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# Study of the effects of on-site greywater reuse on municipal sewer systems

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# PREFACE

This report consists of two main chapters. The first chapter describes the quantification of the effects of greywater reuse on domestic wastewater quality and quantity on a single house scale. The second chapter uses the outcome of the first chapter as an input to a sewer system model and examines the effects of on-site greywater reuse on the municipal sewer network under differing scenarios of reuse targets and penetration ratios of greywater reuse systems.

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# QUANTIFYING THE EFFECTS OF GREYWATER REUSE ON DOMESTIC WASTEWATER QUALITY AND QUANTITY

## INTRODUCTION

Greywater (GW) in general terms is defined as domestic wastewater (WW) generated by the kitchen (KS), washing machine (WM), bathtub (BT), shower (SH) and wash basin (WB)). Blackwater is defined as toilet wastewater. In recent years, due to high pollutants loads, wastewater streams generated from the kitchen (i.e. from kitchen sinks (KS) and dishwashers (DW)), is either defined as dark-GW or included in the blackwater stream. Onsite GW reuse (GWR) for toilet flushing and garden irrigation, is believed to be beneficial in terms of reduction of urban water demand, in alleviating stress from depleted freshwater resources and potentially helping to minimise (or delay) the need to develop new (and costly) sources of drinking water (e.g. seawater desalination plants). Growing implementation of GWR practice may lead to benefits on the wastewater side of the urban water cycle.

Friedler (2008) demonstrated that onsite GWR in residential buildings in urban areas can reduce domestic water consumption in Israel by 28-33%. He further estimated that if the Israeli government would promote onsite GWR systems in new buildings, their penetration rate may reach 18-33% by 2023. With this given penetration rate, the overall possible water saving was estimated to be $30\text{-}54 \cdot 10^6$ $m^3$/year (Israel, 2023). This significant saving can increase to up to 20% of the total urban water consumption (penetration rate close to 100%). Friedler and Hadari (2006) demonstrated that under certain circumstances onsite GWR for toilet flushing can be economically worthy even to the consumer itself, depending on the treatment technology chosen, on the size of the served population and price of water. It should be noted that since their analysis, the price of potable water in Israel more than doubled.

Treatment of GW is necessary prior to reuse, in order to prevent hygienic and health risks (Almieda *et al.*, 1999; Dixon *et al.*, 1999; Diaper *et al.*, 2001 and others) and to minimize negative aesthetic effects (malodors and colors).

In a research carried out by Friedler (2004), it was advised that as the demand for GW within the urban environment (i.e. for toilet flushing and garden irrigation) is significantly lower than its production (i.e. the combines discharge of all the domestic GW streams), it is possible not to recycle all GW streams, but rather to treat and reuse the less polluted ones (i.e. GW generated from the shower, bathtub and wash basin). GW generated from the kitchen sink and washing machine, was thus advised to be discharged (without treatment) together with the blackwater stream to the urban WW system. This practice is generally expected to lead to lower treatment costs and to lower potential of negative health, environmental and aesthetic effect.

In a review held by Li *et al.*, 2009 three types of GW treatment technologies were signaled, namely: physical, chemical and biological. Physical technologies include especially filtration; this method removes only suspended solids and not dissolved ones (organic, nutrients and surfactants). Conventional physical treatment methods usually produce effluents of insufficient quality. According to this review, chemical technologies are efficient in removing suspended solids, organic materials and surfactants in low strength GW. Biological GW treatment technologies are generally based on aerobic biological treatment units, e.g. RBC (rotating biological contactor) and SBR (sequencing batch contactor). These technologies, according to Li *et al.*, are efficient for treatment of medium and high strength GW. The combination of aerobic biological process with physical filtration and disinfection is considered to be the most economic and feasible solution for GW recycling. MBRs (membrane bioreactors) appear to be a very attractive solution for medium and high strength GW recycling, particularly in collective / cluster urban residential units serving more than 500 inhabitants (Li *et al.*, 2009). In rural settlements, where land is more available then in densely populated urban areas, natural treatment systems such as constructed wetlands with horizontal and vertical flow regimes can be found. Excess sludge is a by-product of GW treatment and is usually released in to the municipal sewerage system (if exists).

Together with its positive effect in decreasing urban water demand, GW treatment and reuse changes the quantity and quality characteristics of domestic wastewater released to sewers and conveyed to wastewater treatment plants (WWTPs). As a result, positive and negative effects may influence sewer systems and WWTPs. Friedler and Hadari (2006) portray some of these effects. One of the positive effects postulated was that wastewater collection systems will consume less energy (for pumping sewage). Further, it was suggested that it might be possible to postpone enlargement of existing wastewater collection systems, and to construct smaller new ones. On the negative side, as a result of the reduction of flows released to the sewer system, it was proposed that more blockages might occur in the system. However, it was mentioned that this problem should not be substantial since many (or even the majority of) existing municipal sewers are maintained close to or even over their design capacity. Regarding WWTPs, some positive and negative effects were suggested: Lower loads of biodegradable pollutants are expected to reach the WWTP, while the loads of the non biodegradable pollutants will not change. However, as there will be less dilution, pollutants concentrations are expected to be higher. Further, the energy consumption of WWTPs may be lower, lesser amounts of chemicals may be consumed, and it might be possible to postpone enlargement of existing WWTPs and to construct the new ones smaller.

Studies on GWR to date focused mainly on the single-house scale (primarily on single family homes), paying much attention to different treatment systems and their performances. However the effects of GWR practice on domestic wastewater quantity and quality, and thus the consequent effects on urban wastewater conveyance systems and treatment plants were generally scarcely discussed. The first step towards quantifying the influence of GWR on the municipal wastewater section of the urban water cycle is quantification of the influences of GWR on domestic wastewater quality and quantity. This is the objective of the current paper. It should be mentioned that in order to assess the influence of GWR on urban wastewater conveyance, it is important to refer to the sub-daily diurnal flow patterns, rather then to the average diurnal flows. The reason for that is twofold, the first being the fact that the sewer system operates under unsteady flow conditions and the second being the fact that the sub-daily instantaneous flows are responsible for transport (or precipitation) of solids in sewers. On the other hand, when referring to influences on WWTPs performance, where

retention times are longer and the conditions are less variable, it is possible to refer to the average daily values.

## METHODS

*Conceptual description of a GW recycling house and a non-recycling house*

Three types of houses were conceptualized for the analysis, as follows:

**1.** Non-recycling house - the current situation, where no GWR is practiced and wastewater from all domestic sources runs to the sewer (Figure 1-B). From this house type a single WW stream is discharged to the sewer - the combined stream of all the WW streams produced in the house.

**2.** Recycling house - toilet flushing only, where GW from the bath (BT), shower (SH) and wash basin (WB) are treated and used for toilet flushing having dual flush cisterns of 9 L "full flush" and 6 L "half flush" (Figure 1-A). From this house three WW streams are discharged to the sewer:

I. A combined stream of blackwater and dark GW (generated from the KS and WM) that are not reused.

II. Overflow of excess light GW that is not reused and is discharged to the sewer without treatment.

III. Excess sludge produced by the GW treatment system.

**3.** Recycling house - toilet flushing and garden irrigation, same as type 2 house but with overflow (after treatment) used for garden irrigation during summer (Figure 1-A). During winter, when there is no need for irrigation, the overflow is discharged to the sewer without treatment as in type 2 house.

Based on these three types of houses described, three scenarios were examined.

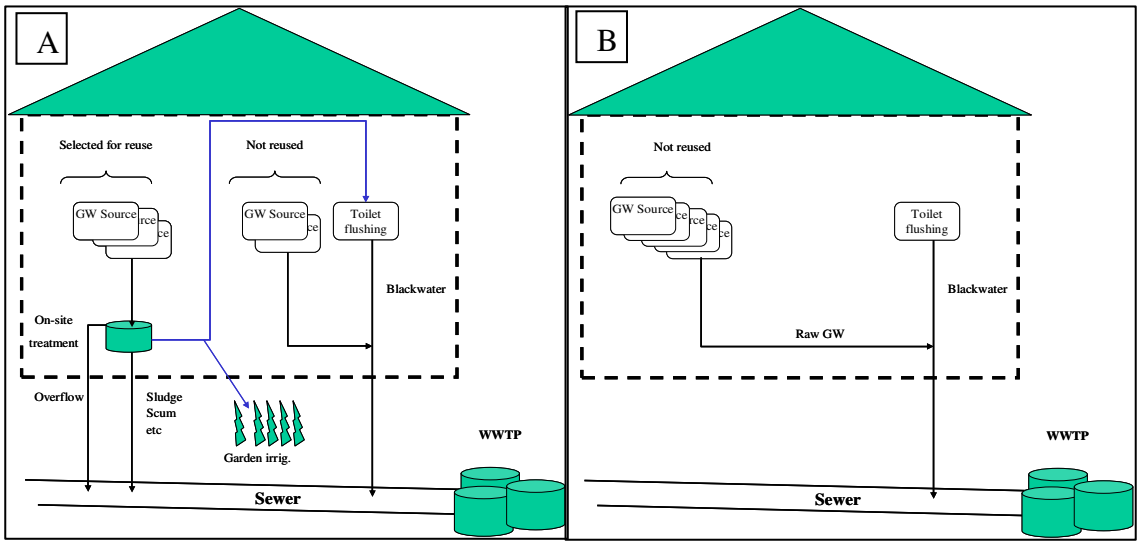


**Figure 1 - Conceptual description of wastewater streams in a GWR house (light GW) (A) and a non-recycling house (B)**

*GW treatment method*

As aforesaid, various methods for treating GW onsite are mentioned in the literature. In both type 2 and type 3 houses the conceptual GW treatment applied was biological treatment by an RBC (rotating biological contactor) based system. An RBC unit is suitable for dense urban areas due to its relatively small space requirements and its low energy consumption. Values of treated GW quality parameters were taken from data obtained from an experimental pilot-scale RBC based unit, situated at a married couples dormitory in the Technion – Israel Institute of Technology – and operated for more than tree years (Friedler *et al.*, 2005; Kovalio, 2005; Gilboa and Friedler, 2008; Aizenchtadt *et al.*, 2009). These were considered as the baseline quality characteristics for toilet flushing and for toilet flushing and garden irrigation in the second and third scenarios, respectively. The abovementioned treatment system consisted of a fine screen, an equalization basin, an RBC unit followed by a sedimentation basin, and disinfection. Detailed description of the treatment system can be found in Friedler *et al.* (2005, 2006).

*Flow pattern analysis*

Person equivalent (PE) diurnal flow patterns, referring to wastewater discharges during weekdays, from wastewater generating household appliances (kitchen sink (KS), wash

basin (WB), bath (BT), shower (SH) and washing machine (WM)), were derived from a 10 minutes interval dataset obtained from previous work performed in single houses in England (Butler *et al.*, 1995). Data on domestic toilet usage was taken from Friedler *et al.* (1996-a, b). The data, on toilet usage at home, referred to the instantaneous (10 min interval) number of incidents throughout the day, and to the character of the incident. The incidents are distinguished by four groups:

I. Urine only.
II. Faeces only.
III. Combination of one and two.
IV. Other then the above (e.g. toilet cleaning, waste disposal and flushing etc.)

The distinction between these four different toilet use incidents is important since the type of use determines the flush volume to be implemented, namely: half flush (6 L) for urine incident and full flush (9 L) for all other incidents.

The momentary flows generated by full flushes (week days) were calculated by multiplying the full flush volume by the sum of the instantaneous type 2, 3 and 4 toilet incidents (Eq. 1).

**1)** $$Q_{WC,full\_flush(i)} = \left(T_{2wd(i)} + T_{3wd(i)} + T_{0wd(i)}\right) \times V_{full\_flush}$$

Where: $Q_{WC,full\_flush(i)}$ is the momentary (at time interval i) WC full flush flow on week days (L/PE/10 min), $T_{2wd(i)}$ is the momentary number of faeces incidents on week days (flushes/PE/10 min), $T_{3wd(i)}$ is the momentary number of faeces and urine incidents on week days (flushes/PE/10 min), $T_{0wd(i)}$ is the momentary number of "other" (not urine or faeces) incidents on week days (flushes/PE/10 min), and $V_{full\_flush}$ is the volume of a full flush (9 L/flush).

Similarly the momentary, week days, half flush flows were calculated by Eq. 2:

**2)** $$Q_{WC,half\_flush(i)} = \left(T_{1wd(i)}\right) \times V_{half\_flush}$$

Where: $Q_{WC,half\_flush(i)}$ is the momentary WC half flush flow on week days (L/PE/10 min), $T_{1wd(i)}$ is the momentary number of urine incidents on week days (flushes/PE/10 min), and $V_{half\_flush}$ is the volume of half a flush (6 L/flush).

Since Israeli domestic water consumption is somewhat higher then in England, and since all flows were obtained from studies conducted in England, the calculated flows from the WB, SH, and BT were multiplied by a factor of 1.4, and the from the KS by 2, in order to better represent daily domestic wastewater discharges in Israel (flows from the WM did not changes). This is based on the assumption that the diurnal patterns of water use, which are dependent on lifestyle, are similar in both countries.

The storage volume needed for ensuring reliable supply of treated GW for toilet flushing was determined by iterations (trial and error), ensuring that the calculated momentary storage volume in the tank will never become negative (Eq. 3).

$$\textbf{3)}\quad V_{(i)} = \begin{bmatrix} V_{(0)} & V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} \geq V_{(0)} \\ \begin{pmatrix} 0 \\ V_{(0)} = V_{(0)} + \left| V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} \right| \end{pmatrix} & V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} < 0 \\ V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} & else \end{bmatrix}$$

Where: Initial condition determine that at time i=0 the storage tank is full; $V_{(0)}$ is the required storage volume (L/PE) i.e. this is the minimum possible volume (found to be 1.2 L/PE by iterative simulation); $V_{(i)}$ is the momentary (10 min. interval) volume of light GW in the storage tank (L/PE); $Q\_lightGW_{(i)}$ is the momentary flow of the light GW (L/PE/10 min); $Q\_WC_{(i)}$ is the momentary water demand for toilet flushing (L/PE/10 min).

GW overflow was then calculated, through a balance between stored water, treated GW generation and water demand for toilet flushing (Eq. 4). Where $Q_{o.f.(i)}$ is the momentary overflow (L/PE/10 min).

$$\textbf{4)}\qquad Q_{o.f.(i)} = \begin{pmatrix} V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} - V_{(0)} & V_{(i-1)} + Q\_lightGW_{(i)} - Q\_WC_{(i)} > V_{(0)} \\ 0 & else \end{pmatrix}$$

In summary: The momentary WW flow discharged from a single house to the sewer (in the case of GWR for toilet flushing) in each time interval, is calculated as the sum of the

flows generated from toilet flushing, the kitchen (sink and dishwasher), the washing machine and from overflow of raw light GW.

*Quality analysis*

Typical qualities of each appliance discharge were extracted from an extensive survey of Israeli homes (Friedler 2004). Quality data of toilet wastewater i.e. fecal and urine flushes (full and half flushes) were taken from a survey preformed in England (Friedler et al., 1996-b; Almeida *et al*., 1999). The main quality parameters of interest for the onsite GW treatment, the wastewater collection system and the WWTP are: $COD_t$, $BOD_t$, TSS, N-$NH_4$, TKN and P-$PO_4$. The average concentrations of these pollutants, at the outlet of each source, are presented in Table 1.

At first each stream was analyzed separately, then, all streams were combined into one single stream discharged to the sewer according to the three scenarios mentioned above. For each stream, momentary loads of each pollutant were derived from the product of the momentary flow by the average concentrations, found in Table 1. Later these loads were summed up to render the total daily loads released to the sewer.

The momentary loads of each pollutant in the untreated GW overflow (overflow released to the sewer in scenario 2), were calculated by Eq. 5 and 6. Where: $M_{o.f(i)}$ is the momentary pollutant load in the overflow (mg/PE/10 min), $Q_{in(i)}$ is the total momentary light GW flow (L/PE/10 min), $Q_{WB(i)}$, $Q_{BT(i)}$ and $Q_{SH(i)}$ are the momentary flows of the washbasin, the bath tab and the shower respectively (L/PE/10 min), $C_{WB}$, $C_{BT}$ and $C_{SH}$ are the pollutants concentration (mg/L) in the washbasin, the bath tab and the shower respectively (as presented in Table 1), and $Q_{o.f(i)}$ is the momentary overflow rate (L/PE/10 min). The assumption used was that the stored overflow volume is small enough, and that its residence time in the storage tank is short enough, thus it can be estimated that concentrations in the inlet of the storage tank equals the concentrations in its outlet.

**5)** $$Q_{in(i)} = Q_{WB(i)} + Q_{BT(i)} + Q_{SH(i)}$$

**6)** $$M_{o.f.(i)} = \left( \frac{Q_{WB(i)} \cdot C_{WB} + Q_{BT(i)} \cdot C_{BT} + Q_{SH(i)} \cdot C_{SH}}{Q_{in(i)}} \right) \cdot Q_{o.f}$$

**Table 1- Average concentrations of the selected pollutants in raw domestic WW streams discharged from in-house WW generating appliances**

| | | KS | WM | WB | BT | SH | TOILET half flush (6 L) | TOILET full flush (9 L) |
|---|---|---|---|---|---|---|---|---|
| | Source | | | 1 | | | 2 | |
| TSS | mg/L | 625 | 188 | 259 | 78 | 303 | 745 | 3,113 |
| COD t | mg/L | 1,340 | 1,339 | 386 | 230 | 641 | 658 | 3,972 |
| BOD t | mg/L | 890 | 462 | 205 | 173 | 424 | 241 | 1,476 |
| NH4-N | mg/L | 0.6 | 4.9 | 0.4 | 0.9 | 1.2 | 71.5 | 72.1 |
| PO4-P | mg/L | 21.6 | 169 | 15 | 4.6 | 10 | 159 | 185 |
| TKN | mg/L | 20.4 | 56 | 6 | 12 | 14.4 | 520 | 380 |

[1.] Friedler 2004; [2.] Friedler *et al.*, 1996 (b), Almeida *et al.*, 1999

Surplus of stored GW is transformed into overflow, as explained above.

- In scenario 2 (GWR for toilet flushing) overflow is released to the sewer without treatment and hence its quality is the weighted average of the quality of its components, raw light GW (SH, WB and BT).
- In scenario 3 (GWR for toilet flushing and irrigation), as the overflow is used for irrigation after treatment, its quality is the quality of the treated GW.

*Sludge production*

Sludge production is an integral part of the biological treatment. As mentioned above the model for the biological treatment is an RBC system. In order to evaluate sludge production by the RBC, daily balance of the COD load removed ($L_{COD}$) was calculated (Eq.7). Where the $L_{COD}$ is the daily removed COD load (mg COD/(PE•d)), $C_o$ and $C_e$ are COD concentrations in the raw and treated light GW respectively (mg COD/L), , and $Q_t$ is the instantaneous WW flow (L/PE/10 min).

7) $$L_{COD} = \sum_{t=0:00}^{t=23:50} Q_t \left(C_o - C_e\right)$$

$L_{COD}$ was transformed into the daily excess sludge load, $M_{VSS}$ (mg VSS/(PE•d)), by Eq. 8. Where the volatile suspended solids load ($M_{VSS}$), is a good approximation to the excess total sludge load, and $Y_{obs}$ is the "observed" yield ("net" yield - yield minus biomass decay rate). For the studied RBC system the $Y_{obs}$ was found to be 0.18 mgVSS/mg COD (Kovalio, 2005)

8) $M_{VSS} = L_{COD} \cdot Y_{obs}$

The daily sludge volume $V_{sludge}$ (ml/(PE•d)) was calculated by Eq. **9** and **10**. Where the sludge volume index (SVI), of RBC sludge was assumed to be 75 ml/gr TSS (Tawfik *et al.*, 2006), the VSS/TSS ratio in the sludge was found to be 0.73 (Kovalio, 2005), and $M_{TSS}$ is the total suspended solids load, discharged to the sewer (mg TSS/(PE•d))

9) $M_{TSS} = M_{VSS} / 0.73$

10) $V_{sludge} = SVI \cdot M_{TSS} \cdot 10^{-3}$

Based on a general stoichiometric formula of bacterial cells in biofilms ($C_5H_7O_2NP_{0.1}$; Metcalf and Eddy, 2003), nitrogen and phosphorus daily loads contributed by the excess sludge were calculated (Eq. 11 and 12, respectively). Where, $L_N$ and $L_p$ are the daily nitrogen and phosphorus loads in the sludge (mg N/(PE•d) and mg P/(PE•d) respectively).

11) $C_5H_7O_2NP_{0.1} \rightarrow L_N = M_{VSS} \cdot \frac{1 \cdot 14}{116.1} = M_{VSS} \cdot 0.12$

12) $C_5H_7O_2NP_{0.1} \rightarrow L_P = M_{VSS} \cdot \frac{0.1 \cdot 30.97}{116.1} = M_{VSS} \cdot 0.03$

Sludge discharge calculation was based on once a day discharge occurring between 7:30-8:00, during the morning WW generation peak. Sludge contains high solids concentrations, hence releasing it at this peak flow time is expected to prevent excess blockages.

## RESULTS AND DISCUSSION

*Flow pattern analysis*

Table 2 presents a summary of the daily flows released to the sewer. The flows presented are the ones calculated in this study and the range found in other studies. It can be seen that flows in this study lie well within the range reported in the literature.

**Table 2- Daily flows released to sewers from in-house WW generating appliances - No GWR (L/(PE·d)).**

| | KS | WM | WB | BT | SH | WC | Total | Light GW | GW |
|---|---|---|---|---|---|---|---|---|---|
| This study | 26.6 | 16.6 | 18 | 22.4 | 16.8 | 37.7 | 138.1 | 57.2 | 100.4 |
| Literature | 5.2-30 | 7.1-60 | 9-15 | 14.9-24 | 10-20 | 29-60 | 69-153 | 19-57.6 | 32-150 |

Literature sources: Parkinson *et al.*, 2007; Vierira *et al.*, 2007; Wheatley and Surendran, 2008; Eriksson *et al.*, 2008; Li *et al.*, 2009, Meinzinger and Oldenburg, 2009

Figure 2 describes the diurnal variations in the domestic per capita flows for the three scenarios examined: 1- current situation - no GWR, 2- light GWR for toilet flushing, and 3- light GWR for toilet flushing and garden irrigation. In scenarios 2 and 3 the reduction in flows is not constant throughout the day, but occurs mainly at times of peak water consumption, i.e. during the morning and the evening peaks, between 6:30 and 10:00 and between 17:30 and 22:00. During the morning peak WW flows are reduced by 13-53% and 43-58% in scenarios 2 and 3, respectively. During the evening peak the reduction amounts to 24-36% and 29-39%, in scenarios 2 and 3 respectively. It should be mentioned that these reduced flows are greater than flows discharged to the sewer during the low-flow periods of the afternoon in a non-reusing house. As aforesaid, between 7:30-8:00 sludge is released and its total volume is 0.25 and 0.38 L/(PE•d) in scenarios 2 and 3, respectively. From here the momentary flows contributed by the released sludge are 0.08 and 0.13 L/PE/10min in scenarios 2 and 3 respectively (for half an hour).

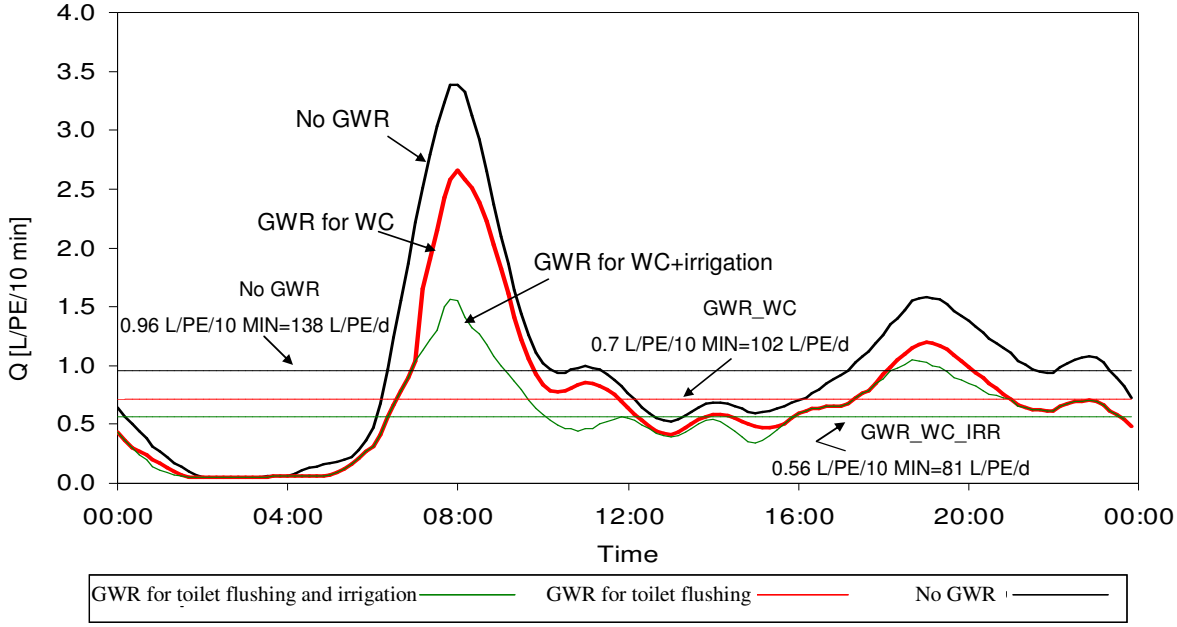


**Figure 2 - Diurnal patterns of domestic WW discharges.**

*Analysis of the influences of GWR on the quality of WW discharged to the sewer*

Table 3 presents the daily loads and average concentrations of the examined pollutants in the different WW streams discharged from a residential house: light GW (SH, BT and WB), dark GW + blackwater (KS, WM and WC) and total GW (KS, WM, SH, BT and WB). Daily flows, loads and average concentrations, under the three scenarios analyzed are depicted in Figure 3. Under the current situation (Figure 3-A), with toilet flush volumes of 9 and 6 L the average daily WC usage amounts to 38 L/(PE·d), and the total daily wastewater discharge to the sewer is 138 L/(PE·d). The introduction of a GWR system, treating the light GW (Figure 3-B), results in a reduction of the daily wastewater discharge to 102 L/(PE·d), 26% reduction as compared with scenario 1. Using the overflow for irrigation further reduces the discharge to 81 L/(PE·d) (41% reduction from baseline scenario). In this scenario the treated light GW overflow contributes 21.2 L/(PE·d) for irrigation. With the average of 3.33 residents in an Israeli home (Israel Central Bureau of Statistics), one house can contribute about 71 L/house hold/d of treated light GW for irrigation which amounts to about 13 $m^3$ during an irrigation season, that usually spans early May until late October.

**Table 3- Daily loads and average concentrations of chosen pollutants in raw light GW, dark GW + blackwater, and in raw combined GW stream**

| Parameter | | Light GW (SH+BT+WB) | | Dark GW +WC (KS+WM+WC) | GW (Total) | |
|---|---|---|---|---|---|---|
| | | This study | Literature | This study | This study | Literature |
| $COD_t$ | g/(PE·d) | 23 | | 125 | 81 | 7-102 |
| | mg/L | 400 | 100-633 | | 804 | 78-1,110 |
| $BOD_t$ | g/(PE·d) | 15 | | 56 | 46 | 1-47 |
| | mg/L | 257 | 50-300 | | 458 | 41-527 |
| TSS | g/(PE·d) | 9.0 | | 78 | 29 | 12.7-29 |
| | mg/L | 157 | 7-505 | | 286 | 25-697 |
| $NH^4$-N | g/(PE·d) | 0.05 | | 2.8 | 0.1 | 0.1-0.2 |
| | mg/L | 0.8 | 0.8-3.8 | | 1.4 | 1-92 |
| $PO_4$-P | g/(PE·d) | 0.54 | | 9.7 | 3.9 | 0.1-5.9 |
| | mg/L | 9.4 | 0.1-48.8 | | 39 | 0.1-70 |
| TKN | g/(PE·d) | 0.62 | | 19 | 2.1 | 0.1-1.7 |
| | mg/L | 118 | 3.6-19 | | 218 | 1.7-34 |

Literature: Parkinson *et al*., 2007; Halalsheh *et al*., 2008; Meinzinger and Oldenburg 2009; Li *et al*., 2009

The analysis revealed that, in contrary to the initial assumption, reusing light GW stream for toilet flushing and for toilet flushing and garden irrigation did not significantly reduce daily pollutants loads, released to the sewer (generally only by few percents) (Table 4). The main reason for this lies in the fact that the daily pollutants loads in the untreated light GW (SH, BT and WB) are relatively low, compared with their daily loads in the other domestic wastewater streams (KS, WM and WC), especially from the WC (Table 3). Another reason for this small decrease is that some of the pollutants, which were removed from the treated light GW return to the sewer with the sludge. For example in scenarios 2 and 3 the sludge contributes 3.3 and 5.1 g/(PE·d) respectively to the daily $COD_t$ loads.

As shown in Figure 2, as the reuse is grater, the flows are reduced , and thus the average daily concentrations (of all of pollutants) rise by about 30% in scenario 2 and by ~50-70% in scenario 3 (Table 4). For example, the total suspended solids (TSS) concentration increases by 31% when light GW is reused for toilet flushing, and by 62% when it is reused for toilet flushing and garden irrigation.

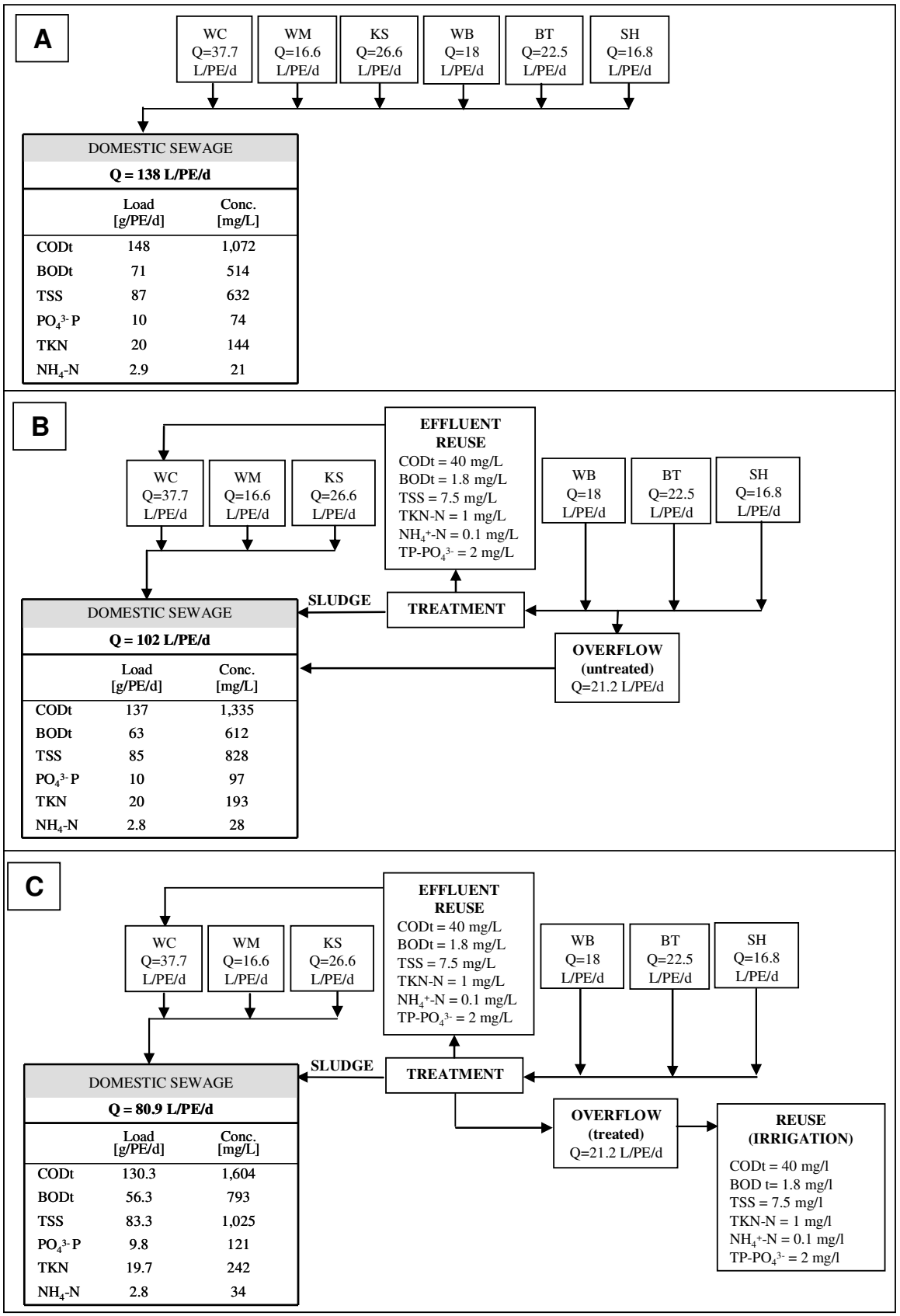


**Figure 3 - Flows and qualities of domestic WW discharges (A) No GWR, (B) Light GWR for toilet flushing, (C) Light GWR for toilet flushing and garden irrigation.**

**Table 4 - Relative decrease in daily pollutants loads and relative increase in their average concentrations in scenarios 2 and 3 as compared with scenario 1**

| | Load decrease (%) | | Concentration increase (%) | |
|---|---|---|---|---|
| **Scenario** | **2** | **3** | **2** | **3** |
| $COD_t$ | 8 | 12 | 25 | 50 |
| $BOD_t$ | 12 | 21 | 19 | 35 |
| TSS | 3 | 4.6 | 31 | 62 |
| $PO_4^{3-}$-P | 3 | 4 | 31 | 63 |
| TKN | < 1 | 1 | 34 | 69 |
| $NH_4^+$ | 1 | 2 | 34 | 67 |

2 – GW reuse for Toilet flushing; 3 – GW reuse for toilet flushing & garden irrigation

Figure 4 depicts the diurnal variations in pollutants concentrations ($COD_t$, $BOD_t$, TSS, $NH_4^+$, $PO_4^{3-}$ and TKN). As aforesaid in a GW reusing house pollutants concentrations are higher then in a non reusing house. For most pollutants the main changes occur during the morning peak hours. The reason is that during this time the domestic WW in a non reusing house are composed mainly of black water (contributed by toilets), and of light GW, (contributed by the shower and wash basin) (Figure 5 and Figure 6). Pollutants concentrations in the light GW stream are relatively low. Hence, when the light GW stream is taken out from the domestic WW stream and reused for toilet flushing the dilution effect of light GW, that occurs in a non-reusing house disappears. In a house reusing GW also for irrigation, this effect increases, since no light GW overflow is discharged to the sewer**.** On the other hand, no significant changes during the morning peak were observed for $PO_4^{3-}$, $NH_4^+$ and TKN (Figure 4 - D, E, F), changes in their concentrations were rather spread along the whole day, with a somewhat higher change during the evening.

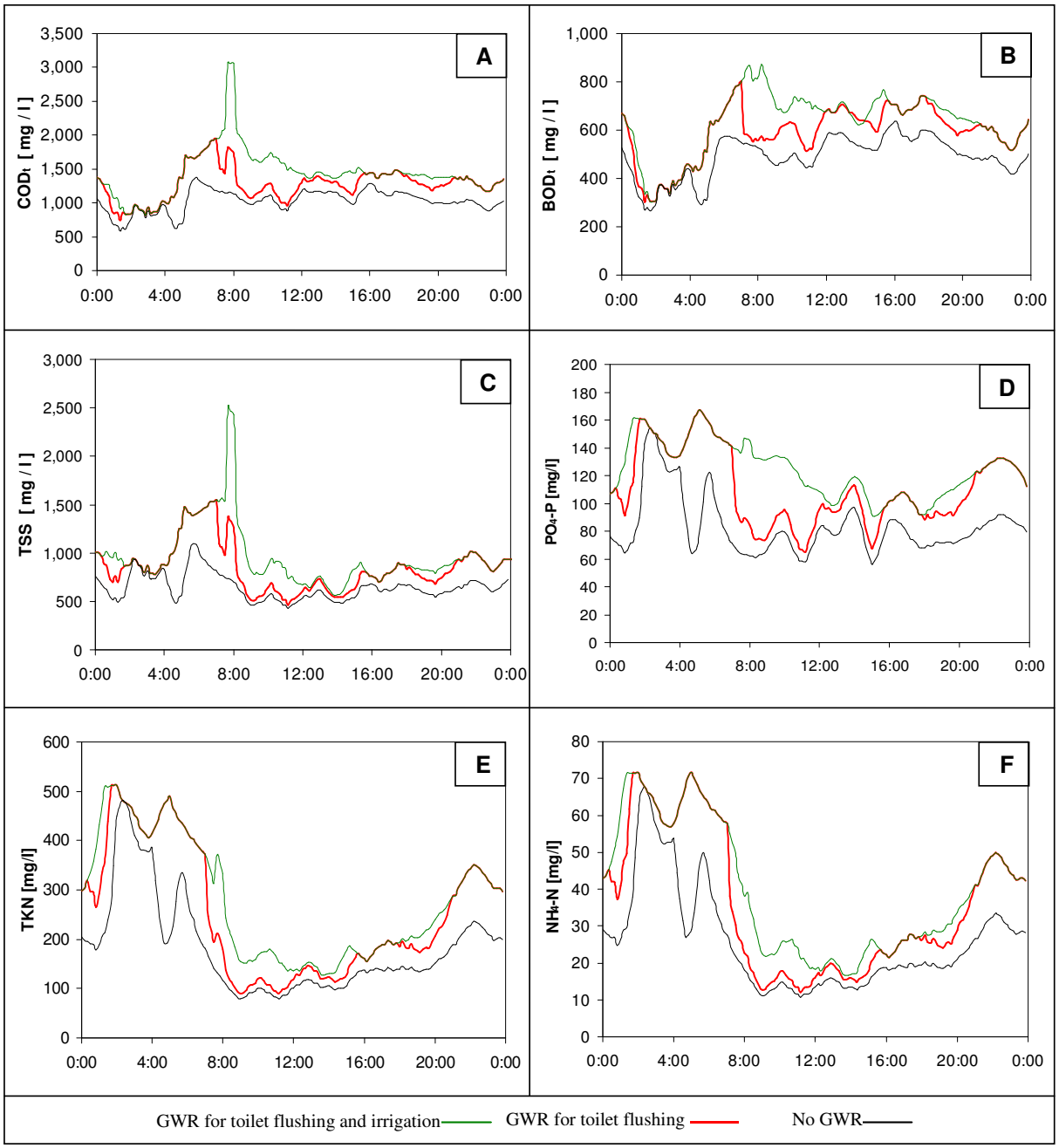


**Figure 4 - Diurnal patterns of selected pollutants concentrations in domestic WW discharged to municipal sewers**

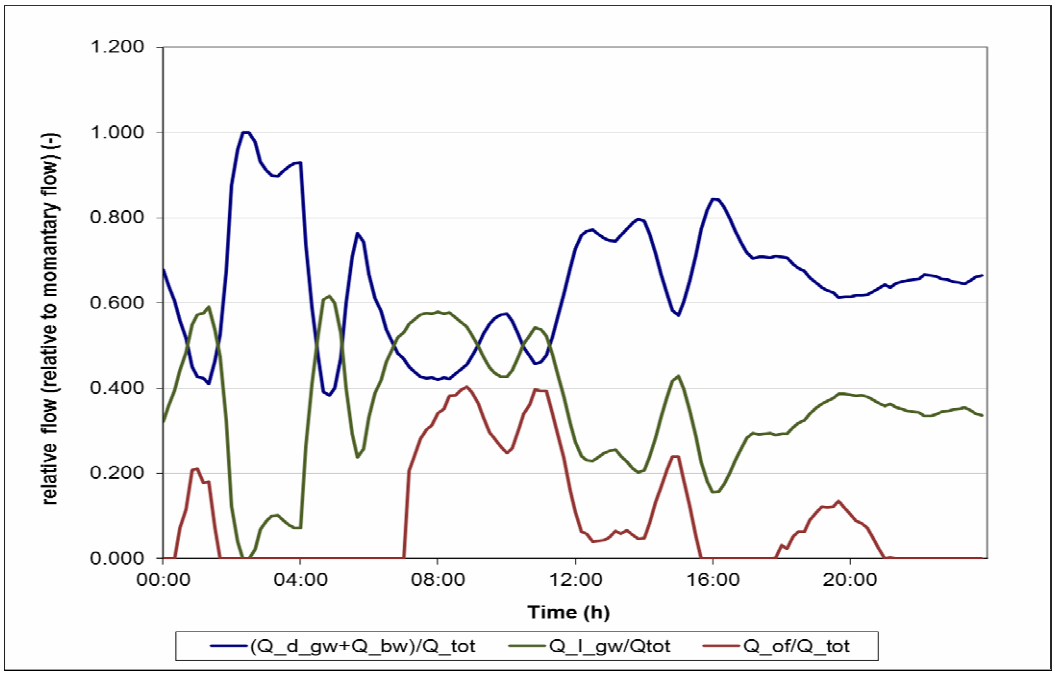


**Figure 5 - Diurnal patterns of domestic relative WW discharges, where Q_d_gw, Q_bw, Q_l_gw and Q_tot are the momentary; dark GW (i.e. KS and WM), black water (i.e. WC), light GW (i.e. BT, SH and WB) and total domestic WW flows discharged to the sewer, respectively**

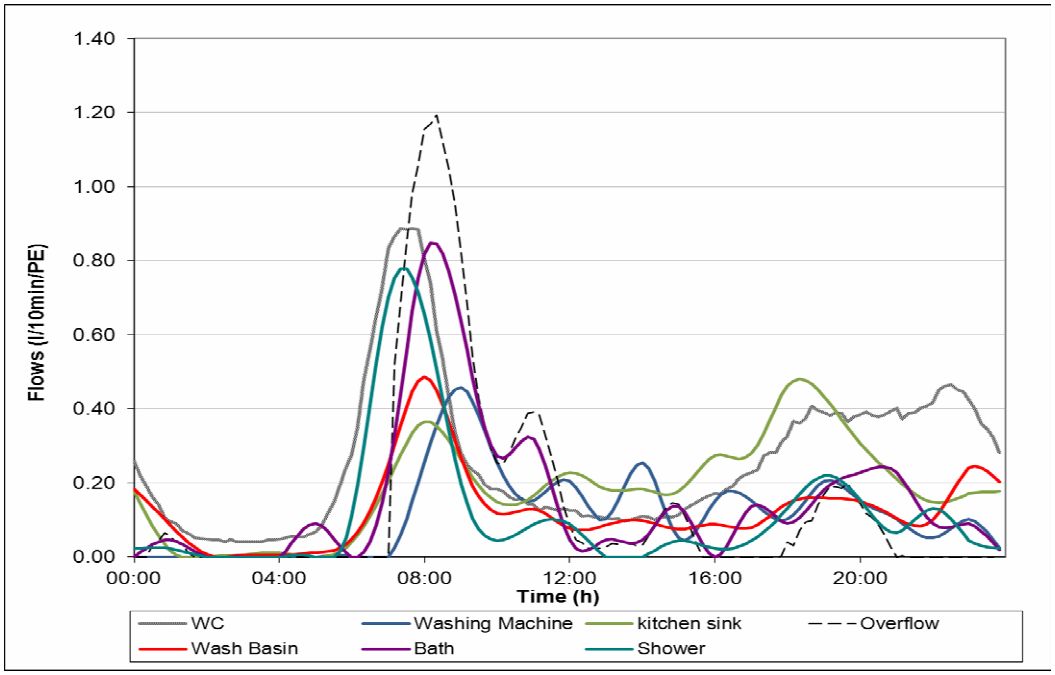


**Figure 6 - Diurnal patterns of domestic WW discharges from the different sources in the house**

## CONCLUSIONS – EFFECTS OF GREYWATER REUSE ON DOMESTIC WASTEWATER

The introduction of a GWR system, treating light GW originating from bathtubs, showers and wash basins, results in a reduction of the daily household water consumption of 26%. Using the overflow of light GW for garden irrigation further reduces the daily water demand to an overall reduction of 41%.

Reusing the light GW stream for toilet flushing, and for toilet flushing and irrigation, does not significantly reduce daily pollutants loads, of organic matter and nutrients discharged to the sewer. This is due to the fact that the main contributors of these pollutants are the toilet, the kitchen sink and dishwasher, and the washing machine and not the bath, shower and washbasin.

As the reuse becomes grater, domestic wastewater flows into sewers are considerably reduced. Thus, the average daily concentrations (of all pollutants) rise by ~30% in the case of GWR for toilet flushing; and by an average of ~60% in the case of reuse for toilet flushing and irrigation. This is due to the lesser dilution effect of light GW which is the less polluted domestic wastewater stream. Nevertheless, it should be noted that for most pollutants the concentration increase is lower than the decrease of the WW quantity, this is due degradation during the treatment process of the light GW before it is reused.

The main reduction in flows occurs, at times of peak wastewater generation. During these periods the majority of the flow originates from toilet flushing and bathing (BT + SH). Thus, when GWR is practiced, during these periods GW is not discharged from the house but rather reused for toilet flushing. During the morning peak, WW flows are reduced by 13-53% and by 43-58% in scenarios 2 (GWR for toilet flushing) and 3 (GWR for toilet flushing and garden irrigation) respectively (as compared with a non-reusing house), while during evening peak flows are reduced by 24-36% and 29-37% in scenario 2 and 3 respectively. Nevertheless, these reduced peak flows were found to be higher than the discharges of a non-reusing house during the afternoon low-flow period. This phenomenon may indicate that the chances for higher blockages rate in the sewer system may be minimal.

Daily loads of $COD_t$ and $BOD_t$ were found to decrease by ~10% and ~20% when GWR for toilet flushing and GWR for toilet flushing and garden irrigation were practiced, respectively. On the other hand, much smaller decrease of TSS, TKN, $PO_4^{3-}$ and $NH_4^+$ loads was observed. Although the loads of all pollutants decreased their concentrations in the wastewater discharged increased due to lower dilution by the light GW stream in which pollutants concentrations are usually lower than in the other domestic wastewater streams. For most pollutants, the highest concentration increase occurs during the morning peak flow period, coinciding with the highest flow reduction during this period.

The findings of this study are a first step towards quantification of the possible effects of onsite GWR on sewer systems. Further research is needed in order to arrive at substantiated conclusions on the extent of the potential positive and/or negative effects that onsite GWR systems, if implemented on a large scale, in the urban sector, may have on the urban wastewater conveyance systems and WWTPs.

# MODELING THE EFFECTS OF ON-SITE GREYWATER REUSE ON MUNICIPAL SEWER SYSTEMS

## INTRODUCTION

This chapter examines the influences (positive and/or negative of the changes in flows and pollutant concentrations discharged from a single house on the urban sewer system. Urban sewer systems function under of non-uniform unsteady flow conditions. Hence, when examining the influences of onsite GWR on the sewer system, there is no significance to average flows, concentrations and loads, but rather these influences should be examined dynamically throughout the day. For this, a dynamic simulation model of the sewer system was used, examining different scenarios. For simplification and in order to receive general overview hydrologic models were set up, simulated and analyzed.

## GENERAL DESCRIPTION OF THE MODELS

*SIMBA models*

The simulation model chosen was SIMBA6, developed by *ifak* (Institute for automation and communication) Magdeburg, Germany. This model is found in great usage in Europe especially in Germany, by research institutes and by engineer practices. A selection of licensees such as:

Water companies: Aggeverband Germany, Bergisch-Rheinischer Wasserverband, Germany, Berliner Wasser Betriebe, Germany, Waterschap Rijn en Ijssel, The Netherlands and more. Municipalities in Germany: Landesumweltamt Nordrhein-Westfalen, Staatliches Umweltamt Aachen and more. Equipment Suppliers: BASF, Germany, Bayer, Germany, CEGELEC, England, Gebrüder Hunziker, Suisse, Nishihara Environmental Technology, Japan, PURAC Lund, Sweden and more.

Research and Development: RWTH Aachen, Germany, Faculty of Electrical Engineering, Bellville, South Africa, University of Birmingham, England and more.
All use SIMBA6

SIMBA6 is integrated modeling and simulation software for sewer systems and WWTPs. The model is suitable for modeling separate sewers and combined sewers. The SIMBA6 model enables to divide the area contributing WW to the sewer system, into sub catchments. Further it enables to allocate to each sub catchment its specific WW hydrographs and pollutographs, this in a sub daily resolution. The model enables to run short term simulations (hours) and long term simulations (days-months).

The urban sewer system can be modeled in SIMBA by two approaches: simplified (hydrologic) and detailed (fully hydrodynamic).

1. Hydrologic modeling: relatively simple and fast. This approach enables simple simulations of the sewer system, in order to receive information regarding the main processes occurring in it. The hydrologic models enable:
   - Translations of flows and pollutants and their attenuation, by inserting attenuation functions. These functions are common in hydrology (e.g. Nash Cascades).
   - The pollutants sources can be one (or more) of the following: dry weather flow, WW flows from a specific source (in this case domestic WW) and rain. An unlimited number of pollutants can be simulated.
2. Hydrodynamic modeling: A more detailed modeling approach, which requires a large amount of data and large computer resources (much longer simulation times). The main characteristic of the hydrodynamic modeling:
   - Based on SWMM (Storm water management model) developed by the USEPA.
   - Applies the diffusive wave approximation of the solution of the Saint Venant differential equations.

*The chosen neighborhood*

The influences of GWR on the sewer system were examined on one representative real neighborhood. The chosen neighborhood is a flat densely populated neighborhood, located on the coast in central Israel. This neighborhood is under construction and hence precise data of its sewer system was available (lengths, heights, diameters, locations of manholes etc.). The total length of the neighborhoods sewer pipes is approximately 6 km. A schematic layout of the sewer system can be seen in Figure 7. The

neighborhood's sewer system is a separate sewer. The sewer pipes manning coefficient was 0.013.

The exact number of residents in the neighborhood was not available; hence estimation had to be made. Two different approaches were used for this purpose, and then a reasonable estimation was made.

<u>Firsts approach</u>: According to the information advertised in the web site of the municipality of the city which the neighborhood belongs to, about 60 buildings with 22 floors each are planned to be built in the neighborhood. An assumption was made that there are 3 apartments in each floor. With the assumption of 3.2 residents in an apartment (a reasonable assumption for Israel homes one can calculate 12,672 residents live in the neighborhood):

$$22\frac{floors}{building} \times 3\frac{apartments}{floor} \times 3.2\frac{residents}{apartment} \times 60\frac{building}{neighborhood} = 12,672\frac{residents}{neighborhood}$$

<u>Second approach</u>: An estimated calculation of the neighborhood area, results in about 30 ha, with the assumption of 5PE/ha (a good assumption for highly densely populated cities will be 4-6 PE/ha), it sums up to 15,000 residents.

Following these approaches, a reasonable assumption of 15,000 residents was made.
An "as made" plan (plan of a sewer network as it has been constructed) for the neighborhoods sewer system was used for the base of the modeling.

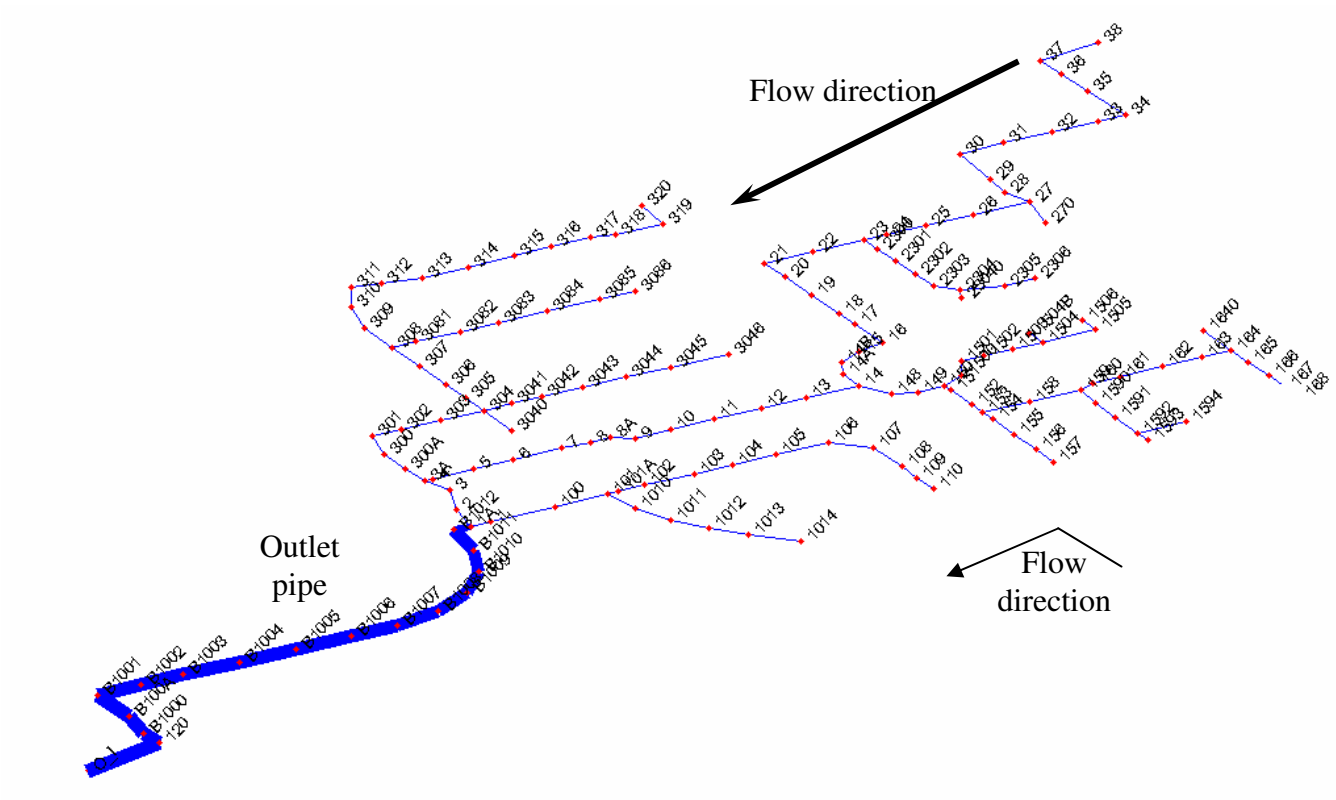


**Figure 7 - Layout of the sewer network, with its nodes, and their names**

## HYDROLOGIC MODELS

*Methods*

Six scenarios were formulated and later examined (Table 5). All scenarios are for separate sewer systems. These scenarios are based on the three types of houses described in the previous chapter (1. A non-recycling house, 2. A house reusing light GW for toilet flushing and 3. A house reusing for toilet flushing and for irrigation). Three types of scenarios were examined:

1. Extreme situation - no GWR, all houses reuse their light GW for toilet flushing, all houses reuse for toilet flushing and for irrigation.
2. To be expected situations - scenarios that are to be expected to occur in the future.
3. To be expected situations - scenarios that are to be expected to occur as a result of strong promotion of the government.

**Table 5 - Scenarios for GWR examined**

| | | Current situation | Extreme situation | | To be expected | | With strong promotion |
|---|---|---|---|---|---|---|---|
| | | | Case 1 | Case 2 | Case 1 | Case 2 | |
| **Type of GWR and implementation proportion in each scenario** | (1)* NR | 100% | 0% | 0% | 70% | 70% | 40% |
| | (2)* RWC | 0% | 100% | 0% | 30% | 15% | 30% |
| | (3)* RWC+IR | 0% | 0% | 100% | 0% | 15% | 30% |
| **Type of city** | Flat / Densely populated | 1** | 2** | 3** | 4** | 5** | 6** |

* Type of GWR:

(1) NR -No greywater recycling is practiced and wastewater from all sources runs to the sewage.

(2) RWC - Greywater from the bath (BT), shower (SH) and wash basin (WB) (i.e. the light GW) are treated and used for toilet flushing. Excess light GW is discharged (without treatment) to the sewer system as overflow.

(3) RWC+IR – Same as type 2 but excess light GW (overflow) is reused for irrigation after treatment.

** The scenarios examined:

Scenario 1 is the current situation in Israel, where no GWR is implemented. Scenario 2 and scenario 3 are for comparison of extreme implementation scenarios. Scenario 5 represents the typical penetration ratios of GWR to be expected in 20 years time. The last scenario (6) represents the expected penetration ratio if strong promotion for GWR will be practiced.

The comparison of the different scenarios was performed using the following criteria:

- Flow in the pipes
- Concentrations and loads of selected pollutants, in the outlet of the sewer system (inlet of the WWTP). The selected pollutants include COD, BOD, TSS, $NH_4^+$, $PO_4^{3-}$ and TKN.

*Quantification of the model*

<u>The chosen neighborhood</u>

As aforesaid, the number of residents in the neighborhood was estimated to be 15,000, and an "as made" plan for the neighborhoods sewer system was used as a basis of the modeling.

For the simplification of the hydrologic modeling process, further assumptions had to be made. These assumptions are indicated and explained below.

Assumptions:

- Length and diameter of the pipes:

  The diameters of the pipes in the neighborhood sewer system run between 200mm (upstream) to 355mm (downstream). For the simplification of the hydrologic model, an average diameter of 315mm was chosen.

  The area contributing WW in the neighborhood was divided into three sub catchments. It was assumed that all three sub catchments are within the same distance from the outlet of the sewer network (333m). Since, all three pipes have the same parameters; a shift in time of the hydrographs and pollutantgraphs is the only influence of the pipe. It should be mentioned that in the hydrodynamic model, such simplifications were not made, and more specific data of the length and diameter of the pipes will be used.

- A flow attenuation block was added, in order to simulate the attenuation of flows in the main collecting pipes between the sub catchments. The attenuation of the flows was modeled by Nash cascades. Where n, the integer number of conceptual reservoirs, was set to be 3 (a conventional value), and k, the storage constant was set to be 50 minutes. Although a conventional value for k is 5 minutes, this would not have a sufficient influence on attenuation, and hence a higher, but still reasonable, value was chosen. In the hydrodynamic model, an attenuation block and an assumption of the storage constant value will not be necessary, since the attenuation will be calculated by the model itself.

  Nash cascades models flow in sub catchments by conceptually routing it through a series of linear reservoirs, thereby achieving attenuation of the wave (Figure 8).

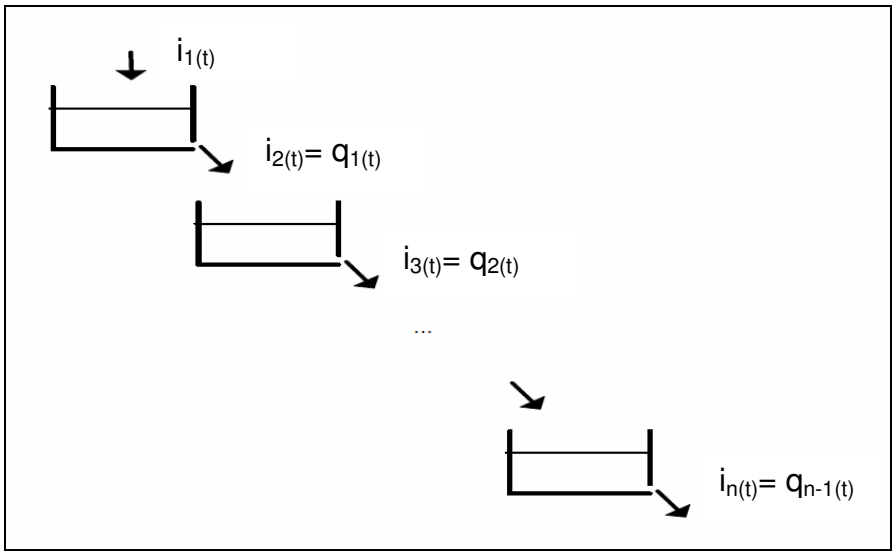


**Figure 8 - The concept of a reservoir cascade**

Each of the n identical reservoirs in series can be described by the storage equation (describing change in stored water volume) (equation 13) and the continuity equation (relating outflow to storage) (equation 14)

13) $$\frac{dS(t)}{dt} = I(t) - Q(t)$$

14) $$S(t) = k \times Q(t)$$

Where:

I(t): inflow at time t [$m^3/s$]

Q(t): outflow at time t [$m^3/s$]

S(t): storage at time t [$m^3$]

k: storage constant [s]

Different equations may be used in order to determine the values of the parameters n (integer number of conceptual reservoirs) and k (storage constant) (equations (15) to (17)).

15) $$t_p = (n-1) \times k$$

Where:

tp: time to maximum flow

16) $$h(t = t_p) = \frac{1}{k \times (n-1)!}\left(\frac{n-1}{e}\right)^{n-1}$$

Where:

h(t=tp): maximum ordinate (maximum flow for unit input)

17) $$t_L = n \times k$$

Where:

$t_L$= flow time to center of gravity of the input

As mentioned above the hydrologic models set were carried out for the six scenarios (Table 5). Each model simulated a different scenario, and represents the entire neighborhood. In each model the neighborhood was divided into three sub catchments and to each one the area and the population density was determined. Each sub catchment represents a different type of GWR (1 or 2 or 3 as explained above). The number of residents in each sub catchment was determined according to the percentages of usage for the corresponding type of GWR, i.e. the number of residents in a sub catchment was calculated by multiplication of the total number of residents in the neighborhood by the corresponding GWR type usage percentages. For example in scenario 6 40% of the houses don't reuse, 30% reuse for toilet flushing and 30% reuse for toilet flushing and for irrigation. Hence, the population contributing WW from sub catchment number one will be 40% of the total number of residents in the neighborhood, in sub catchment 2 30% and in sub catchment 3 also 30%.

Figure 9 depicts the schematic representation of the sewer system as it was modeled in the hydrologic model (the scenario demonstrated is scenario number 6). The inlet of each sub catchment was the appropriate (i.e. from type 1 or 2 or 3 of GWR) diurnal hydrographs and pollutographs of the wastewater discharged to the sewer, from one person. These hydrographs and pollutographs were taken from the results obtained in the previous chapter. Later these graphs were multiplied by the number of residents found in the corresponding sub catchment. For example in Figure 9 the inlet of the first sub catchment, called "no_ru", was the diurnal hydrographs and pollutographs of the wastewater released to the sewer, from one person, when there is no GWR. The inlet of the second sub catchment, called "ru_wc", was the diurnal hydrographs pollutographs of

the wastewater released to the sewer, from one person, when GW is reused for toilet flushing. And the inlet of the third sub catchment, called "ru_wc_irr", was the diurnal hydrographs and pollutographs of the wastewater released to the sewer, from one person, when GW is reused for toilet flushing and for irrigation. The results presented later are, as will be explained, from the outlet of the neighborhood, i.e. from the block "out_40_30_30".

It should be mentioned that the inlet for the “Rain_specific” block, was zero, since a separate sewer was modeled.

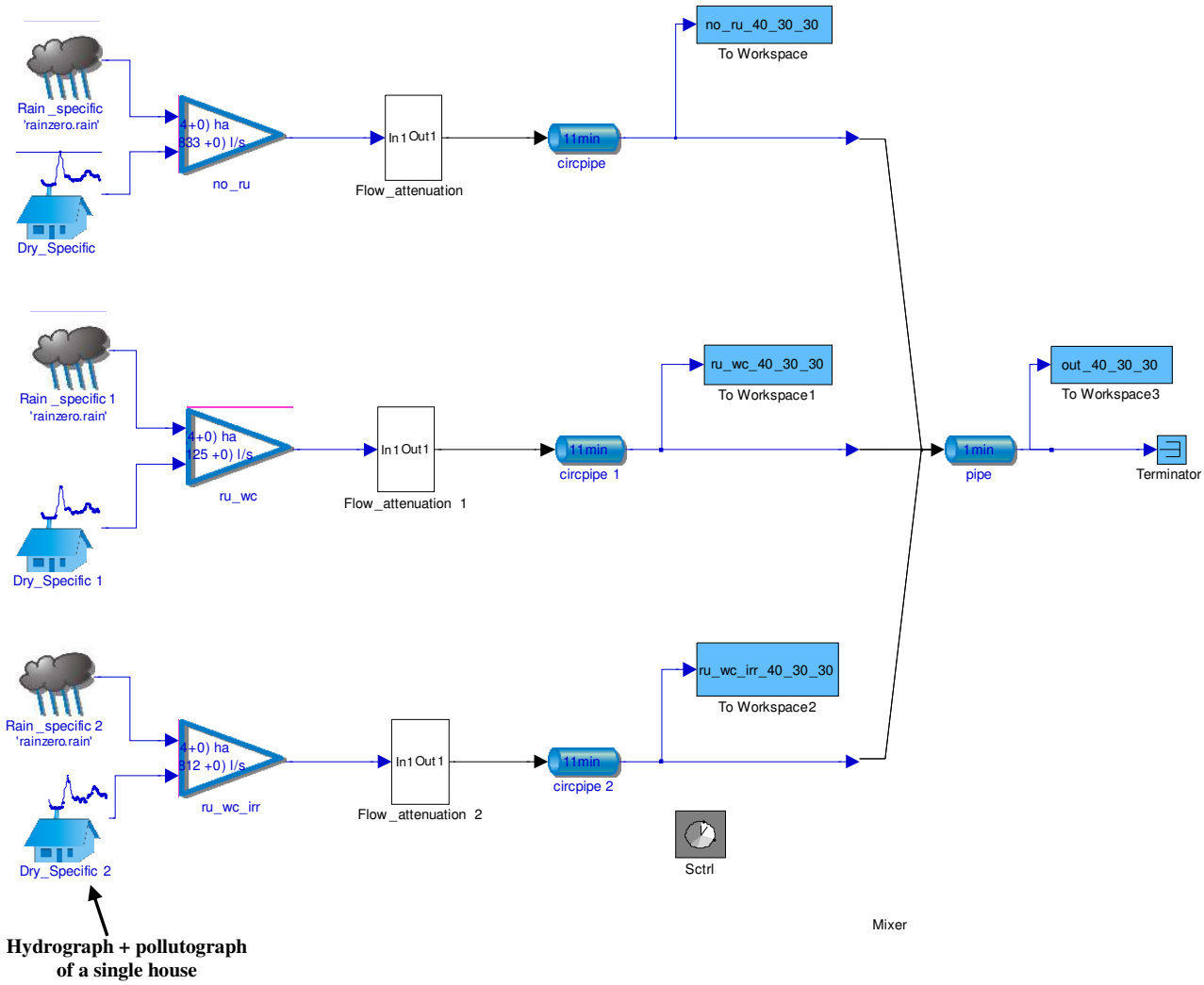


**Figure 9 - Schematic description of the hydrologic model design, for the neighborhood, as it is represented in SIMBA (scenario number 6)**

no_ru- no GWR, ru_wc- light GWR for toilet flushing, ru_wc_irr-reuse for toilet flushing and for irrigation.

## RESULTS AND DISCUSSION - HYDROLOGIC SIMULATIONS

All results introduced are from the outlet of the neighborhood, where all the wastewater streams are mixed and combined.

Figure 10, Figure 11 and Figure 13 depict the diurnal wastewater, flows, loads and concentrations (respectively) in the sewer system, at the outlet of the neighborhood, from scenarios 1-6. The simulation results in very similar diurnal pattern, for the flow, the concentrations and the loads, for all six scenarios.

The diurnal flow patterns (Figure 10) are characterized by two main peaks, morning peak, arriving a bit after 10:00, and an evening peak, arriving around 22:00. The morning peaks are much more significant than the evening ones. As expected, as the reuse is grater the flows reduction is higher (compared to scenario 1). Further it can be seen that the highest reduction occurs during the peak usage hours, especially during the morning peak, while during lower flows periods there is hardly any reduction. Peak flows at the highest reuse scenario (all houses reuse for toilet flushing and irrigation, #3), are still higher than the lowest flows in the scenario which represent the current situation, when there is no GWR (i.e. scenario l). This is meaningful from the point of view of the sewer system. Meaning that the minimum flows and velocities of the WW hardly change, while the maximum flows decrease. When examining the probability of blockages, based on these findings it can be assumed that the rate of blockages will not increase because of the GWR, although further investigation on this issue is required. Moreover it can be assumed that the maximum proportional depth of the pipes will decrease, and hence, in the future, it might be possible to connect additional WW contributors to the same sewer system.

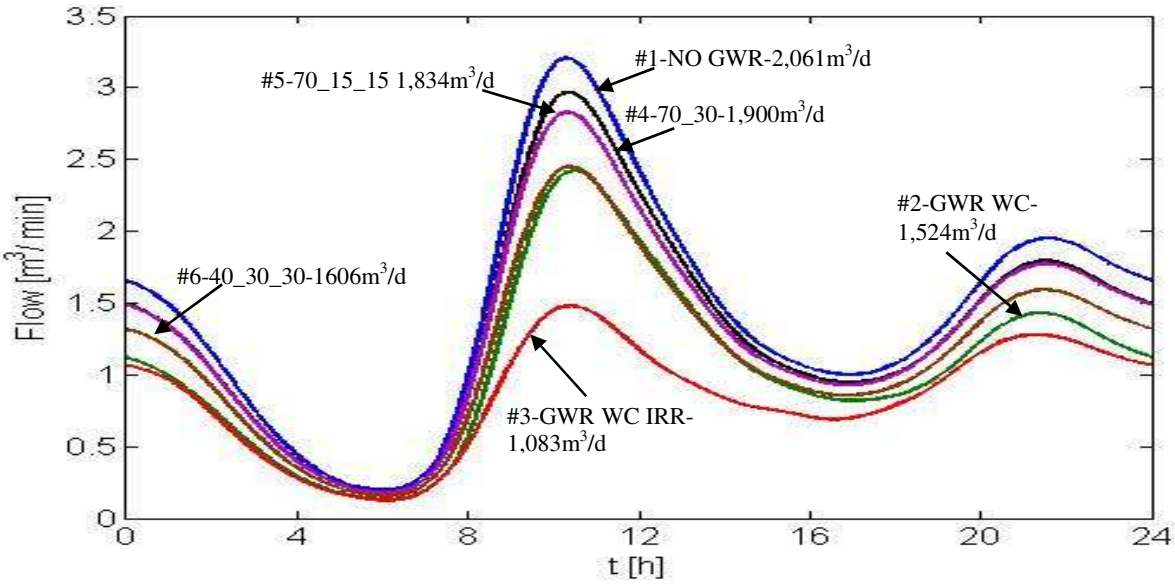


**Figure 10 - Diurnal changes in wastewater discharges in the main sewer line, at the outlet of the neighborhood, scenarios1-6**

**Table 6 - Average daily flows, in the main sewer line, at the outlet of the neighborhood, and their proportional decrease compared to the reference scenario, (scenario 1) - Scenarios 1-6**

| Scenario | #1 no_GWR | #2 GWR_WC | | #3 GWR_WC_IRR | | #4 70_30 | | #5 70_15_15 | | #6 40_30_30 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $m^3/d$ | $m^3/d$ | % | $m^3/d$ | % | $m^3/d$ | % | $m^3/d$ | % | $m^3/d$ | % |
| **Average daily flow** | 2,061 | 1,524 | 26 | 1,083 | 47 | 1,900 | 8 | 1,834 | 11 | 1,606 | 22 |

The diurnal patterns of pollutants are influenced, as was shown in the previous chapter, by the diurnal patterns of the different household wastewater discharges (WC, shower, kitchen etc.) and their components, hence different pollutants have different diurnal patterns. $NH_4^+$, $PO_4^{3-}$ and TKN are discharged to the sewer mostly from the toilet. Since during the night toilet usage dominates domestic sewage discharges, the loads of these pollutants have night peaks (Figure 11 d,e,f). The diurnal patterns of the COD, BOD and TSS loads are very similar to the flow pattern, the reason for that is that their concentrations in the different wastewater discharges from the different wastewater sources in the house are similar (Figure 11 a,b,c).

Compared with the current situation, no GWR (i.e. scenario 1), hardly any decrease in the diurnal pollutants loads can be noticed. Some decrease is noticed during peak water consumption times, particularly in the morning. The reason for that is that though the

pollutants concentrations and daily loads in the light GW (SH, BT and WB) are relatively low, during peak usage hours, especially in the morning, flows discharged from these sources are, relatively to the rest of the day, high (especially from the shower) and therefore the loads discharged to the sewer at these hours are relatively high. Hence treatment and reuse of the water discharged from the light GW sources reduces the total loads discharged to the sewer, at the time of peak usage hours, as can be seen in Figure 11. The average daily loads of the different pollutants are hardly reduced (figure 12). This is because the main pollutants contributors are the black water (i.e. WC) and dark GW (i.e. KS and WM). As the reuse is grater the reduction is higher. Since light GWR reduces the flows discharged to the sewer, but hardly changes the pollutants loads, the pollutants concentrations, at the outlet of the neighborhoods sewer system, increase (Figures 13-14, and Table 7). As the reuse is higher, a higher increase in concentrations occurs. In all the reasonable scenarios, except for the extreme situations where all houses reuse their light GW for toilet flushing and for irrigation, the increase in concentrations is relatively low, and somewhat lower than the decrease in the WW flows (Table 6 and Table 7). During morning peak water usage hours, sludge (which is a byproduct of the onsite GW treatment system) is discharged to the sewer. The sludge contains mostly COD and TSS, hence it can be seen that at the time of its release there is a peak in the corresponding pollutants concentrations.

If a GW treatment system was not implemented in the scenarios in which light GW were reused, the average concentrations of the pollutants in the sewer, were higher and hence their proportional increases, compared to the reference scenario (i.e. scenario 1) were higher as well (Table 7 and Table 8).

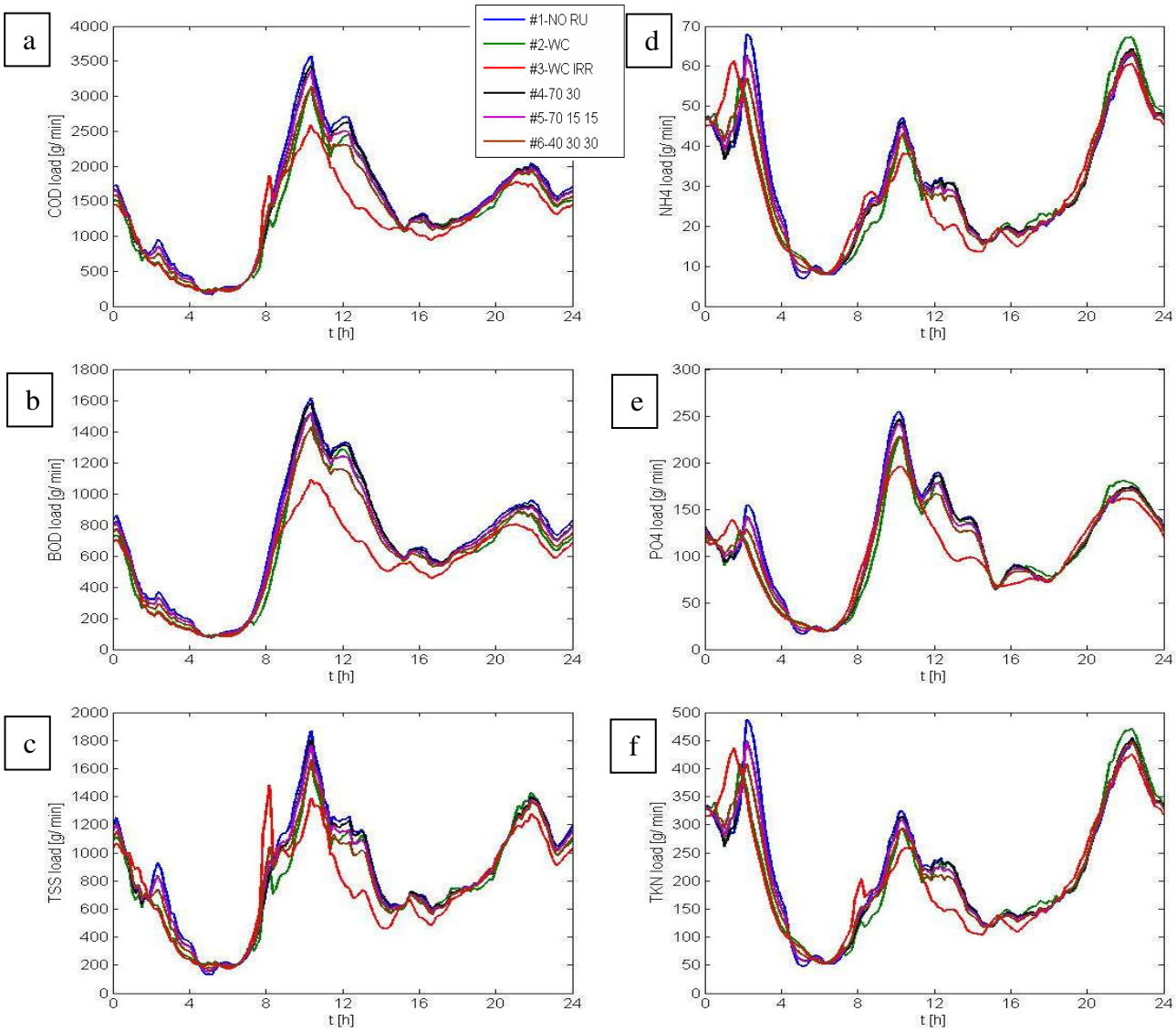


**Figure 11 - Diurnal changes in pollutants loads in the main sewer line, at the outlet of the neighborhood - Scenarios 1-6**

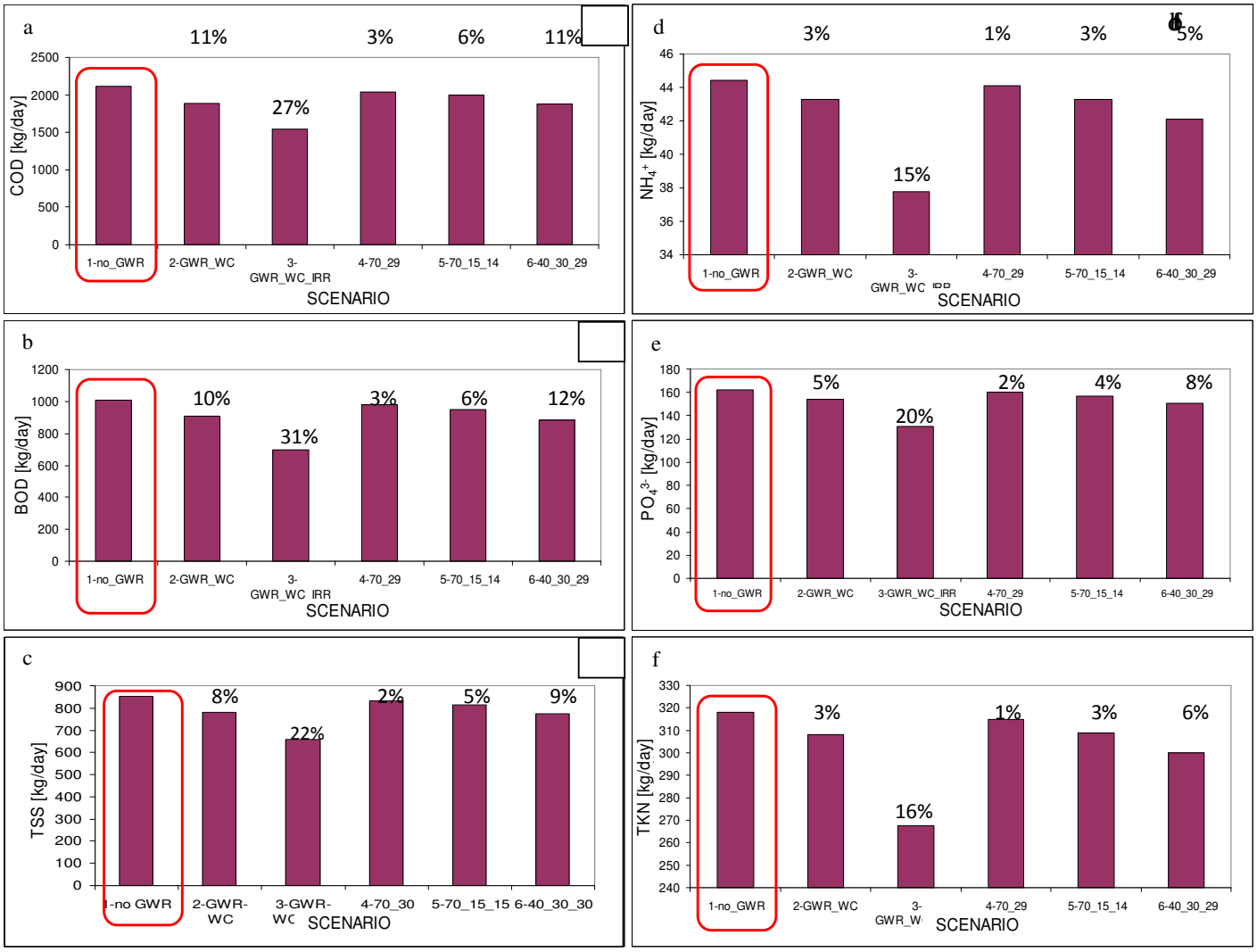


**Figure 12 - Average daily loads of different pollutants in the main sewer line, at the outlet of the neighborhood - Scenarios 1-6.**

Numbers above the columns indicate the decrease rate in loads. The scenario marked by the red rectangle, is the baseline scenario (no GWR).

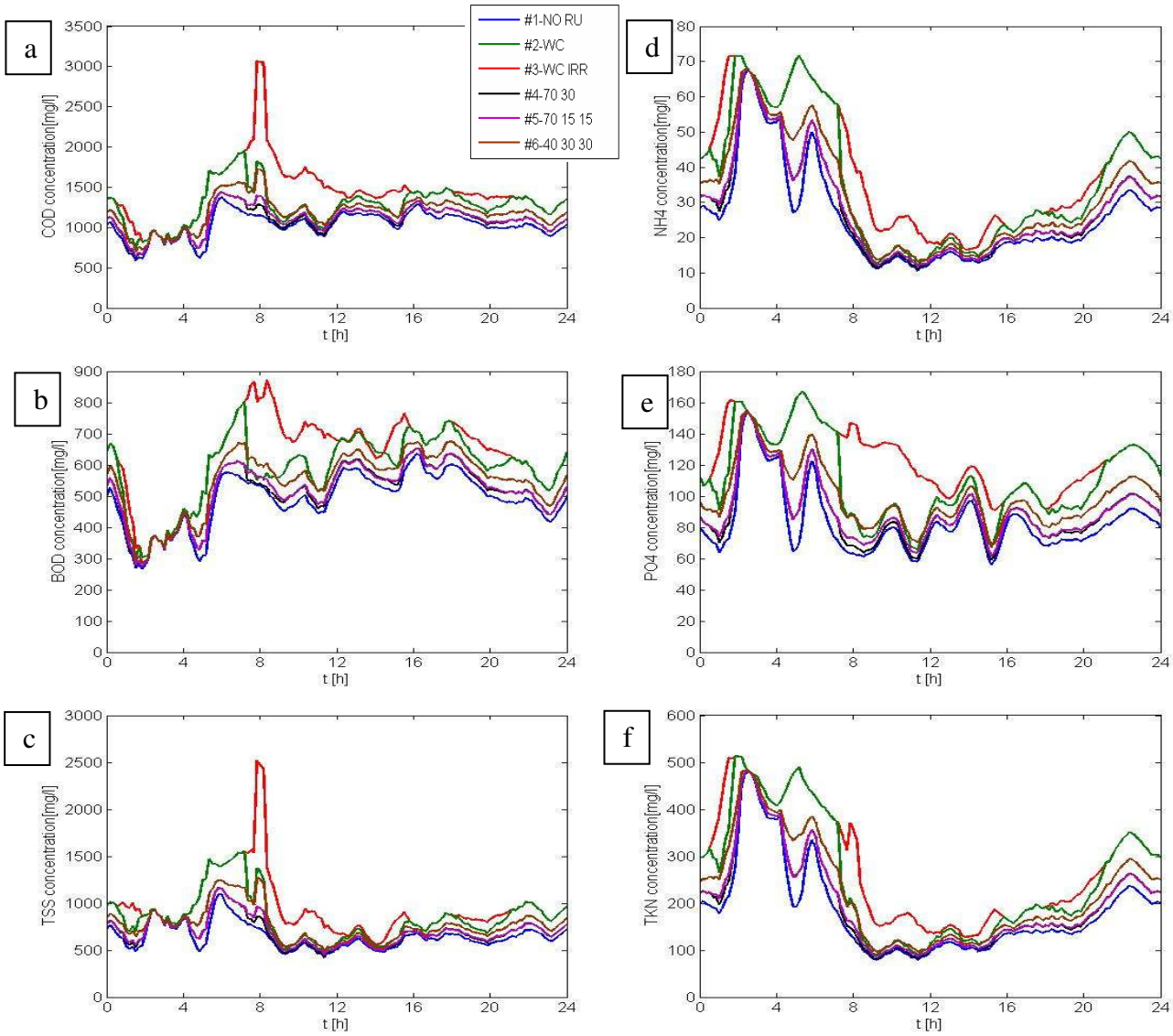


**Figure 13 - Diurnal changes in pollutants concentrations in the main sewer line, at the outlet of the neighborhood - Scenarios 1-6**

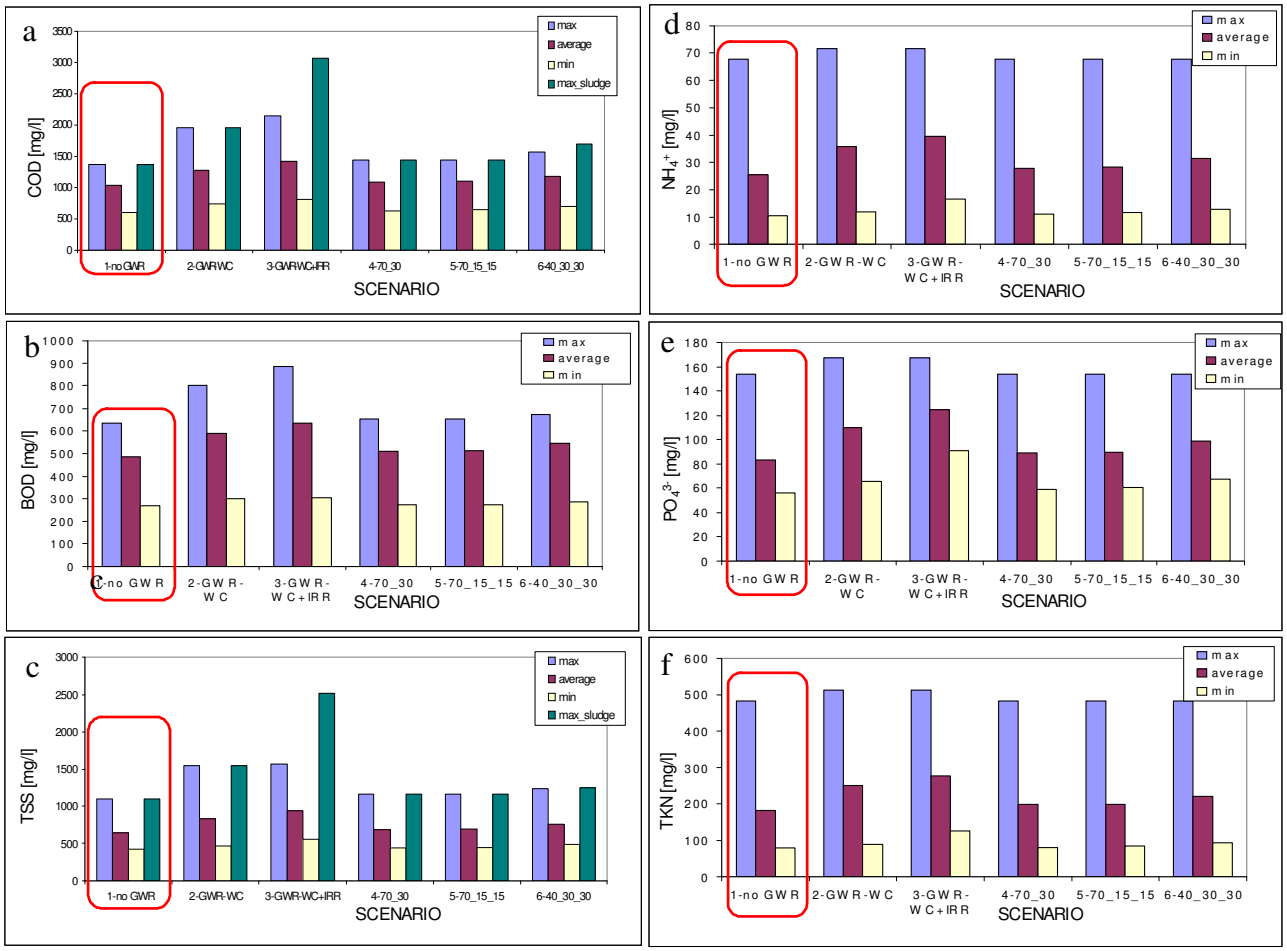


**Figure 14 - Maximum, average and minimum concentrations, of the different pollutants, at the outlet of the neighborhoods sewer, scenarios 1-6. The values marked as "max_sludge" are maximum values obtained when sludge is released; other values are received when sludge release is disregarded.**

The scenario marked by the red rectangle, is the baseline scenario (no GWR).

**Table 7 - Average concentrations of the different pollutants, in the main sewer line, at the outlet of the neighborhood's sewer, and their proportional increase compared to the reference scenario, (scenario 1) - Scenarios 1-6.**

| Scenario | #1 no_GWR | #2 GWR_WC | | #3 GWR_WC_IRR | | #4 70_30 | | #5 70_15_15 | | #6 40_30_30 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | mg/l | mg/l | % | mg/l | % | mg/l | % | mg/l | % | mg/l | % |
| **COD** | 1,028 | 1,275 | 24 | 1,438 | 40 | 1,085 | 5.5 | 1,094 | 6.4 | 1,179 | 14.7 |
| **BOD** | 488 | 587 | 20.4 | 633 | 29.7 | 511 | 4.7 | 513 | 5.1 | 545 | 11.7 |
| **TSS** | 644 | 844 | 31.1 | 955 | 48.4 | 670 | 4 | 695 | 7.9 | 762 | 18.3 |
| **$NH_4$-N** | 25.7 | 35.8 | 39.1 | 39.5 | 53.7 | 28 | 8.9 | 28.2 | 9.7 | 31.4 | 22.2 |
| **$PO_4$-P** | 83 | 110 | 32.9 | 125 | 50 | 89.3 | 7.6 | 90 | 8.4 | 99 | 19.3 |
| **TKN** | 183 | 252 | 38.1 | 278 | 52.5 | 198 | 8.2 | 200 | 9.3 | 222 | 21.3 |

**Table 8 Estimated average concentrations of different pollutants, if GW treatment was not implemented, in the main sewer line, at the outlet of the neighborhood's sewer, and their proportional increase compared to the reference scenario, (scenario 1) - Scenarios 1-6**

| Scenario | #1 no_GWR | #2 GWR_WC | | #3 GWR_WC_IRR | | #4 70_30 | | #5 70_15_15 | | #6 40_30_30 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | mg/l | mg/l | % | mg/l | % | mg/l | % | mg/l | % | mg/l | % |
| **COD** | 1,028 | 1,390 | 35.2 | 1956 | 90.3 | 1115 | 8.5 | 1,155 | 12.4 | 1,319 | 28.3 |
| **BOD** | 488 | 659 | 35.2 | 928 | 90.3 | 529 | 8.5 | 548 | 12.4 | 626 | 28.3 |
| **TSS** | 644 | 870 | 35.2 | 1225 | 90.3 | 698 | 8.5 | 723 | 12.4 | 826 | 28.3 |
| **$NH_4$-N** | 25.7 | 34.8 | 35.2 | 48.9 | 90.3 | 27.9 | 8.5 | 28.9 | 12.4 | 33 | 28.3 |
| **$PO_4$-P** | 83 | 112 | 35.2 | 158 | 90.3 | 89.3 | 8.5 | 93.3 | 12.4 | 107 | 28.3 |
| **TKN** | 183 | 247 | 35.2 | 347 | 90.3 | 198 | 8.5 | 205 | 12.4 | 234 | 28.3 |

Figure 15 depicts the maximum concentrations of the different pollutants examined and the time of their appearance, in all six scenarios examined. It can be seen that like in the current situation, when there is no GWR (i.e. scenario 1), the maximum concentrations appear in the times of low water consumptions, i.e. low flows of WW discharged to the sewer. It can be seen that there is hardly any difference in the maximum concentrations of TKN, $PO_4^{3-}$ and $NH_4^+$ (compared to scenario 1). The reason for that is that, as mentioned above, the main source of these pollutants is the toilet. Because the loads discharged to the sewer from the toilets do not change, and because the maximum concentrations arrive during low flows, when, as mentioned above, there is hardly any reduction in the flows, hence the maximum concentrations at these hours, hardy change.

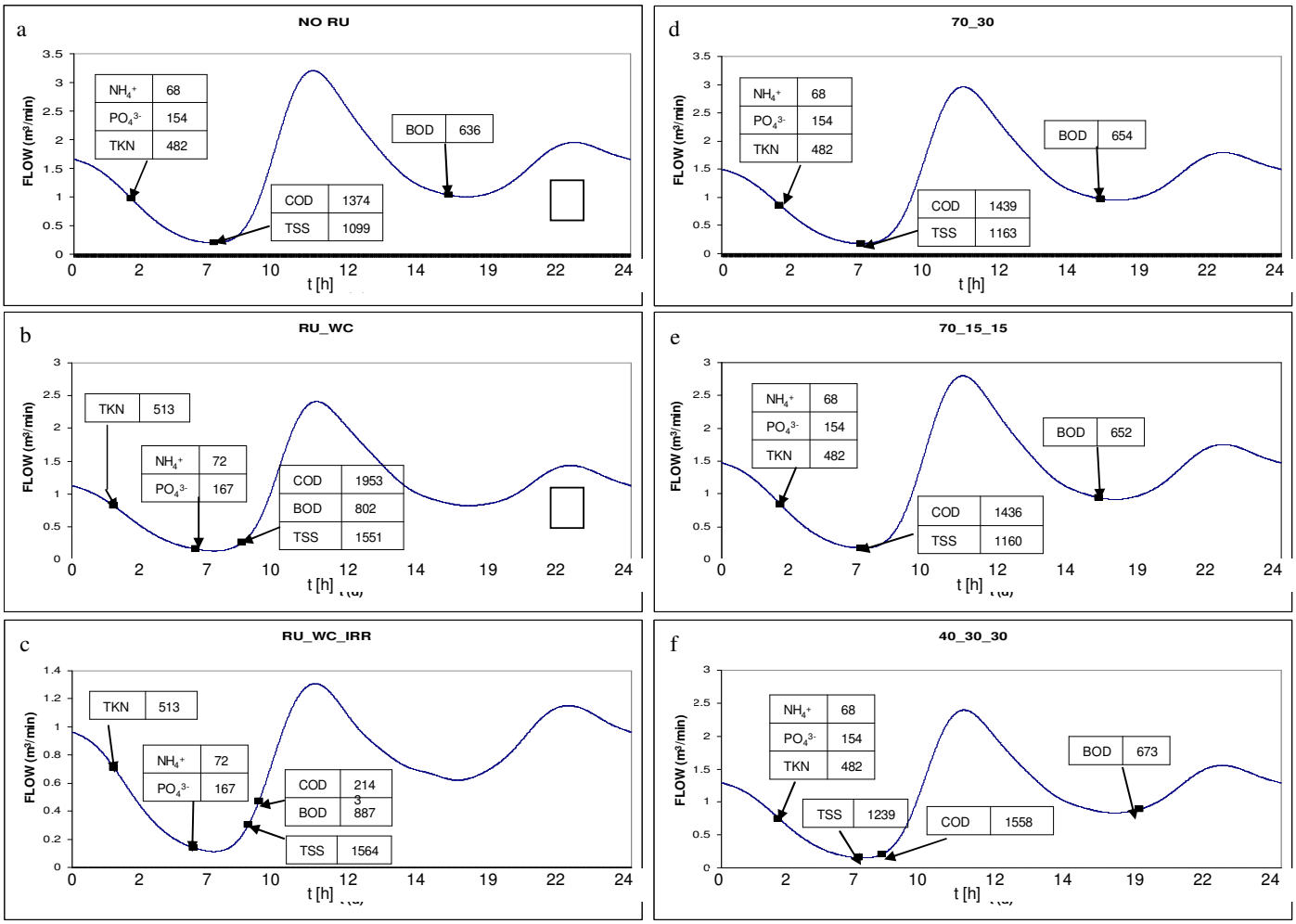


**Figure 15 - Maximum concentrations, of pollutants, and the time of their appearance - Scenarios 1-6. The values are related to the situation when sludge discharged is disregarded.**

## CONCLUSIONS – EFFECTS ON SEWER SYSTEMS

In general simulation proves to be very useful for the analysis of the potential of greywater systems.

The hydrologic simulation results in very similar diurnal (flows, loads and concentrations) patterns for all six scenarios. The diurnal flow patterns are characterized by two main peaks, a significant morning peak and a lower evening peak.

As the reuse proportion is greater the flows are more reduced (compared to scenario 1). Furthermore, the highest reduction occurs during peak usage hours, and during the

lowest flows times hardly any reduction in flows was observed. Moreover, peak flows at the highest reuse scenario (scenario 3), are higher than the lowest flows in the scenario which represent the current situation, no GWR (i.e. scenario l). This fact testifies that it can be assumed that the chances of excess blockages rate to occur are minimal.

Different pollutants have different diurnal patterns. Some decrease in the diurnal pollutants loads is noticed during the peak water consumption times, particularly during the morning. As reuse is grater this reduction is higher, resulting in an increase of, the pollutants concentrations. Nevertheless the increase in concentrations was usually lower than the decrease in flows, this is due to the removal of the pollutants in the light GW during the treatment process.
During the morning peak water usage sludge from the GW treatment system is released to the sewer and at that time there is a peak in the COD and TSS concentrations.

This chapter presented the results of the primary study, and hence gave a primary picture of the trends expected to occur in the urban sewer system as a result of onsite GWR. The next chapter will describe the hydrodynamic models that were set, in order to describe better and in more detail the trends in the sewer system as a result of onsite light GWR.

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