Iraqi Journal of Chemical and Petroleum Engineering
Vol.12 No.1 (March 2011) 43 - 52

ISSN: 1997-4884

University of Baghdad

College of Engineering

REUSE OF DOMESTIC WASTEWATER FOR IRRIGATION: CONCEPTUAL AND BASIC DESIGN ELEMENTS

Abdul-Fattah Mohamed Ali
Environmental Eng.Dept., College of Eng., Baghdad University

ABSTRACT

In Iraq, water shortages and drought, especially during the hot summer months, necessitates that municipal authorities adopt water reuse projects like reusing treated domestic wastewater for crop irrigation. This work gives the conceptual and basic design elements for the necessary steps of filtration, UV irradiation and chlorination to make such a wastewater fit for agricultural use. A typical rural community of 50,000 people is considered as an example case for which functionality and relative simplicity of the proposed designs are prime factors. The objectives are 1) to show what is required and 2) that the presented information may be utilized to embark on the following phases of detailed design and execution of such projects.

Keywords: Wastewater reuse; Crop irrigation; Wastewater filtration; UV irradiation of wastewater ; Wastewater chlorination.

INTRODUCTION

Historically, Iraq enjoyed an abundance of fresh surface water resources (Euphrates, Tigris Rivers and their tributaries). However, in recent decades this situation has changed dramatically and irreversibly due to water impoundment projects in river-source neighboring countries and erratic rainfall probably caused by climate change.

Consequently, the country now faces water shortages especially during the hot summer months of June, July and August. Additionally, many rural communities have been provided with potable water supply units but lack domestic wastewater treatment facilities creating an ongoing potential environmental problem. It is, therefore, highly relevant that such planned facilities should incorporate means to render treated domestic wastewater suitable for irrigation.

This work provides the conceptual and basic design elements necessary to embark on the detailed design and execution phases for the achievement of the aforementioned goal.

A rural community of 50,000 people with an assumed domestic wastewater flow rate of 12,600 m^3/d is considered as an example case for illustration purposes. The presented design elements include sufficient margins to ensure functionality and could serve as a general guideline for other capacities. They are based on information available in the literature and author's experience in water/wastewater treatment.

Pre-requisites

It is assumed that a conventional medium-rate activated sludge process ($0.2 < \frac{F}{M} < 0.5\ d^{-1}$) is adopted for the domestic wastewater treatment facility resulting in a standard secondary effluent ($BOD_5 \cong 25$, S.S $\cong$ 35, FOG $\cong$ 10, all mg/l) which is 10 to 20 % nitrified (Vernick and Walker [1]).

For the community size under consideration, this process should include a properly sized equalization basin to dampen out diurnal and seasonal wastewater flow and concentration variations.

Treatment requirements

The secondary effluent with the said specifications requires the following to make it suitable for crop irrigation (Abdulraheem [2]; El-Wakeel et al. [3])

- The count of pathogenic microorganisms in the ww must be reduced to the level commensurate with this reuse category.
- Additionally, a means of maintaining the ww quality should be provided to ensure its suitability until it reaches the agricultural fields.

Reduction of pathogenic microorganisms count is achieved by disinfection, the extent of which is based on indicator microorganisms; namely faecal/total coliforms. However, the efficacy of disinfection is negatively affected by the S.S/turbidity of the ww (due to its masking effect shielding some microorganisms from disinfection [4]). Therefore, proper filtration of the ww is a necessary preliminary stage whenever disinfection is o be carried out.

Following filtration and disinfection, maintenance of the ww quality until it is used is related to the type of disinfection process chosen as will be made clear in due course.

Filtration

The adopted design approach for this operation is based on the assumption that it should be similar to that used in potable water supply units, since considerable design and operational experience exist in this respect in municipalities all over the country.

Filtration system general specifications

(Metcalf and Eddy [5]; Abdul-Fattah [9]; Crites and Tchobanoglous [10]; Degremont [11]; Tebbutt [13])

- Type: Rapid gravity flow, deep-bed, mono-medium filter.
- Medium : Sand with an effective size of 2 to 3 mm and a uniformity coefficient of 1.2 to 1.6.
- Medium depth: 0.9 to 1.8 m.

- Filtration rate : 100 $\frac{\text{l/min}}{\text{m}^2}$ (6.0 $\frac{\text{m}^3\text{/h}}{\text{m}^2}$)
- Filtration runs duration: 8 to 24 h.
- Mode of operation: Constant level with constant flow rate.
- Water level above sand during filtration run : To be maintained at 1.2 m.
- Underdrain system : False-floor equipped with long-stem plastic nozzles.
- Backwash mode : Air scour followed by air plus water then only water.

Remarks concerning filtration of secondary effluent

1. A properly designed and operated filtration system should remove 65 to 80% of the suspended solids in the wastewater.
2. The specified filtration rate of 100 $\frac{\text{l/min}}{\text{m}^2}$ would ensure that more than 90% of the 10 µm size particles are removed ; which is of particular relevance to the following disinfection process by UV irradiation.
3. Filters should be backwashed at least once every 24 h to avoid formation of mudballs (i.e. agglomeration of biological floc, dirt and sand) as well as buildup of grease.
4. The specified mode of operation and filtration run duration would ensure that loss of filtration medium is minimized ⁄ prevented.

Filtration conceptual and basic design elements are given in Appendix A.

Disinfection

The relevant indicator microorganism for disinfection is faecal coliform whose count should be around 100 MPN/100 ml for crop irrigation (Abdulraheem [2] ; El-Wakeel et al. [3])

The two widely-used disinfection methods for this purpose are chlorination and UV irradiation. Each method has its merits and drawbacks. To appreciate the adopted design approach in this work, some relevant characteristics of filtered secondary effluent are in order. This kind of water contains NH_3 ranging from 18 to 30 mg/l, so that when it is chlorinated, two outcomes are possible depending on the amount of Cl_2 added (Metcalf and Eddy [6] ; Qasim [15]) :

a) Formation of chloroamines (combined chlorine) with partial or total removal of NH_3 but before the breakpoint is reached.
b) The breakpoint is passed with an amount of free Cl_2 present.

Combined chorine requires 100 times the contact time of free Cl_2 to achieve the same microorganism kill level (Tebbutt [14]). Therefore, if a relatively small amount of Cl_2 is added (outcome (a)) , the required contact time will be excessive (of the order of days). Conversely, to limit the contact time to the range of 30 to 90 min. with free Cl_2, an excessive amount of Cl_2 must be added (outcome (b)). This is uneconomical and may result in unacceptable levels of DBP's. Therefore, chlorination on its own is not a cost-effective option to disinfect wastewater that has not been nitrified/ denitrified.

An alternative approach, adopted in this work, would be to irradiate the wastewater using UV light and then to chlorinate it to a limited extent to utilize chloroamines as combined chlorine residual to maintain the water suitability until it reaches the agricultural fields. In this case only a fraction of the NH_3 present is converted to chloroamines, the predominant species of which is monochloroamine (NH_2Cl) due to the water pH and the Cl_2 to NH_3 molar ratio. The necessary concentration of NH_2Cl is a function of the distance/time required for the water to reach its destination. Stoichiometrically four weights of Cl_2 are required to convert one weight of NH_3 to NH_2Cl. It is also noteworthy that combined chlorine does not form DBP's (Metcalf and Eddy [7]; Degremont [12]).

UV irradiation (Metcalf and Eddy [8] ; Suntec Environmental [17] ; Tchobanogolous et al. [18])

This system is based on the widely-used low-pressure low-intensity UV lamp with a useful life of about one year. The system shall constitute three parallel channels to match the filtered water exiting the three aforementioned parallel filters. The basic design elements of each UV channel are as follows:

- Wetted dimensions : 0.6×0.6 m
- No. of UV banks : one
- Lamp configuration (placement) : horizontal
- No. of lamp modules per bank : 8
- No. of lamps per module : 8
- No. of lamps per bank : 64
- Effective arc length of lamp : 1.47 m
- Diameter of lamp's quartz sleeve : 23 mm
- Lamp spacing : 75 mm (center to center)
- Power consumption per bank : 5 kW
- Lamp current & voltage : 0.34 amp at 220 volt
- Diffuser no. and type : Two ; perforated plate. One upstream and one downstream of bank. Each perforated plate has 256 (16×16) 13-mm holes; hole spacing 37.5 mm (center to center).
- Head loss across UV-bank including diffusers: 32 cm WC; this may be overcome by two 16-cm steps one upstream and on downstream of bank.

All the aforementioned elements specifications are typical. However, they are collectively consistent and match the given wetted dimensions of the UV channel. This matching, plus the employed of diffusers (to ensure plug-flow condition) is necessary to ascertain that an acceptable average UV intensity value is achieved within the bank as the ww passes through it.

Details of UV irradiation system basic design elements are given in Appendix B.

Chlorination

As pointed out earlier, the aim of this process is to produce the necessary concentration of NH_2Cl in the filtered, disinfected secondary effluent to maintain its suitability until it is utilized for irrigation.

The three wastewater streams exiting the UV irradiation system should transfer the water to a buffer basin of suitable size. Typically an effective volume equivalent to about three-hour flow is used. For the capacity under consideration this would result in a 1500 m^3 size basin which could be constructed of three parallel basins, each with dimensions of 4×5×25 m (D×W×L). Therefore, each UV channel shall be connected to one 500 m^3 basin.

It is envisaged that three separate chlorination systems, one per basin, should be employed. Each system consists of a pump, an educator and a nozzl arrangement plus the necessary piping connecting them. The centrifugal pump withdraws water from the upstream end of the basin , pumps it through the educator whose gas side is connected to the Cl_2 supply line from the Cl_2 cylinder, then returns the chlorinated water back to the downstream end of the basin through a hanging – nozzle type multiple diffuser mixer placed across the width of the channel as illustrated in Fig .(1) (Qasim [16]).

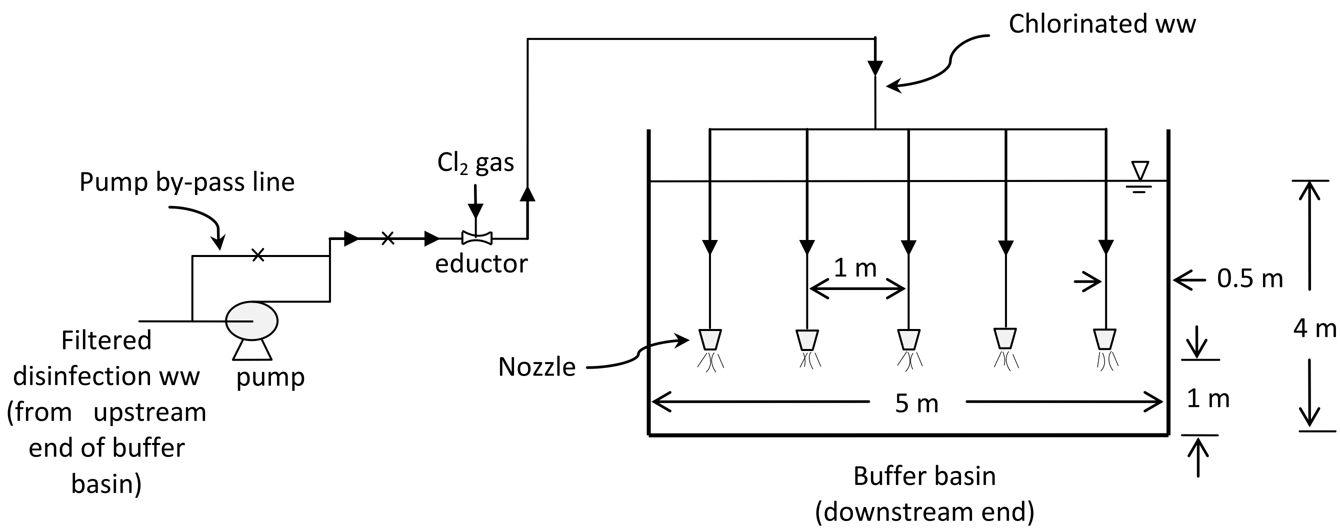

Figure 1 Schematic diagram of chlorination system arrangement.

Cl_2 reacts readily with NH_3 to form chloroamines and in order to utilize all the supplied Cl_2 for this purpose (i.e to minimize undesirable side reactions) the chlorinated water should have an excess of NH_3. This point is illustrated in Appendix C which gives the basic design elements of a chlorination system for an example case.

Final note

The presented design has a turn down ratio of 3:1 ; i.e it can be operated at 33%, 67% or 100% of its design capacity because it is based on three parallel trains. Such flexibility is important due to seasonal variation in irrigation needs. Additionally, it serves to facilitate maintenance requirements.

CONCLUSIONS

Secondary effluent from a conventional medium–rate activated sludge process may be filtered, UV irradiated and partially chlorinated (chloraminated) to render it suitable for crop irrigation. The necessary conceptual and basic design elements for these steps have been given to enable local municipal authorities to appreciate the requirements as well as to serve as a basis for the subsequent phases of detailed design and execution of such projects.

REFERENCES

[1] Vernick, A.S. and E.C. Walker, Editors (1981) "Handbook of Wastewater Treatment Processes", Marcel Dekker, New York, p's. 30,32,40.

[2] Abdulraheem, M.Y. (2008) "Public Health Aspects of Wastewater Treatment and Reuse in Kuwait", in "Wastewater Reclamation and Reuse", Al-Sulaimi and Asano, Editors, Arab School on Science and Technology, Kuwait, New Age Int., New Delhi, pp.135-180.

[3] El-Wakeel, A.F, El-Nashar, B.M. and O.A. Al-Hamad (2008) "Reuse of Sewage Effluents for Irrigation", in "Wastewater Reclamation and Reuse", Ibid.pp. 244-259.

[4] Metcalf & Eddy,Inc. (2003) "Wastewater Engineering Treatment and Reuse", 4th Edition, Revised by Tchobanoglous,G. , Burton,F.L. and H.David Stensel, McGraw-Hill , NewYork, p.'s.1080,1328.

[5] Ibid. pp.1044-1096.

[6] Ibid. pp. 1231-1239.

[7] Ibid. p. 1257.

[8] Ibid. pp. 1298-1329.

[9] Abdul-Fattah, A.M.A(2002) "Conceptual Treatment Scheme for One Stream of Intended Rasafa Potable Water Works", Report No.Am-R3-02-RP-R4,Revised Issue, Al-Magd General Co.,Baghdad. pp. 25-28.

[10] Crites,R. and G. Tchobanoglous (1998) "Small and Decentralized Wastewater Management Systems", McGraw-Hill , New York, pp. 813-820.

[11] Degremont (2007) "Water Treatment Handbook",7th Edition, Lavoisier Publishing , Paris, Chapter 13 Sec.'s 3,5.

[12] Ibid. Chapter 17 Sec. 6.5.

[13] Tebbutt , T. H. Y. (1998) "Principle of Water Quality Control", 5th Edition , Butterworth-Heinemann , Oxford, pp.169-174.

[14] Ibid. p.206.

[15] Qasim,S.R. (1985) "Wastewater Treatment Plants Planning, Design and Operation", CBS College Publishing , New York, p.384.

[16] Ibid. p.388.

[17] Suntec Environmental, Inc .(2003) "UV Disinfection of Secondary Effluent , LPX200 System, NSF International/USEPA" , ETV Report 03/09/WQPC-SWP.

[18] Tchobanoglous,G., Loge,F., Darby,J. and M. Devries (1995) "Design of UV Disinfection System", USEPA 4th National Wastewater Treatment Technology Transfer Workshop , Kansas City.

[19] Kiely,G.(1997) "Environmental Engineering", Irwin/ McGraw-Hill, United Kingdom, p. 556.

[20] Wilson, F. (1981) "Design Calculations in Wastewater Treatment", E. & F.N. Spon, London, p.124.

Abbreviations

BOD_5: Five-day biochemical oxygen demand

DBP: Disinfection by-products

$\frac{F}{M}$: Food to microorganism ratio

FOG: Fat, oil and grease

MPN: Most probable number

S.S: Suspended solids

UV: Ultraviolet ray

WC: Water Column

ww: Wastewater

Appendix A

Flirtation system conceptual and basic design elements

- Filtration area

$$= \frac{12600\ (m^3/d)/24(h/d)}{6(\frac{m^3/h}{m^2})} = 87.5m^2.$$

- No. of filters and arrangement :

Three parallel filters, each having a filtration area of 30 m^2.

1) Each filter shall be constituted of two side-by-side compartments with a common in-between channel. Each compartment shall be 5×3 m.

The intermediate channel between any two compartments shall be divided into two parts. The lower part serves as the filtered water outlet during a filtration run or as the backwash water inlet during a backwash run. The upper part remains empty during a filtration run or serves as the backwash water outlet during a backwash run.

- Filtering medium, i.e. sand, specifications :

1) Effective size d_{10} =2.38 mm (U.S. sieve No.8)

   60% size d_{60} = 3.36 mm (U.S. sieve No.6)

   Coefficient of uniformity CU = $\frac{d60}{d10}$ =1.41.

2) Amount of sand :
   Sand depth = 1.2 m
   Sand volume =
   (1.2 m) (15$\frac{m^2}{compart.}$) =
   18 m^3/compartment
   Average relative density of sand = 2.6 (range 2.55 to 2.65)
   Mass of sand = (18) (2.6) =46.8 ≈ 47 tonne per compartment
   Total amount of sand = (47) (6) = 282 tonne.

3) Sand size breakdown, basis: one compartment
   (47000) (0.1) = 4700 kg passing through sieve No.8.

(47000)(0.5) = 23500 kg passing through sieve No.6 but blocked by sieve No.8.
(47000)(0.4) = 18800 kg passing through sieve No.4 (4.76 mm) but blocked by sieve No.6.

Sieve No.	Size Range (mm)	Mass %	Mass (kg)
4-6	4.76 to 3.36	40	18 800
6-8	3.36 to 2.38	50	23 500
<8	< 2.38	10	4 700
			∑= 47 000

- No. of long stem-plastic nozzles on false-floor

  On the false-floor of each compartment whose dimensions are 5×3 m, there shall be 24×14 = 336 nozzles with a nozzle spacing of 20 cm (center to center). Therefore, total number of nozzles for the filtration system shall be (336) (6) = 2016.

- Backwash run sequence and flow rates per filter

1) Air scour at a rate of 60 to 70 m^3/min for couple of minutes.
2) Air scour plus 12 to 18 m^3/min of backwash water for five to ten minutes.
3) Stopping air scour and doubling the backwash water rate for three minutes

During the backwash run, lasting 10 to 15 minutes, the filter influent is maintained ; acting as a cross-wash flow to sweep particles from the top of the filter bed in the two compartments (i.e. surface sweep) to the intermediate channel. The backwash plus surface sweep water should be recycled to the primary clarification system of the activated sludge process

Appendix B

UV irradiation system conceptual and basic design elements

- Determination of required UV dose :

  In UV irradiation system design, the indicator microorganism is total coliform. Faecal coliform count may constitute one-sixth to one-third of total coliform count (Kiely [19] ; Wilson [20]). For a total coliform count of 500 MPN/100 ml in a filtered secondary effluent, a UV dose of 20 mJ/cm^2 is required. This dose would result in a faecal coliform count of around 100 MPN/100 ml.

- Determination of exposure time required in UV bank :

  The UV dose (D) is related to the exposure time (t) by:
  D = (I) (t) , where I is the average UV intensity in the UV bank.

In the UV irradiation system considered, I may range from 4 to 14 mW/cm^2 . Taking the minimum I value of 4 mW/cm^2,

$t = \frac{D}{I} = \frac{20\ (mJ/cm^2)}{4\ (mW/cm^2)} = 5\ s$

- <u>Determination of flow rate in UV channel</u> :

  Average water velocity in UV bank

  $u = \frac{L}{t}$, wher L is the effective arc length of lamp.

  u = 1.47 (m)/5 (s) = 0.294 m/s.
  Considering aging and fouling of UV lamps during one-year operation, design average velocity is taken as one-half u. Therefore,

  $u_{design} = \frac{0.294}{2} = 0.147$ m/s

  Free cross-sectional flow area in bank = 0.33 m^2

  Design flow rate per channel = (0.147)(0.33) = 0.0485 m^3/s ≡4191 m^3/d (≅ 4200 m^3/d).

  Design flow rate per UV lamp =

  $\frac{(0.0485\,m^3/s)\left(1000\frac{l}{m^3}\right)(60\ s/min)}{(64\ lamp/bank)} = 45.5\ \frac{l/min}{lamp}$

  Which is a reasonable design figure.

- <u>Determination of head loss across diffusers and bank</u>:

  The flow area of the perforated plate diffuser is 5 to 10% of the channel flow area. Hence, a 0.6×0.6 m plate with 16×16, 13-mm holes, would result in a diffuser area which is 9.4 % of the channel flow area. The head loss per diffuser is calculated as follows:

  $h_L = K\frac{u_{hole}^2}{2g}$, where K = 1.56 the headloss coefficient

  and $u_{hole} = \frac{Q_{channel}\ /(16\times16)}{(\frac{\pi\times0.013^2}{4})}$,

  $Q_{channel}$ = 0.0485 m^3/s the channel flow rate.

  Therefore h_L per diffuser = 0.16 m WC.

  The head loss per UV bank is calculated as follows:

  $h_L = K\frac{u_{bank}^2}{2g}$, K = 1.8 , the bank head loss coefficient,

  u_{bank} = 0.147 m/s.

  Hence, $h_L = 1.98\times10^{-3}$ m or 2 mm WC which is negligible.

Appendix C

<u>Chlorination system conceptual and basic design elements per buffer basin for example case</u>

- NH_3 in filtered ww : 20 mg/l (assumed)
- NH_3 to be converted to NH_2Cl : 4 mg/l (assumed ; i.e. one-fifth of NH_3 present)
- Cl_2 amount required : 4×4 = 16 mg/l
- Chlorinated water flow = (0.25)(4200) = 1050 m^3/d = 43.75 m^3/h (12.15 lps)
- (cf. Note (1) below)
- Pipe size (pump, eductor) : 3" (velocity∼ 2.5 m/s)
- Nozzle size* : 1" (velocity ∼ 22 m/s)

Cl_2 flow from cylinder to eductor : about 200 mg/s (16 mg/l×12.15 l/s)

- Concentration of NH_2Cl in chlorinated ww : (4/17)(51.45) ≅ 12 mg/l
- Concentration of NH_2Cl in irrigation ww : (12)(0.25) = 3 mg/l

At the aforementioned Cl_2 consumption, a one-ton (907 kg) Cl_2 cylinder will last more than two weeks for the three basins (~600 mg Cl_2/s)

---

* In this example case, the nozzle is simply a 3" to 1" reducer.

Notes:

1) Fraction of chlorinated water flow from total water flow should be more than fraction of NH_3 converted to NH_2Cl to total NH_3 present to ensure excess NH_3 in chlorinated water (e.g. one-fourth vs. one-fifth in the above example)
2) Pump head to be determined following detailed piping design.

مفاهيم و عناصر التصميم الاساس لمراحل جعل مياه الصرف الصحي المعالجة صالحة لسقي المزروعات

الخلاصة:

ان شحة المياه والجفاف الذي يشهدهُ العراق ، خصوصاً اثناء اشهر الصيف الحارة، تجعل من الضروري على السلطات البلدية اعتماد مشاريع اعادة استعمال المياه المصرفة المعالجة كمياه الصرف الصحي . يبين هذا العمل مفاهيم وعناصر التصميم الاساس لمراحل جعل هكذا مياه صالحة لسقي المزروعات. حيث تشتمل هذه المراحل على الترشيح والمعالجة بالاشعة فوق البنفسجية ومن ثم الكلورة. تم اعتماد تجمع سكاني ريفي عدد نفوسه 50000 كمثال لتوضيح التصاميم المقترحة المتسمة بالبساطة النسبية والفاعلية. يهدف العمل الى 1) ايضاح المتطلبات و 2) الأفادة من المعلومات المقدمة للشروع بطوري التصاميم التفصيلية والتنفيذ لهذه المشاريع.