosphorus removal is more dependent on chemical and chemical/physical reactions than biological, and may be better described by sorption based models (Kadlec and Wallace 2009).

Table 2 and 3 shows that the rate constants increase with hydraulic loading rate for both models. This is also shown for other wetland systems optimal use of the system is reached (Kadlec 2000).

For the pilot- scale wetland k_a values for BOD based on p-k-C* model was 19.6 m/yr. For the full-scale bed k_a for COD was 29.12 m/yr. Kadlec (2009) found an average k_a for 53 wetlands of 37 m/yr using the p-k-C* model. Rousseau et. al. (2004) has compiled k_a values from several authors, mostly from climates colder than Kathmandu, and find a variation from 21.8 m/yr to 364 m/yr using the k-C* model. So, the rate constants in the warmer climate of Kathmandu were small compared to many of the values reported in the literature. Since rate constants increase as wetlands mature (Brix et al. 1998) the low values for the Kathmandu wetlands could be due to low system age. However, Vymazal (2011) did not find such an increase in the rate constant with age. For the k-C* model the rate constant for BOD in our pilot- scale wetland was 17.8 m/yr, This is lower than all the values reported by Rousseau et al. (2004) and Kadlec (2009) and indicates that the higher temperature in Kathmandu does not result in larger values of k_a.

Both the k-C* model (eq. 2) and the p-k-C* model (eq. 5) describe reduction of pollutants as an exponential decrease of through the wetland. If the decrease is larger, k_a will be larger. If preferential flow occurs, the pollutant reduction in the wetland is suboptimal and the apparent k_a will be smaller. Thus the preferential flow may have contributed to the low k_a values for the Kathmandu wetlands. Surface area available for the biofilm bacteria attribute to the rate of degradation (Khatiwada and Polprasert 1999, Kaseva 2004). In optimal conditions, the water flowing through the wetland is transported to all the potential sites of biofilm growth. However, such an ideal condition was not achieved, as shown from the tracer study. Besides, the effective biofilm surface area will increase as the system matures due to more root development giving more area for biofilm growth. Therefore it is likely that both the flow conditions and system age has contributed to the low (k_a) values in this study.

Conclusions

The wetlands were not hydraulically optimal and preferential flow paths occur. If a flow situation is achieved, where more wetland volume is utilized, the wetland volume (and area) can be reduced. This should be focus of further development of horizontal flow subsurface wetlands in Nepal. Comparison of the tracer data from the planted and the unplanted bed, showed that the presence of plants increased dispersion. The modified first order model (k-C*) and the tank in series model (p-k-C*) are suited to describe biological removal reactions as for BOD, but not for phosphorus removal that is mainly dependent on sorption reactions. The p-k-C* model is regarded to describe the flow conditions in a horizontal flow wetland better than the k-C* model. However, in this study the models performed equally well (R^2) when fitted to the experimental data for total suspended solids (TSS), and organic matter measured as BOD and COD. The rate constants increased with hydraulic loading rate (q) for both models. The rate constants (k_a) for organic matter removal determined in this study were low in comparison to rate constants determined for wetlands in colder climates. This indicates that the organic matter removal rate constants are not necessarily higher in warmer climates. Preferential flow and lack of established biofilms due to young system age may have contributed to the low k_a values.

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Paper V

Manoj K. Pandey and Petter D. Jenssen. Wastewater infiltration for purification and groundwater recharge: International experience and potential in Nepal.
Submitted to Water Practice and Technology (WPT).

Wastewater infiltration for purification and groundwater recharge - international experience and potential in Nepal

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Abstract

Treatment of wastewater by infiltration through soil is a proven robust and low cost method that works well also in cold climate provided the natural conditions are favorable. In Norway there is more than 100,000 wastewater infiltration systems. The majority serves single homes and the largest treat wastewater from a town of 8000 people. Infiltration systems utilize soil for treatment and often operate by gravity alone; hence, there is no need for electricity. Still the performance can be very high. Soil infiltration can also help to recharge groundwater, but utmost care has to be taken during design and planning so as not to cause pollution of drinking water sources. By upgrading soakpits to modern infiltration systems the risk of polluting underlaying aquifers is reduced. If the excreta is handled separately and greywater only, is infiltrated the risk of microbial pollution is minimized. Soil infiltration requires permeable soils and a separation distance to the groundwater below the system. Small systems, for single or a few dwellings can be constructed in less permeable, silty or clayey soils. Large systems require more hydraulic capacity and, hence, sand and gravel deposits are preferred. In Nepal the mountains, glaciers and corresponding water systems has created many deposits (fluvial, alluvial and glaciofluvial) that are well suited for wastewater infiltration. This paper gives a short overview of soil infiltration systems as well as their potential in Nepal. There is a comprehensive international knowledge base regarding wastewater infiltration that can be utilized to tailor guidelines for infiltration systems in Nepal and similar regions. Pretreatment of wastewater reduce the organic load and enhance infiltration. This paper also suggests the new possibility of combining constructed wetlands and infiltration.

Keywords: Infiltration, Wastewater, Soak-pit, Constructed wetlands Groundwater recharge, Soil properties, Purification

Introduction

Treatment of wastewater by infiltration through soil is a proven robust and low cost method that works well also in cold climate provided the natural conditions are favorable (Siegrist et al. 2000, Jenssen et al. 2014). Infiltration systems utilize soil for treatment and can operate by gravity alone; hence, electricity is not a prerequisite (Jenssen and Siegrist 1990). Soil infiltration can also help to recharge groundwater, but utmost care has to be taken during design and planning so as not to cause pollution of drinking water sources. Soil infiltration requires permeable soils and a separation distance to the groundwater below the system. Small systems, for single or a few dwellings can be constructed in less permeable, silty or clayey soils. Large systems require more hydraulic capacity and, hence, sand and gravel deposits are preferred (Jenssen and Siegrist 1990). If the soil and hydrogeological conditions are favorable infiltration can be the main treatment component for tourist facilities, villages or towns. In the USA 25% of the population is served by onsite and decentralized wastewater systems and the majority of them are soil infiltration systems (Siegrist et al. 2000). In Norway there are more than 100,000 mostly single household wastewater infiltration systems, but the largest one treat wastewater from a town of 8000 people. In developing countries septic tanks and soak-pits are common onsite sanitary systems both in rural and urban areas. One problem with such systems is the poor quality of the septic tanks (Rana et al. 2007) leading to leakages or inflow of water at rainfall events. Tanks should not leak for proper performance of the system (Bounds 1997) .

When septic tank effluent (STE) is discharged to a soak-pit a high hydraulic load is applied to a small area increasing the risk of overloading the treatment capacity of the soil (Fig 1 a). By applying the STE in shallow infiltration systems designed to match local soil conditions (Fig 1 b) the soils ability to mineralize, retain and remove pollutants can be utilized (Siegrist et al. 2000, McKinley and Siegrist 2011). In addition, the distance to the groundwater is important since the purification properties normally are best in the unsaturated (vadoze) zone. A soak pit is often deep and discharge pollutants closer to the groundwater than a shallow infiltration trench (Fig. 1a).

Major research regarding wastewater infiltration systems, their ability to remove pollutants, and their design was performed in the last decades of the previous millennium (Mc Gauhey and Krone 1967, Healey and Laak 1974, Bouma, 1975, Otis et al. 1980, Lewis et al. 1982, Jenssen and Siegrist 1991, Tyler and Converse 1994, Emerick et al. 1997, Siegrist et al. 2000). After the turn of the millennium there has been less research activity in the field. Heistad et al. (2001) pointed to some innovative design details and the use of drip irrigation systems for disposal of wastewater has been studied and successfully practiced (Parzen et al. 2007). The knowledge of process details including removal of microorganisms and organic pollutants as pharmaceuticals has been expanded (Van Cuyk et al. 2007, Lowe et al. 2008, Forquet et al. 2009, Heistad et al. 2009, Conn et al. 2010, McKinley and Siegrist 2010 and 2011, Eveborn et al. 2012, Jenssen et al. 2014).

Infiltration systems, when properly designed and installed, provide tertiary treatment (Jenssen and Siegrist 1990), but the potential for use of such systems is far from fully utilized. Since the infiltration systems rely on the local natural conditions for purification they are exploiting what we can term ”ecosystem services”. When the conditions are favorable these technically simple systems are often extremely cost effective (Jenssen et al. 2014).

In this paper some international experience is reviewed as background for suggesting infiltration as final treatment and disposal method for domestic wastewater for various situations in Nepal, but also for other countries with similar challenges.

Constructed wetlands (CW´s) are, as infiltration systems, technically simple and robust. CW´s produce a low BOD effluent and can reduce the number of indicator organisms substantially. However, CW´s normally do not reduce the nutrient content above secondary treatment standards so effluent from CW´s may cause eutrophication upon discharge. If the BOD in the incoming water is low infiltration is facilitated (Siegrist 1987, Jenssen and Siegrist 1991). This paper also points to the possibility of combining CW´s and subsequent infiltration.

Soil infiltration systems

The design, performance and the service life of an infiltration system depend upon the purification properties of the soil, composition of the wastewater and hydraulic properties of the porous media (Jenssen and Siegrist 1991). The main pollutant removal occurs in the unsaturated zone, hence, its depth should be sufficient to purify the wastewater before it mixes into the groundwater. Infiltration systems are based on discharge of partially treated wastewater effluent, normally STE, to subsurface soils with recharge to ground water underlying the site (Fig. 1b).

System design

A modern infiltration system consists of a septic tank followed by a shallow disposal part of underground trenches, beds or open basins (Fig. 2). For large systems (>500 pe) open basins are often chosen (Fig. 2A). If open infiltration basins are used an adequate buffer zone should be provided between the basins and the community to minimize the odor and nuisance. In warm climate open infiltration basins may provide hatching grounds for mosquitoes and this should be taken into consideration when the system design is selected.

Smaller systems are normally buried under ground. Trench geometry with shallow placement has been advocated to maximize infiltration surface area and exploit the most biogeochemical active zone of the soil profile (Siegrist et al. 2000). At places with high ground water tables mound systems (Fig. 2C) can be used to obtain sufficient unsaturated zone above the groundwater (Tyler and Converse 1985).

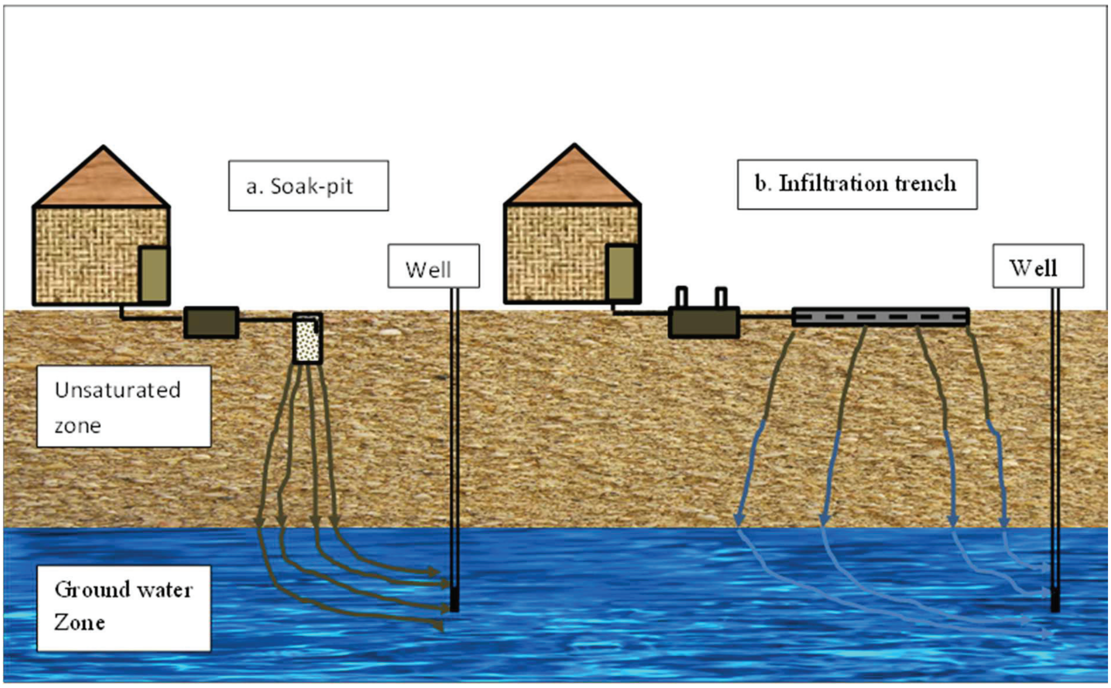

Figure 1 Cross sectional view of; (a) a soak-pit and, (b) a shallow infiltration trench.

In order to be able to infiltrate wastewater, the soil at a given site must have capacity to receive the given amount of wastewater for an indefinite time without causing excessive groundwater mounding. The amount of wastewater that a site can receive is termed the hydraulic capacity of the site and can be calculated based on a hydrogeological assessment of the site (Jenssen and Siegrist 1990).

A. Open

B. Buried

C. Mound

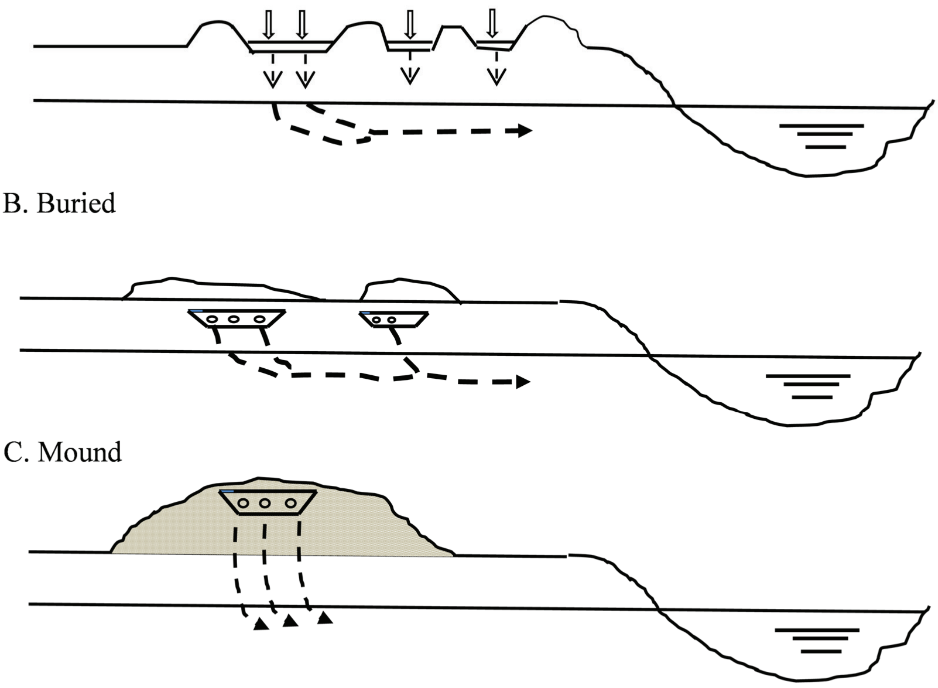

Figure 2 Infiltration system design. A: Open basins, B: Buried systems and C: Mound systems.

In order to determine the the necessary area needed for infiltration the hydraulic loading rate of the soil needs to be assessed (Jenssen and Siegrist 1991). The hydraulic loading rate can be decided based on soil morphology (Tyler and Converse 1994), but the most common methods are based on texture (Naturvårdsverket 1987) or on hydraulic measurements alone or their combination (Jenssen and Siegrist 1991). To facilitate selection of an appropriate loading rate a diagram is developed (Jenssen and Siegrist 1991). This diagram is a standard part of Norwegian guidelines, but can be adopted to other countries or conditions.

The common practice is that the design application rates for soil infiltration systems are in the range of 1 to 5 cm/day with site-specific rates based on soil textural properties e.g 5 cm/d for coarse sand and 1 cm/day for fine-grained (clayey, silty) soils (Siegrist et al. 2000).

Removal of pollutants

Removal processes in soil are complex and difficult to model (Siegrist et al. 2000, Ausland et al. 2002). However, practical experience along with scientific results show that both bacteria and viruses can be reduced to acceptable levels within a few decimeters of travel through unsaturated soil provided the system is properly designed for the given site conditions (Lewis et al. 1982, Stevik et al. 1999, Heistad 2008, Conn et al. 2010). In coarse-grained material as gravel and coarse sands removal of microorganisms is less efficient, but can be enhanced by applying STE in many small doses over the day (Emerick et al. 1997). This will require a dosing device as a pump, siphon or tipping bucket. The soil microorganisms quickly adapt to transforming substances in STE and both reduction of organic matter and the transformation of nitrogen from ammonia to nitrate is efficient in soil (Van Cyuk et al. 2001), but the extent of denitrification depends on the soil and system design (Lance et al. 1976). The range of removal of total nitrogen therefore varies from 30% to 80% (Jenssen et al.1988). The main processes for phosphorus removal are adsorption, complexation and precipitation. Fe- and Al-oxides and hydroxides along with calcium compounds are known to be important agents for P removal in soil (Enfield 1974, Stuanes and Nilsson 1987). Phosphorus removal is normally high provided shallow systems are used (Jenssen et al. 2014).

The risk of groundwater pollution can be greatly reduced by proper siting and design of the infiltration systems. The mass fecal load in blackwater (urine and faeces) is 2-4 orders of magnitude more than in greywater (Ottoson and Stenström 2003). In addition more than 80% of the nitrogen and phosphorus, 84% and 87% respectively, is found in blackwater (Todt et al. 2015). Substantial risk reduction can therefore be obtained if only greywater is infiltrated.

The removal of trace organic chemicals and pharmaceuticals in soil is not studied to the same extent as the traditional pollutants; biochemical oxygen demand (BOD), suspended solids (SS), nitrogen, phosphorus and bacteria. However, studies by Conn et al. (2010) indicate that concentrations of trace organic compounds in septic tank effluent are reduced by more than 90% during transport through 240 cm (often within 60 cm) of soil most likely due to sorption and biotransformation.

Longevity, loading rate and performance

In open systems (Fig. 2 A) the infiltration rate can be controlled and the surface scraped when the rate becomes to low. Hence, open systems are loaded at higher rates (20 – 50 cm/d) than buried systems (1 - 5 cm/d). The low loading rate applied to buried systems ensures a long hydraulic service life. Hill and Frinck (1980) analyzed more than 2000 systems in Connecticut and found an average expected hydraulic service life to be more than 30 years for most buried systems. Bardu municipality located at 69^{o} northern latitude in Norway built an open infiltration system to treat wastewater from 5000 inhabitants in 1987 (Jenssen at al 2014). The unsaturated zone below the basins is 7m. The groundwater has been checked regularly (Table 1). The total nitrogen removal has declined after garbage grinders were introduced in 1996. For chemical oxygen demand and total phosphorus there has been no decline in removal with time. Despite an average annual temperature of $+0.7^{o}C$ nitrification with subsequent denitrification can explain the high N-removal. Under each basin the capacity for phosphorus removal is estimated to last 12 years. The concentration of indicator bacteria has only been assessed sporadically, but has never exceeded 20 CFU/100ml.

Table 1 Treatment performance (%) at the infiltration system in Bardu municipality for total phosphorus (total-P), total nitrogen (total-N) and chemical oxygen demand (COD). Average values of 6 – 12 yearly samples before and after 1996 (from Jenssen et al. 2014).

Parameter	1986-95	1996-2013
Total -P	99	99
Total -N	77	59
COD	87	90

The system has saved the municipality an estimated 45 million NOK over 25 years compared to investment and operation of a conventional mechanical/chemical treatment system (Jenssen et al. 2014).

Soil Infiltration in Nepal

In Nepal about 37% of urban households and 20% of rural households have septic tank based onsite sanitation systems (UN habitat 2008). In urban areas, particularly, the use of septic tanks with overflow to soak-pits or nearby open drainage cause serious health problems due to pollution of groundwater and surface water (Rana et al. 2007). Around 30% of the households in the Kathmandu valley have toilets connected to septic tanks (HPCIDBC 2011). Only 35% who have septic tanks have soak pits associated with the septic tanks (ADB 2000). In most cases the septic tanks are built without proper designs, hence they leak, and therefore there is always a risk of groundwater contamination (HPCIDBC 2011). Since the systems are built based on local tradition and without considering local soil characteristics and ground water conditions the systems are often overloaded (ADB 2009). Concentrations of TN and TP in groundwater in urban areas were higher than in rural areas (Kannel et al. 2007). The groundwater pollution potential from these systems can be reduced by using water tight septic tanks and replacing the soak-pits with shallow soil infiltration trenches. The shallow infiltration trenches utilizes larger soil volumes for purification and provides more distance to ground water than soak pits (Fig. 1a).

Infiltration can also be important for groundwater recharge. Groundwater is the major source of water in the Kathmandu valley. The total annual abstraction is presently estimated at 21.56 million cubic meters and the maximum recharge is estimated to be about 9.6 million cubic meters (Pandey et al. 2010). Properly designed wastewater infiltration systems can increase recharge of the groundwater reserve. However, there is always a risk of ground water pollution, particularly of shallow phreatic groundwater reservoirs, if the infiltration systems are not properly designed and constructed.

Soil and hydrogeological conditions in Nepal

The design and use of soil infiltration systems requires that the site conditions are suitable (Jenssen and Siegrist 1990). Nepal does not have guidelines for infiltration systems, but indications of the potential for infiltration can be made from existing soil and hydrogeological investigations and by comparing to similar conditions elsewhere. Pradhanang et al. (2012) have assessed the potential for infiltration of surface runoff to augment the depleting ground water resources. The study shows that the northern part of the valley and the areas along the Bagmati river are composed of unconsolidated highly permeable materials that are about 60 m thick and forms the main phreatic aquifer in the valley (Fig. 3). The central and southern part of valley are comprised of silty clay lake deposits, forming a clay aquitard protecting the deeper confined aquifer. Based on the information given by Pradhanang et al. (2012) the northern part of the valley have potential for large infiltration systems whereas in the central and southern part of the valley small household infiltration systems can be built with proper assessment of soil and groundwater conditions. However, the northern part has the highest risk for groundwater pollution.

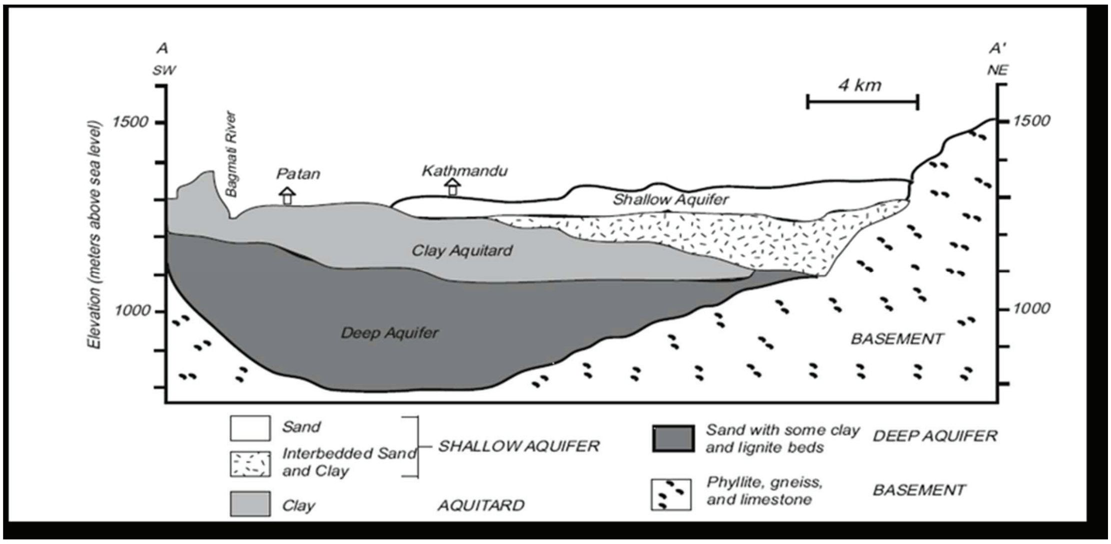

Figure 3 Cross section through the Kathmandu valley showing soil morphology (Source: Pradhanang et al. (2012).

Norway and Nepal are both mountainous countries where rivers have been depositing large volumes of silt, sand and gravel along river plains and valley bottoms. Such sediments are often well suited for infiltration systems (Jenssen et al. 2014) provided the groundwater pollution issues are handled properly and the groundwater is not to shallow. Nepal has soil maps and soil classification of varying quality and scale that can be used for the assessment of soil infiltration potential, but the amount of information that can be extracted from maps is normally not sufficient for design of a system. A site assessment is therefore needed.

Some designconsiderations

Infiltration systems in Nepal can in principle be of the types shown in Fig. 2. For sites with soil of low hydraulic conductivity, as in the southern part of the Kathmandu valley (Fig. 3) pretreating the wastewater to secondary quality can enhance subsequent infiltration (Jenssen and Siegrist 1991). Sandfilters, biofilters or constructed wetlands can be used for pretreatment. Such systems will remove substantial amounts of BOD, suspended solids, nitrogen and microorganisms prior to the infiltration (Siegrist et al. 2000, Jenssen et al. 2005, Pandey et al. 2013).

Constructed wetlands are commonly used in Nepal (Shrestha et al. 2001). Constructed wetlands are usually made with a sealed bottom. In fine-grained soils of low hydraulic conductivity as silt and clay soils, the authors suggest that wetlands are constructed with an unlined bottom (Fig. 4). This gives a cheaper wetland construction, and allows pretreated water to infiltrate whenever possible. In such fine-grained soils the purification capacity is generally excellent. How much that will infiltrate depends on the sizing of the system the hydraulic conductivity of the underlying soil and potential clogging of the wetland base. The overflow is treated water of secondary quality and can be discharged to open waterways, but preferably dispersed in a shallow infiltration or drip irrigation systems. The effluent from a constructed wetland is rich in nitrogen and phosphorus but low in BOD and bacteria and, thus, well suited to irrigate green- or agricultural areas. An open bottom construction may give a fluctuating water level in the wetland and. thus, plants that can tolerate varying moisture conditions (eg. *Phragmites australis*) should be selected (Pagter et al. 2005).

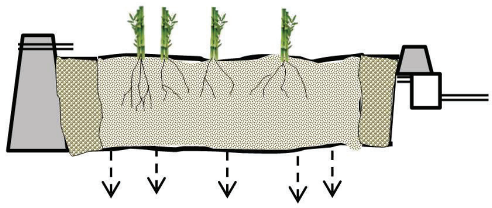

Figure 4 A horizontal subsurface flow constructed wetland with an unsealed bottom.

Conclusions

Decentralized wastewater management using soil infiltration systems can be a solution for rural as welle as urban and periurban areas of countries like Nepal. Upgrading existing septic tank/soak pits to modern shallow infiltration systems can reduce both groundwater pollution risk and increase groundwater recharge. However, new systems must be properly constructed and septic tanks should be watertight. Infiltration systems, as natural systems in general, require fairly large areas and this do limit urban applications. Infiltration of greywater only can greatly reduce the risk for groundwater pollution. For correct design an assessment of the soil conditions should be undertaken. For smaller or single house systems simple investigations can be made. For large-scale infiltration systems a more comprehensive hydrogeological assessment including large-scale infiltration tests and tracer studies may be required. Loading rates can be determined based on a simple infiltration test, from soil texture or a combination. In order to successfully implement infiltration systems in Nepal local guidelines for site assessment and system sizing and design should be developed. There is substantial international experience regarding infiltration systems that can facilitate development of Nepalese guidelines and design criteria. Constructed wetlands and subsequent effluent infiltration will yield excellent purification as well as robust and flexible treatment systems.

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