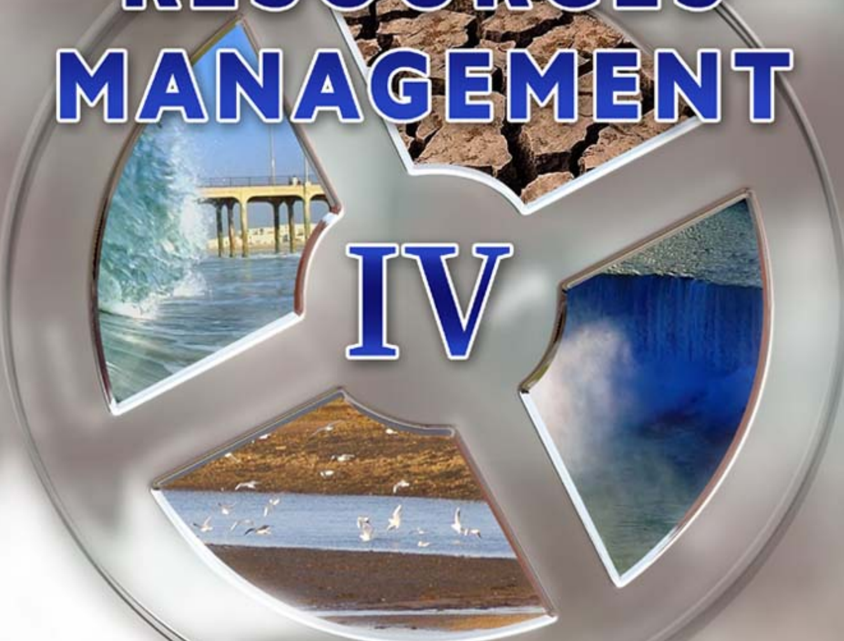
WATER RESOURCES MANAGEMENT IV



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Water Resources Management IV

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Fourth International Conference
On Water Resources Management

WATER RESOURCES MANAGEMENT IV

Conference Chairmen

C.A. Brebbia
Wessex Institute of Technology, UK

A. Kungolos
University of Thessaly, Greece

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EDITORS

C.A. Brebbia
Wessex Institute of Technology, UK

A. Kungolos
University of Thessaly, Greece

WIT*PRESS* Southampton, Boston

Editors:

C.A. Brebbia
Wessex Institute of Technology, UK

A. Kungolos
University of Thessaly, Greece

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PREFACE

The present volume contains the papers presented at the 4th International Conference on Sustainable Water Resources Management, held in Kos, Greece in May 2007. Previous conferences of the series were held in 2001 in Halkidiki, Greece, in 2003 in Las Palmas, Gran Canaria, and in 2005 in Algarve, Portugal. The fourth conference was organized by Wessex Institute of Technology, UK and was sponsored by WIT Transactions on Ecology and the Environment, the ASCE UK International Group, and the Prince Sultan Research Center for Environment, Water and Desert of Saudi Arabia.

Nowadays, water consists one of the most important natural resources in the world. Therefore, population growth and higher living standards will cause ever increasing demands for good water quality in the future, exerting an extreme pressure in water resources. Water is essential for supplying domestic, municipal, industrial, and agriculture needs. Furthermore, while growing populations and increasing water requirements are a certainty, it is not known how climates will change and at what extent they will be affected by man's activities. Climate changes (both natural and anthropogenic) are essentially unpredictable, As a direct result water resources management should be flexible and ready to cope with changes in availability and demands for water. Integrated water management on a local scale is thus imperative and all pertinent factors should be considered in the decision-making process. This integrated approach evaluates supply management demand management, water quality management, recycling and reuse of water, water economics, public involvement, public health among many other parameters.

Therefore, effective strategies for integrated water resources management both on local and national scale should be implemented in every country, taking in mind the aforementioned factors. This conference provides a common forum for all researchers specialising in the range of subjects included in sustainable water resources management.

The scientific topics presented at Sustainable Water Resources Management 2007 conference included:

- Water management and planning
- Hydrological modelling
- Water quality
- Groundwater flow problems and remediation

- Waste water management
- Waste water treatment and management
- Water markets and policies
- Pollution control
- Irrigation problems
- Urban water management
- Decision support systems
- Coastal and estuarial problems

The editors would like to thank all the authors for their contribution, as well as the members of the International Scientific Advisory Committee for their invaluable help in reviewing the papers.

The Editors,
Kos, Greece
2007

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Section 1
Water management and planning

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Incorporating CO_2 net flux in multipurpose reservoir water allocation optimization

J. Lopes de Almeida
Department of Civil Engineering, Coimbra University, Portugal

Abstract

Multipurpose hydropower plants play an important role in water resources management throughout the world. In many cases the water stored in the reservoir dam can be used for agricultural irrigation or for hydroelectricity production. From a planning point of view this conflicting, mutually exclusive, water use can be remedied by judicious water allocation. This can be computed by a decision model that maximizes global return from both electricity and agricultural production, as we showed in previous papers. However, the increasing importance of environmental constraints, especially CO_2 emission targets, demands new approaches in order to incorporate these aspects in the decision model. This paper describes a mathematical model that computes optimum water allocation taking into account the returns from hydroelectricity and agricultural production and also the corresponding CO_2 net fluxes, in order to achieve a sustainable multipurpose hydropower management. After formulation the problem is solved using nonlinear programming.

Keywords: multipurpose hydropower reservoir management optimization, irrigation, CO_2 net flux, nonlinear programming.

1 Introduction

Reservoir dams are hydraulic structures used pretty well all over the world. Their multipurpose character, combined with the natural scarcity of water resources, often leads to complex water management problems. This problem can arise when multipurpose reservoirs are committed to the two main tasks of agricultural irrigation, by diverting upstream water, and electricity production. From a sustainable planning point of view, water sharing should be established, taking into account both the revenue from the production activities and the

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environmental aspects. After the Kyoto Protocol [1], several signatory nations embarked on an extraordinary effort to reduce their CO_2 emissions to slow down global warming. The Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC), released in Paris on 5th February 2007, pointed out that carbon dioxide is the most important anthropogenic greenhouse gas and that the global atmospheric concentration of carbon dioxide has increased 35% since the pre-industrial period.

It is known that hydroelectric power plants can help reduce CO_2 emissions by replacing fossil fuel electricity production. The CO_2 emitted during construction is a small fraction of the CO_2 savings during the lifetime of the hydroelectric power plant. This environmental importance of hydropower was recognized at the World Water Forum hosted by the World Water Council, 16-23 March 2003, in Japan. The Forum culminated in the ratification of a formal declaration, which includes a specific reference to hydropower: "We recognize the role of hydropower as one of the renewable and clean energy sources, and that its potential should be realized in an environmentally sustainable and socially equitable manner." In the Portuguese electricity system, D-L No. 33-A/2005 of 16th February states that for each kWh of independent hydroelectricity production 370 g of CO_2 is avoided.

The irrigation of farmland seems to be another way of reducing atmospheric CO_2, if suitable agricultural management practices are adopted. Extensive research is presently being done to evaluate the potential of sequestering carbon by increasing soil organic carbon (SOC) using appropriate agricultural management practices [2–4]. West and Marland [2] analysed the full carbon cycle in corn crops, computing the CO_2 emissions associated with agricultural activities, which included: tillage, seed production and application, planting, fertilizer production and application, herbicide production and application, and harvesting. The net carbon flux (NCF) was evaluated considering the carbon emitted into the atmosphere as a positive flux, while carbon sequestered from the atmosphere into the soil is represented as a negative flux. Several fertilizer application rates were considered as well as two tillage practices: conventional till and no-till. According to these authors [2] the conventional till continuous corn crop in Kentucky is a net contributor to the atmospheric CO_2 pool (positive NCF). However if conventional till is replaced by no-till (when only a narrow band of earth is disturbed where the seed is to be planted and fertilized) the NCF can become negative if adequate fertilization is used. If we disregard CO_2 emissions associated with farm construction (farm infrastructures last for decades and are very small compared to the extensive corn fields) we can compute the following indicative values of CO_2 sequestration in topsoil per kg of corn yield from [2]:

- CO_2 net flux to the atmosphere for conventional till with conventional fertilization rate : + 146 g of CO_2 per kg of corn yield;
- CO_2 net flux to the atmosphere for no-till with increased fertilization rate: - 53 g of CO_2 per kg of corn yield. This carbon sequestration can be expected to last from 20 to 50 year according to [3] and [5].

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We would like to stress that West and Marland [2] focused exclusively on CO_2 emissions. Yet other greenhouse gases emissions like N_2O and CH_4 may have a considerable impact on global warming [4].

The production of a given type of plant depends on many different factors, like the soil characteristics, the climate, and particularly on the availability of water during the vegetative life cycle.

In order to reproduce the agricultural production of a corn crop analytically, Cunha *et al.* [6] developed an agricultural production function based on models taken from, Doorenbos and Kassam [7],

$$\left(1 - \frac{Ya_i}{Ym_i}\right) = Ky_i \left(1 - \frac{ETa_i}{ETm_i}\right) \tag{1}$$

Ya_i=actual production in period i; Ym_i=maximal production (when no factor limits production) in period i; Ky_i=yield response coefficient in period i; ETa_i=actual evapotranspiration in period i; ETm_i=maximal evapotranspiration in period i (if there is not an irrigation deficit), and Bowen and Young [8],

$$\frac{Ya}{Ym} = \prod_{i=1}^{N} \left(\frac{Ya_i}{Ym_i}\right) \tag{2}$$

Ya=actual production ; Ym=maximal production (when no factor limits production). This agricultural production function was applied to a corn crop in Turkey.

If we combine the agricultural production function with the indicative values of CO_2 net flux per kg of corn yield, we will get a simplified model for carbon dioxide sequestration estimation. It should be pointed out that this model derived from different data obviously cannot reproduce a specific situation. However, it analytically reproduces a simplified mechanism linking irrigation policy to CO_2 sequestration, allowing the mathematical development and computation of a decision model that can later be adapted to specific data.

In order to achieve sustainable multipurpose hydropower management we should try to maximize the monetary return and minimise CO_2 emissions. This is typically a multiobjective problem. According to Revelle and McGarity [9], these problems can be solved by two approaches:

- A multiobjective approach. In this case we can maximize the tangible monetary return from hydroelectricity and agricultural production and at the same time minimise the intangible monetary value of CO_2 emissions.
- A single objective approach. If it is possible to reduce all tangible and apparently intangible aspects to monetary values (benefits and costs), we can transform a multiobjective problem in to a single objective problem that maximizes overall net benefit function. "This is the basic logic behind benefit-cost analysis, which has been the dominant analytical tool for civil and environmental problems for 60 years" [9: 515].

At the moment CO_2 emissions tend to have a very precise monetary value. The European Union Emission Trading Scheme (EU ETS), the largest multi-national greenhouse gas emissions trading scheme in the world, came into operation on 1 January 2005, although a forward market has existed since 2003.

Other countries like Canada and Japan will establish their own internal markets in 2008 and may well link up with the EU ETS. Even in the countries like the USA that have not ratified the Kyoto Protocol, voluntary organizations are establishing CO_2 credits markets.

Since a tangible monetary value can be attributed to the CO_2 emitted or avoided, the problem can be formulated as a single objective optimization decision problem.

2 Formulation of the problem

Figure 1 gives a schematic layout of the multipurpose reservoir.

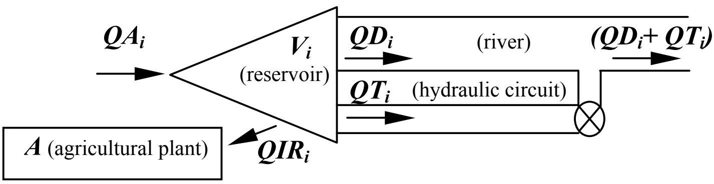

Figure 1: Schematic layout of inflows and outflows.

The problem can be stated as follows:

For each of the *i=1 to 12* fortnights of the 6 month crop period: how much water shall be allocated to agriculture production (QIR_i), to hydroelectric production (QT_i) and released downstream (QD_i) in order to maximize global return, taking in to account the CO_2 net flux and satisfying the problem constraints.
Inflows are represented by QA_i, stored reservoir volume by V_i, agricultural area by A, and the downstream required outflows are represented by $(QD_i + QT_i)$.

The objective function is:

$$\max R = \prod_{i=1}^{N} \frac{Ya_i}{Ym_i} Ym \cdot A \cdot (Py + CAF \cdot P_{CDA}) + \sum_{i=1}^{N} \eta_i \cdot \gamma \cdot QT_i \cdot H_i \cdot (Pe_i + CHF \cdot P_{CDH}) \tag{3}$$

R=global remuneration; N=number of time steps (ex.: fortnights); i=integer that represents the time step period; Py=unit price of agricultural production; CAF= CO_2 net flux per unit of corn yield; P_{CDA}=unit price of CO_2 in agriculture; η_i=overall efficiency of the hydropower plant during period i; γ=constant that depends on the water density; QT_i=volume of water to be used by the turbines during period i; H_i=Gross head during period i; Pe_i=tariff price of the hydroelectricity production during period i; CHF= CO_2 net flux per kWh; P_{CDH}=unit price of CO_2 in hydroelectric production.

The constraints of the problem can be divided into four main types:

- The constraints associated with agricultural production that give corn yield as a function of the irrigation policy. Figure 2 shows minimum and maximum corn yield associated with the least and most efficient distribution of total irrigation by the three bimonthly vegetative periods of the agricultural production function. This function, reproduced by equations (1) and (2), was adopted in the application examples.
- The constraints associated with hydroelectric production that provide the physical and technical restrictions of the hydroelectricity generation process, as well as the tariff for production remuneration. In Figure 2 we can see an arbitrated tariff based on the Portuguese tariff for independent producers. According to D-L No. 33-A/2005 of 16th February, each ton of avoided CO_2 is worth 20 €. Hydroelectricity is paid taking in to account the peak and average consumption hours, the off-peak consumption hours, as well as the average monthly hydroelectric power. Figure 2 was computed by arbitrating a management policy to transfer inflows from low consumption hours to peak and average consumption hours.
- The constraints associated with water use that provide minimum and maximum limits for: reservoir water surface levels, outflows, required energy production, and initial and final stored reservoir volumes, throughout the 12 fortnights.
- The constraints associated with the hydraulics of the problem, such as the mass balance equation in the reservoir, the elevation-storage curve at the reservoir, the elevation-flow curve at the end of the hydraulic circuit, and the stationary condition for initial and final reservoir stored water volumes.

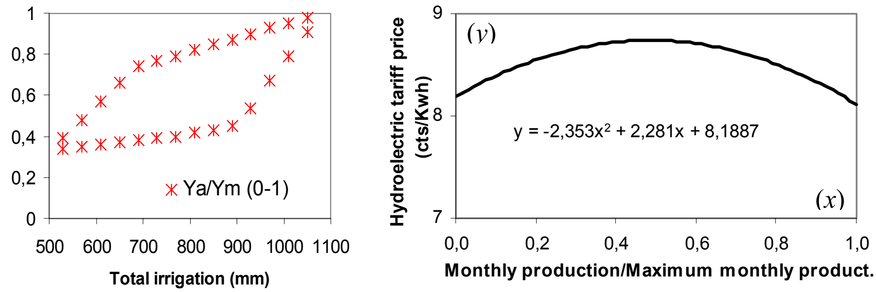

Figure 2: Production functions: Agricultural production (*Ya/Ym*) versus most efficient (upper values) and least efficient (lower values) irrigation policy, and hydroelectric tariff as a function of monthly production.

The analytical expressions that reproduce these constraints can be found in Almeida and Cunha [10].

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The nonlinear character of the objective function and some constraints indicated that nonlinear programming was the appropriate method to use. The model was solved using the GAMS/MINOS software [11, 12].

3 Examples

The computational feasibility and the dynamic behaviour of the decision model were analysed by means of several tests. Real data was mixed with artificial data to create extreme illustrative situations. The agricultural area was A=600 ha and the maximum agricultural production per hectare was Ym=6 t/ha. Three bimonthly vegetative periods were considered with $Ky_1=0.4$, $Ky_2=1.1$ and $Ky_3=0.4$. Based on information from Portuguese regional agriculture market we adopted a unit base price for corn yield of Py=182 €/t. Irrigation, with a hydraulic efficiency of 70%, occurs during the 6 month crop period from March to August. Further data associated with the agricultural production function presented in Figure 2, like for instance, soil moisture conditions, effective precipitation and evapotranspiration, can be found in Cunha *et al.* [6].

Table 1 gives the inflows to the hydropower plant reservoir. The installed capacity was $PINST$=10 MW, reservoir bottom altitude was 500m, and bottom altitude at the end of the hydraulic circuit was 234.5m. The hydroelectric tariff was given by the expression from Figure 2 where maximum price is 8.9 cts/kWh. As mentioned above, the base formulation of the problem allows the imposition of several multipurpose constraints. However, given that we are interested in analyzing the full impact of CO_2 emission in the water allocation, we will adopt illustrative examples with minimum constraints in order to provide a high degree of freedom to the decision model. The minimum and maximum surface water levels in the reservoir were set to 517 m and 536 m respectively. Outflows were not limited by minimum or maximum values. No obligatory hydroelectric power production was imposed. Further data can be found in Almeida and Cunha [13].

Table 1: Inflows to the hydropower plant reservoir.

i	1	2	3	4	5	6
$QA_i\,(m^3)$	2443890	2118140	1624010	1513950	1234500	671630
i	7	8	9	10	11	12
$QA_i\,(m^3)$	259130	76570	122200	39550	28390	30830

We considered 3 illustrative scenarios:

1) In scenario 1, water allocation optimization is computed without any considerations about the monetary evaluation of the CO_2 net flux. In this case, the application of D-L No. 33-A/2005 of 16th February leads to a mean depreciation of about 45%. The corn yield is evaluated with the base unit price of Py=182 €/t.

2) In scenario 2, the water allocation optimization is computed considering the hydroelectric tariff presented in Figure 2, and considering a conventional till corn crop. As a conventional till corn crop is a net contributor to CO_2 emissions, the unit price is depreciated to a final value of *Py*=179 €/t. To compute this price, the +146 kg of CO_2 per ton of corn yield are multiplied by the 20 € per ton of CO_2. A final depreciation value of 3 € per ton of corn yield was adopted.
3) In scenario 3, the water allocation optimization is computed considering the hydroelectric tariff presented in Figure 2, and considering a no-till corn crop. As a no-till corn crop is a non conventional technique, we estimated the monetary value of the avoided CO_2 emissions using exactly the same criterion as for the independent hydroelectric producers. This criterion basically increases the monetary value of the ton of CO_2 by a factor of 4.5. Using the -53 kg of CO_2 per ton of corn yield, a final increase of 5 € per ton of corn yield was adopted, which gives us the final price of *Py*=187 €/t.

Examples were computed with a monthly time steep. Figure 3 and 4 present results in each scenario.

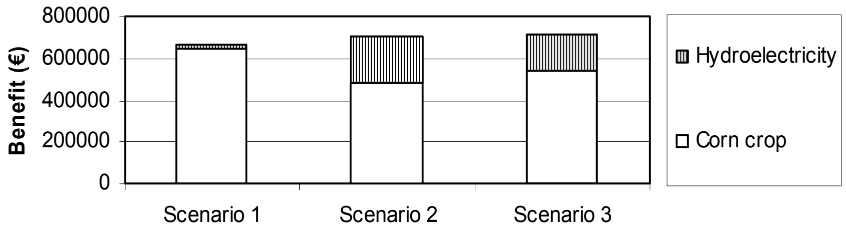

Figure 3: Benefit in each scenario.

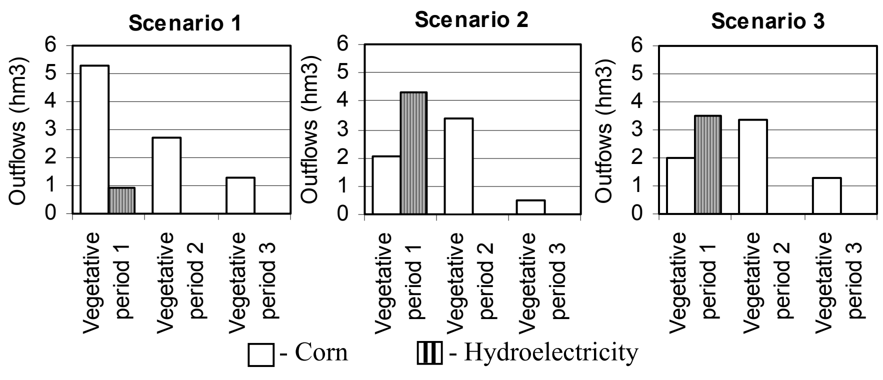

Figure 4: Water allocation in each scenario.

3.1 Scenario 1

We can observe that in scenario 1 the decision model gives priority to agricultural production because the hydroelectric tariff is low. The optimum solution implements an irrigation policy without water deficits that leads to a maximum agricultural production of Ya/Ym=1.00. Irrigation during each bimonthly vegetative period, next to the plants, is I_1=615mm, I_2=314mm, I_3=151mm (which corresponds do the upstream diversion from the reservoir of QIR_1=5.269hm^3, QIR_2=2.688hm^3 and QIR_3=1.292hm^3 respectively). The agricultural yield is 644400 €.

The optimum solution only implements hydroelectric production when surplus water is available after priority agricultural use. The outflow to the turbines is QT_1= 0.914hm^3. The hydroelectric production benefit is 25221 €.
Global benefit in scenario 1 is 669621 €.

3.2 Scenario 2

In scenario 2, the hydroelectric production tariff increases and the unit price of corn yield slightly decreases relative to the corresponding values of scenario 1. The decision model correctly identifies that hydroelectric production becomes more lucrative and responds to this modification by reallocating considerable volumes of water from agricultural use to hydroelectric use. The optimum solution implements an irrigation policy with water deficits which reduces agricultural production to Ya/Ym=0.74. The irrigation during each bimonthly vegetative period, next to the plants, is I_1=236mm, I_2=392mm, and I_3=57mm (which corresponds do the upstream diversion from the reservoir of QIR_1= 2.022hm^3, QIR_2= 3.364hm^3 and QIR_3= 0.488hm^3 respectively). Since the yield response coefficient in the second vegetative period is almost triple the yield response coefficients in the remaining vegetative periods, the decision model adopts an irrigation policy that favors this period. The agricultural yield is 482463 €.

The optimum solution implements a hydroelectric production policy that allocates the initial high natural inflows to hydropower production, rather than to ensuring a no-deficit agricultural irrigation in the first vegetative period, as occurred in scenario 1. The outflow to the turbines is QT_1= 4.290hm^3.

The hydroelectric production benefit is 221378 €.

Global benefit in scenario 2 increased to 703841 €.

3.3 Scenario 3

In scenario 3, the hydroelectric production tariff and the unit price of corn yield increase relative to the corresponding values of scenario 1. The decision model correctly identifies that hydroelectric production becomes more lucrative and responds to this modification by reallocating considerable volumes of water from agricultural use to hydroelectric use.

The optimum solution implements an irrigation policy with water deficits that reduces the agricultural production to Ya/Ym=0.80. The irrigation during each bimonthly vegetative period, next to the plants, is I_1=236mm, I_2=392mm, I_3=151mm (which corresponds do the upstream diversion from the reservoir of QIR_1=2.022hm^3, QIR_2=3.364hm^3 and QIR_3=1.292hm^3 respectively). The agricultural production in scenario 3 is higher than in scenario 2 because an increase of irrigation in the third and driest vegetative period occurs. The agricultural yield is 538560 €.

The optimum solution implements a hydroelectric production policy that allocates the initial high natural inflows to hydroelectric production, rather than to ensuring a no-deficit agricultural irrigation in the first vegetative period, as occurred in scenario 1. The outflow to the turbines is QT_1= 3,485hm^3.

The hydroelectric production benefit is 179332 €.

Global benefit in scenario 3 increased to 717892 €.

With respect to scenario 2, it is clear that the adoption of the no-till corn crop increased the competitiveness of the agricultural production relative to the hydroelectric production. Consequently, the decision model slightly rearranged the water allocation increasing agricultural water use and decreasing hydroelectric water use.

4 Conclusions

Comparing scenario 1 with scenarios 2 and 3 we can conclude that the incorporation of the monetary evaluation of the CO_2 net flux can have a considerable impact on optimum multipurpose water allocation.

From scenarios 2 and 3 we can conclude that the replacement of a conventional till by a no-till corn crop has a low impact on optimum multipurpose water allocation, when a CO_2 net flux monetary evaluation approach is adopted.

From a conceptual, mathematical and computational point of view, the approach presented above was able to incorporate the CO_2 net flux in a multipurpose reservoir water allocation optimization model for agricultural production and hydroelectric production.

In the examples presented above the decision model showed a logical response, from a dynamic point of view, to the modifications made to the data.

References

[1] United Nations, *Kyoto Protocol to the United Nations Framework Convention on Climate Change*, United Nations, 1998.

[2] West, T.O. & Marland, G., Net carbon flux from agriculture: Carbon emissions, carbon sequestration, crop yield, and land-use change. *Biogeochemistry*, **63**, pp. 73–83, 2003.

[3] West, T.O. & Post, W.M., Soil Organic Carbon Sequestration Rates by Tillage and Crop Rotation: A Global Data Analysis. *Soil Sci. Soc. Am. J.*, **66**, pp. 1930–1942, 2002.

[4] Six, J., Ogle, S.M., Breidt, F.J., Conant, R.T., Mosier A.R. & Paustian, K., The potential to mitigate global warming with no-tillage management is

only realized when practised in the long term. *Global Change Biology*, **10**, pp. 155–160, 2004.
[5] Smith, P., Powlson, D.S., Glendining, M.J. & Smith, J.U., Preliminary estimates of the potential for carbon mitigation in European soils through no-till farming. *Global Change Biol.*, **4**, pp. 679–685, 1998.
[6] Cunha, M.C.M.O., Hubert, P. & Tyteca, D., Optimal management of a groundwater system for seasonally varying agricultural production. *Water Resources Research*, **29(7)**:2415–2425, 1993.
[7] Doorenbos, J. & Kassam, A.H., Yield response to water. *FAO Irrigation and Drainage Paper*, 33, Food and Agric. Org., Roma, Italy, 1979.
[8] Bowen, R.L. & Young, R.A., Financial and economic irrigation net benefits functions for Egypt's Northern delta. *Water Resources Research*, **21**, pp. 1329-1335, 1985.
[9] Revelle, C. & McGarity, A.E. (eds). *Design and Operation of Civil and Environmental Systems*, Willey-Interscience John Wiley & Sons, Inc., 1997.
[10] Almeida J.P.P.G.L. & Cunha M.C.M.O., Optimum water allocation in a multipurpose reservoir with hourly varying hydroelectric tariff and seasonally varying agricultural production response. *Proceedings of the Joint International Conference on Computing and Decision Making in Civil and Building Engineering*, Montreal, Canada, June 14-16, 10 p., 2006.
[11] Murtagh, B. A. & Saunders, M. A., *MINOS 5.4 user's guide Rep. SOL. 83-20R*, Syst. Optm. Lab., Stanford University, Stanford, Calif., 1995.
[12] Brooke, A., Kendrick, D., Meeraus, A. & Raman, R., *Gams a user's guide*, Gams Development Corporation, Washington DC, USA, 1998.
[13] Almeida, J.P. & Cunha, M.C., Optimização da exploração de uma albufeira de fins múltiplos. *Proc. of the 4º Simpósio de Hidráulica e Recursos Hídricos dos Países de Língua Oficial Portuguesa, APRH / ABRH / AMCT*, CD, 15p., Coimbra, Portugal, 1999.

Integration of water resources modelling approaches for varying levels of decision-making

S. J. L. Mallory[1] & S. J. van Vuuren[2]
[1]*Water for Africa (Pty) Ltd, South Africa*
[2]*University of Pretoria, South Africa*

Abstract

Within government organisations there is a hierarchy of decision making ranging from broad, strategic decisions taken at top management level, through detailed planning, to the routine operational decisions. Due to the complex nature of water resources management, some form of water resources model is required to determine how much water is available, and to balance this against water requirements, taking the variability in assurance of supply to different users into consideration. In South Africa, as in many other countries, the trend has been to develop different models aimed at advising different levels of decision making. While this has proved successful, it has led to multiple models and hence multiple databases, and inevitably to inconsistencies in the conclusions reached by adopting different scales of modelling intensity and complexity. This paper describes an approach to water resources modelling which seamlessly deals with several levels of complexity, from broad-scale, strategic planning through detailed planning (or systems analysis), to the short to medium term operation of reservoirs (including hedging strategies) to deal with droughts and unexpected situations such as over-abstraction. The initial model setup commences at a broad strategic (or reconnaissance) level. As the model user progresses to higher levels of modelling intensity he is prompted to provide the data required by these higher levels. The core of this modelling system is a database of water user and monthly naturalized hydrology, rainfall and evaporation which is used throughout all levels of modelling. The fundamental concepts of reservoir behaviour analysis forms the basis of the modelling procedure, commencing with a single iteration cascading monthly time-step simulation for strategic planning purposes and progressing to a multi-iteration solution using up to 500 stochastically generated hydrological sequences, including curtailment rules for each user, to solve complex reservoir operation problems. Comprehensive testing and application of this system has not yet been undertaken, but preliminary observations are made in this paper based on a trial application in a test catchment.
Keywords: water resources modelling, reservoir simulation, database structures, reconnaissance modelling, systems analysis, reservoir operation.

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1 Introduction

The trend in water resources model development today seems to be towards the optimization of increasingly complex systems. Optimization of systems in developed countries is undoubtedly necessary as available water resources become increasingly stressed. However, in developing countries it is important to first gain a fundamental understanding of the water resources of the country or region as a whole before embarking on detailed and costly optimisation analyses. This broad or strategic level of understanding is needed by high level decision makers who are, however, unlikely to have the required expertise in water resources modelling. Simpler models are therefore required to support this strategic level of understanding.

A frequently overlooked but important aspect of modelling is that when model developers strive to solve increasingly complex problems, their models tend to become correspondingly more difficult to understand and use, often leaving the decision maker behind in the process. There is therefore a niche to be filled by simpler models, which may be less accurate and unable to deal with complex problems, but which can at least provide a broad level of understanding to the decision maker at a higher level of management within an organization.

There is, therefore, a trade off between simple, easy to use models, which do not adequately address the complexities of the actual operation of a catchment, and the more complex models capable of addressing these issues but which are not accessible to higher levels of management or decision makers.

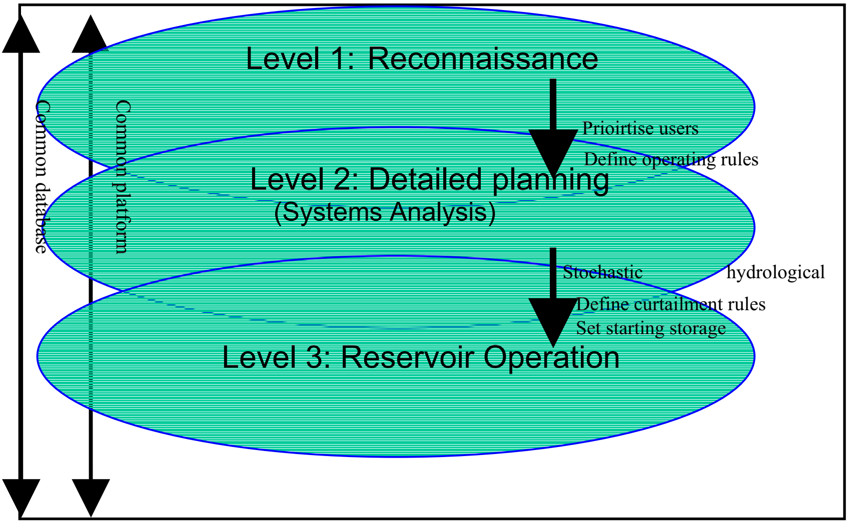

Figure 1: Hierarchical modeling: conceptual layout.

This paper discusses the integration of modelling approaches at various levels (Reconnaissance, Systems Analysis, and short-term Reservoir Operation) onto a single platform in order to allow organizations (or individuals) to progress seamlessly from simple strategic modelling through to more complex operational modelling using the same underlying data and model structure. The objective of such a modelling platform is to improve efficiency. Training and support costs could be substantially reduced through the use of such a system, since many of the supporting tools such as graphics and GIS interfaces are the same regardless of the level at which the model is being used. This hierarchical modelling concept is presented in Figure 1.

2 Principles of deploying and managing a common database

An obvious but often neglected prerequisite to achieving consistent and defendable decision–making, from the broad strategic level through to detailed operation of complex systems, is the use of a common data source for both the hydrology and the water use data within the region under consideration. Furthermore, in order to achieve the objective of reconnaissance level or strategic planning, models must be sufficiently flexible, and able to function rapidly enough to run in a workshop environment. It is argued that this can only be achieved by setting up a sub-catchment reference system in which all the required information is referenced to the sub-catchments in which they occur. It is also essential to use the same sub-catchment reference system for all levels of modelling if the results are to be comparable as increasing levels of modelling sophistication are added.

In South Africa, a standard sub-catchment reference system, referred to as quaternary catchments, was established in the early 1990's. (Midgely et al [1]). This data set, which consists of monthly streamflow and rainfall time series as well as mean monthly evaporation data are readily available in Midgely's suite of reports as well as on the SPATSIM database (Hughes [2]). Other South African databases, such as the Information Management System (IMS) (Nyland and Watson [3]) or the WSAM (Schulze and Watson [4]) either do not use the accepted catchment definition as described in Midgely et al [1] or else only contain annual mean data rather than the entire time series. In the case of the modeling system described in this paper, referred to as the Water Resources Modelling Platform (WReMP), Midgely's quaternary catchment names are used a reference to create a simple Paradox database in order to make it readily available for modelling. Simply by referencing the sub-catchment name, the natural flow and rainfall time series data are loaded into the model at run time. Spatially referenced data such as this also lends itself to easy deployment on GIS, an important feature in any water resources model.

Water use data is more difficult to manage than hydrological data since it is constantly subject to change. Water use data was collected in South Africa for the whole country in preparation for the development of the National Water Resources Strategy (NWRS) (Department of Water Affairs and Forestry [5]) and is readily available as mean annual values through the WSAM model

(Schulze and Watson [4]). However, a mechanism to manage and update this data, together with the monthly distribution of these water demands is still lacking. This is important for efficient modelling and has been dealt with in the development of this modelling platform through the concept of data mapping. Data mapping entails the creation of a sub-set of a National database for each scenario to be modelled. As a default operation, the relevant quaternary catchment data is copied from the National database into the sub-set database which can then be edited and saved by the user without over-writing (and hence corrupting) the National database.

The objective of structuring these databases was to strive for the minimum amount of data which is required by a monthly time step model. Water demands that are largely independent of rainfall events can be described adequately by the annual average demand in each sub-catchment as well as by twelve monthly factors which distribute the annual requirement realistically into twelve monthly values. The water demands have been categorized into user sectors, namely, rural, strategic, industrial, mining, urban and irrigation. Due to the limited water resources in South Africa, the priority and assurance of supply differs across user sectors. An important outcome of any water resources modeling is a strategy or operating rule which ensures that the various water use sectors will receive their water at the levels of assurance specified in the NWRS. These user sectors are therefore modeled separately so as to give a sectoral breakdown of water requirements and supply as required by the NWRS.

The water requirements of the irrigation sector, as well as those for the ecology are highly dependant on rainfall events. A separate irrigation database is used which contains crop areas and crop factors for each sub-catchment, again copied from a National database. Crop requirements are then calculated at each time step taking account of the estimated rainfall in each month. Ecological water requirements (EWR) are determined on a month by month basis using a pre-defined relationship between the natural flow and the EWR, which is provided by ecological specialists. Currently there is no complete database of the EWRs in South Africa, which is a limitation on the application of the modeling techniques described in this paper. The data that is available is stored as text files, which are accessed by the model via the catchment reference name.

3 Level 1: Reconnaissance level

The reconnaissance level modeling proposed by this paper does not read water use data from a time series file but rather minimizes user input by calculating it at every time step using the annual average requirement and the monthly distribution information provided by the water use database. See Eqn (1). Water requirements are calculated in order of priority and then checked to see how much of the requirement can actually be supplied either from storage or from the available flow in the river, before proceeding to the next user sector.

$$Requirement\left[j,i,t\right]=\left(AAR\left[j,i\right]\right)\left(DF\left[j,i,mnth\right]\right) \tag{1}$$

where: $Requirement\left[j,i,t\right]$ = Water requirement of user *j* and node *i* at time step *t*

$AAR\left[j,i\right]$ = Average annual requirement of user *j* at node *i*.

$DF\left[j,i,mnth\right]$ = Monthly distribution factor for user *j* and node *i* for month *mnth*.

Should the situation occur, where there is insufficient storage or flow to meet the requirement of user j then all users of lesser priority, i.e. *j*+1, *j*+2, etc, will not receive any water. In a well managed system, users would be restricted in times of drought with the aim of supplying all users with some water, a strategy which is modeled in more detail in the Reservoir Operation mode. Reconnaissance mode however makes the simplifying assumption of pre-defined prioritization of use at each node.

The result of the simplification presented in eqn (1) is that while there is monthly variation in the water requirement, the requirement is constant from one year to the next. The actual supply can however vary since if water is not available the supply will be less than the requirement.

Irrigation requirements are calculated every month during model run time using a typical crop requirement equation which takes into account effective rainfall and the efficiency of the irrigation method used. For brevity, these equations are not repeated here. The reader is referred to website of the Food and Agricultural Organisation [6] for more details on this aspect.

The output from a reconnaissance level simulation is a summary of all requirements and actual supply, expressed as annual average values as well as a calculation of the yield that is theoretically obtainable from each sub-catchment, also referred to as a node, within the model.

Yields are expressed in the following forms, as described in detail in Mallory [7].

- *Cumulative yield* expresses the yield that could be obtained at a particular node given all inflows (and hence taking into account all upstream abstractions and accumulating all inflows up to the node) but not taking into account abstraction from the node under consideration.
- *Incremental yield* expresses the yield that could be obtained if the calculated upstream yield was abstracted. The concept of incremental yield is useful in that it gives a good indication of the relative ability of each sub-catchment in a system to generate utilizable yield, without clouding the issue of what abstractions are already taking place within the catchment. Incremental yields can also be summed, to give an indication of the total yield available in a catchment if operated in a simple cascading fashion.
- The yield balance indicates the yield remaining at each node in the system after all abstractions at the node. This is a useful indicator of catchment

stress: if the yield balance is zero, it is very likely that not all the water demands at the node can be met.

4 Level 2: Systems analysis modelling (for detailed planning)

The reconnaissance level modelling proposed above assumes a cascading priority of water use in a catchment, which is not necessarily always the case. The sectoral water supply priorities defined in South Africa's NWRS are not related in any way to a user's location within a catchment. Hence, a high-priority user at the downstream end of a catchment poses a challenge, both in terms of the actual operation of the catchment as well as the modelling of this operation. Numerous models are available which deal with these complexities, popular methods for solving such systems numerically being linear programming and dynamic programming. South Africa's Water Resources Yield Model (WRYM) is based on the Canadian ACRES model which uses the out-of-kilter algorithm - essentially a form of linear programming (Department of Water Affairs and Forestry [8]).

The modelling system proposed in this paper for Level 2 modelling or systems analysis uses an iterative cascading solution similar to that used in the HEC5 model (Hydrological Engineering Centre [9]). It is described in more detail in Mallory and van Vuuren [10]. The shortages or deficits experienced by high priority downstream users are calculated in the first iteration. In the second iteration the shortages are released from storage or allowed to flow past upstream users by imposing curtailment rules. This method closely duplicates the actual method used by catchment managers to meet high priority downstream demands. This is an advantage over linear programming methodologies, which are based on subjective weighting factors or penalties, which are then minimized by the model, without necessarily providing any insights as to how the catchment is actually operated. Determining operating rules for a large number of users can however be time-consuming in stressed catchments using curtailment rules. Until such time as this process can be automated there is still a place in water resources modelling for linear programming models.

The advantage of the iterative cascading solution described in Mallory and van Vuuren [10] is that it is a simple extension of the proposed reconnaissance level model described above. To advance the reconnaissance level model to systems analysis mode requires the following input from the modeler:

Multiple sources of water supply: In most water supply systems, water users can obtain their water from more than one source although they may not even be aware of this. For example, an irrigation scheme typically makes use of run-of-river flow as its principal source of water, but when required, this run of river flow is supplemented by releases from upstream dams. In the Systems Analysis mode described here this would be defined as multiple source of supply, with the priority source being the run-of-river flow.

Curtailment rules: In catchments where no dams are available to supply downstream users, the catchment manager could impose water restrictions on upstream users in order to ensure that water earmarked for high-priority downstream users does actually reach them. These restrictions can either be described as a function of actual river flow, or natural flow, or the water level in a reservoir. Currently within WReMP, these curtailment rules are determined through trial and error which can be a time-consuming and inefficient exercise in large, water-stressed catchments.

User-defined time series: While the reconnaissance level model makes use of readily available information through an indexed database, the modeler may have access to better, more detailed information. By opting for time series input, the modeler can use other models to generate these time series if he so wishes or use time series provided by others with expertise in specific fields such as irrigation. The Systems Analysis mode therefore switches from “on-the-fly” calculation of demands as defined by eqn (1) to user-defined time series of all water demands. Utilities are also provided within the WReMP modelling framework to generate these time series from the database.

Return flows: The systems analysis mode allows the user to specify the node at which return flows accumulate, as well as the percentage of the flow that is returned.

Stochastic hydrological sequences: Stochastic analysis is introduced at the Systems Analysis level of modelling, allowing the modeler to source stochastic hydrological sequences from his preferred stochastic hydrological model. WReMP allows up to 500 hydrological sequences for every node in the system.

Advancing to Systems Analysis mode, the following additional output is ouput is provided:

- time series plots of releases made from dams to meet downstream demands.
- So called long-term yield curves (Basson et al [11]). These curves are useful to estimate the degree of reliability associated with the historic yield.

5 Level 3: Reservoir operation modelling

The objective of Reservoir Operation mode is to ensure that users obtain some water all of the time rather than all of their water for some of the time. In arid countries such as South Africa, which have highly seasonal and erratic rainfall, it is common practice to progressively curtail abstractions from reservoirs as the storage is progressively depleted. This type of operation, also referred to as hedging, may seem to indicate a lack of faith in the planning process which has already determined the yield that can be obtained from a dam, as well as the level of assurance of that supply. The reality is, however, that hydrology is not an exact science and there is no guarantee that the next drought will not be worse than all the previous droughts that were used as the basis of planning.

The strategic level water resources planning that has been carried out in South Africa (Department of Water Affairs and Forestry [5]) is based on a drought recurrence interval of 1 in 50 years, which implies a 2% risk of the water supply system failing in any one year. When it comes to the actual operation of a bulk water supply system, the objective is that it must *never* fail. This is achieved by curtailing the water use as the reservoir progressively empties. Therefore, while such curtailments actually reduce the water supplied from the yield determined in Systems Analysis mode, the assurance of supply (or at least partial supply) increases to 100%.

In order to model this type of reservoir operation, a curtailment rule is required for every source of supply to every user. The data structure for this is already provided in the Systems Analysis mode, since it is necessary to curtail users in line with the defined priorities. The additional information required to advance to Reserve Operation mode is as follows:

Curtailment rules: Required for each user abstracting from each reservoir. Currently these are established through trial and error until a scenario is reached in which the reservoirs do not fail.

Starting storage: This enables the modeller to take into account the actual storage in each reservoir in the system when making decision relating to curtailments.

Analysis length: This must be specified in years and should be a year or two longer than the critical period of the reservoir.

Number of stochastic sequences: As the number of hydrological sequences used in the model is increased, the probability of failure occurring in practice decreases provided that the curtailment or operating rule is adhered to. However, with increasing number of sequences, the model run time increases and hence a balance between accuracy and modelled intensity needs to be found.

The output from the Reservoir Operation mode of simulation would typically be a probabilistic plot of storage in the reservoir over time.

6 Application of hierarchical modelling to a trial catchment

The three modelling modes described in this paper have been applied individually to numerous systems in South Africa, notably during the development of the so-called Internal Strategic Perspectives (Department of Water Affairs and Forestry [12]), the Algoa Systems Analysis (in progress) and the modelling of the Letaba System (Department of Water Affairs and Forestry [13]). However, the concept of progressing seamlessly from Reconnaissance mode through to Reservoir Operation mode still needs to be thoroughly tested and documented. A trial run carried out in South Africa's Mgeni System, (which is Durban's source of water), has been carried out but space limitations of this publication prohibit detailed publication of the results of this trial run. Preliminary conclusions from this analysis are, however, as follows:

Reconnaissance level modelling tends to underestimate the yield of the system as a whole. This is not surprising since it is well documented (McMahon et al [14], Ndiritu [15]) that conjunctive use of reservoirs, and the conjunctive use of run-of-river flow and reservoirs can substantially increase system yield.

Systems Analysis over-estimates assurance of supply when compared with the results of the Reservoir Operation mode. Systems Analysis is also a poor indicator of when the yield of a system needs to be augmented.

Reservoir Operation seems to be the most realistic modelling mode and should be used more extensively for important decision-making such as the allocation of water to users in stressed systems and in determining time frames for augmentation of the supply system. This operation mode is however the most complex and time consuming.

7 Conclusions

A hierarchical water resource modelling system has been developed which enables modelers or organisations to progress easily from the simple modeling required for large catchments to make broad strategic decisions, through to the detailed modeling of reservoir operations and curtailment of water supply to users in response to drought conditions. This modeling system, referred to as the Water Resources Modelling Platform, offers efficiency through the use of a common database of hydrological and water use information which is used at all levels of complexity, as well as through the use of a common interface. The main advantage of such a system is that it allows decision makers to make reasoned decisions backed by scientifically sound analysis as to which water supply systems should be advanced to higher levels of modeling detail and intensity.

While the individual modeling modes described in this paper have been thoroughly tested and applied in practice on numerous projects, thorough testing and application of the integrated system is still required.

References

[1] Midgley, D C, Pitman, W V and Middleton, B J. Surface Water Resources of South Africa 1990. Appendices and Book of Maps. WRC Report Number 298/6.1/94. 1994.

[2] Hughes, DA. SPATSIM, an integrating framework for ecological Reserve determination and implementation. Water Research Commission Report No. TT 245/04. 2004.

[3] Nyland G F, Watson M D. Water Resources Decision Support Framework (WRDSF). 12th South African National Hydrology Symposium. 2005.

[4] Schulze C, Watson M D. Water Situation Assessment Model – A decision support system for reconnaissance level planning. Theoretical Guide compiled for the Department of Water Affairs and Forestry. 2002.2.

[5] Department of Water Affairs and Forestry, South Africa. National Water Resource Strategy, First Edition. 2004.

[6] Food and Agricultural Organisation. www.FOA.org.za.

[7] Mallory, SJL. Water resources modelling for the National Water Act. Eleventh South African National Hydrology Symposium. 2003.

[8] Department of Water Affairs and Forestry, South Africa. Water Resources Yield Model User Guide. Report no. P0000/00/001/98. 1998.

[9] Hydrological EngineeringCentre, US Army Corps of Engineers. HEC-5: Simulation of flood control and conservation systems: User Manual. 1989.
[10] Mallory S J L, van Vuuren S J. A practical approach to water resources modelling of complex catchments. In Press.
[11] Basson M S, Allen R B, Pegram G G S, van Rooyen J A. Probabilistic Management of Water Resource and Hydropower Systems. Water Resources Publications, Colorado, USA. 1994.
[12] Department of Water Affairs and Forestry, South Africa. Internal Strategic Perspective: Mvoti to Mzimkulu Water Management Area. Report no. P WMA 11/000/00/0304. 2004.
[13] Department of Water Affairs and Forestry, South Africa; River System Annual Operating Analysis (2005/2006), 2006. Report no. P WMA 02/000/00/0406. 2006.
[14] McMahon, T and Adeloye, A. Water Resources Yield. WRP Publications, Colorado, USA. 12 - 80. 2005.
[15] Ndiritu J G. Maximising water supply system yield subject to multiple reliability constraints via simulation-optimisation. Water SA 31(4) 423 – 434. 2005.

Impact of climate change on the water resources of the Auckland region of New Zealand – a case study

B. Mahmood
School of Built Environment, Unitec New Zealand,
Auckland, New Zealand

Abstract

In this century, global warming is one of the biggest environmental problems which is always puzzling human being. The focus of this study was to determine the potential impact of climate change on water resources in the Auckland Region. The results showed that the total amount of CO_2 in the air is increased by 16.6% since 1970. It is estimated to increase to 400 ppm by 2020 in the Auckland Region. The amount of CH_4 and N_2O were 1739 and 322 ppb (parts per billion), respectively, in 2006. The mean temperature has increased by 1.5°C over the last century. It is estimated that the future temperature of the Auckland Region will increase by 0.15°C by 2020. The results also showed that the average annual rainfall in the Region varied between 1241 and 1276 mm since 1925. Overall trend in sea level rise is 1.4 mm per year and it is estimated that it would increase to 1928 mm by 2020. From water resources perspective, the study showed no cause for alarm. Possible impacts that may especially affect water planning and project evaluation include changes in precipitation and runoff patterns, sea level rise, and land use and population shifts that may follow from these effects.
Keywords: global warming, climate change, water resources, Auckland.

1 Introduction

Global warming is caused by greenhouse gases which warm the atmosphere by absorbing some of the thermal radiation emitted from the Earth's surface.

Incoming solar radiation is transmitted through the atmosphere to the Earth's surface. The energy is retransmitted by the Earth's surface as thermal radiation.

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Some of the thermal radiation is absorbed by the greenhouse gases instead of being retransmitted out to space, and so there is a warming of the atmosphere. The important greenhouse gases which are directly influenced by human activities are carbon dioxide (CO_2), methane (CH_4), nitrous oxide (N_2O), chlorofluorocarbons (CFCs) and ozone. Increased amounts of greenhouse gases in the atmosphere will absorb more thermal radiation, and the earth's surface and the lower atmosphere will warm [7]. The warmed Earth emits radiation upwards, just as a hot stove or bar heater radiates energy. In the absence of any atmosphere, the upward radiation from the Earth would balance the incoming energy absorbed from the Sun, with a mean surface temperature of around -18°C. The presence of "greenhouse" gases in the atmosphere, however, changes the radiation balance. Heat radiation (infra-red) emitted by the Earth is concentrated at long wavelengths and is strongly absorbed by greenhouse gases in the atmosphere, such as water vapour, carbon dioxide and methane. As a result, the surface temperature of the globe is around 15°C on average, 33°C warmer than it would be if there was no atmosphere. This is called the natural greenhouse effect [7, 8]. In short, the greenhouse effect is a warming of the earth's surface and lower atmosphere caused by substances such as carbon dioxide and water vapour which let the sun's energy through to the ground but impede the passage of energy from the earth back into space.

1.1 Have greenhouse gas emissions caused global temperatures to rise?

Many scientific studies carried out illustrate that Earth's climate has been changing over the last century. Climate change is held responsible for a global increase in surface temperature, a decrease in snow cover and overall an increase in sea level [6]. The 2004 record of global average temperatures as compiled by the Climatic Research Unit of the University of East Anglia and the Hadley Centre of the UK Meteorological Office [3] showed that the global mean surface temperature has increased by between 0.3 and 0.6°C since the late 19th century, a change which is unlikely to be entirely natural in origin. The balance of evidence suggests a discernible human influence on global climate. Much of the 10-25 cm rise in global average sea level over the past 100 years may be related to the rise in global temperature [3]. NIWA reported that the greenhouse gases have continued to increase in the atmosphere [7]. This is due largely to human activities, mostly fossil fuel use, land-use change, and agriculture. About 64% of the warming effect of greenhouse gas increases over the last 200 years is due to carbon dioxide. The second most important greenhouse gas produced by human activities is methane which accounts for about 19% of the increased warming. (This is an important aspect of New Zealand's greenhouse gas emissions since sheep and cows produce methane). Greenhouse gas growth in global atmosphere slowed during 1991-1993 but has since picked up again. NIWA research suggests that low methane growth rates were due to a temporary reduction in agricultural burning in the tropics [7]. Warming by greenhouse gases is offset in some regions by a cooling due to small airborne particles generated by burning fuel. These are concentrated around areas of industrial activity in the Northern Hemisphere and in developing countries. The cooling effect of aerosols over the

New Zealand region is expected to be small. The global warming in New Zealand (NZ) may have an impact on aspects that are related to the changing of water resources and climate change is expected to change rainfall patterns over the country. Therefore, the focus of this study was to determine the potential impact of climate change on water resources in the Auckland Region. This paper covers the definition of natural greenhouse effect, 'has greenhouse gas emission caused global warming?' the available water resources in the Auckland Region, and how these might change in the nearer future. A brief discussion of climate change impacts in New Zealand and globally is also covered in the paper.

2 Methodology

2.1 Auckland region weather and data collection

Auckland's sunniest days occur during anticyclonic conditions in a light south-easterly flow. Due to minor causes (e.g. sea breeze convergence zones or an anticyclonic subsidence inversion) local variations in wind direction and cloud cover may occur, but fine weather predominates. Over twelve hours of sunshine in Auckland City. Auckland's heaviest rainfalls occur when there is a depression to the north or north-west with a strong north to north-east wind flow over the city, and a front embedded in the flow. The two most important parameters for the determination of climate change are temperature and rainfall. In mountainous areas of the Region, these are both modified substantially by the prevailing winds, but in a relatively flat region changes in wind direction do not induce much change in the weather unless they are associated with pronounced pressure systems or fronts. Another important factor is the proximity of the sea, which affects the smaller scale weather systems in the Auckland Region. In each of the two most common wind directions, *south-west* and *north-east*, air passes over an extensive sea path before reaching a narrow strip of land, and thus air temperatures remain comparatively low in summer and mild in winter under windy conditions. Also, converging sea breezes in summer may cause heavy showers which remain very local, but which often alleviate the dry conditions associated with anticyclones. Thus the Auckland Region is partly protected from climatic extremes because of its physical situation.

The CO_2, CH_4 and N_2O concentrations or mixing ratios (measured in ppm – by volume, which is micromole of trace gas per mole of dry air) were measured by NIWA at Baring Head, which is near Wellington Harbour. This site was chosen as a clean-air site, and for winds from the southerly sector the sampled air is representative of the south Pacific basin, unaffected by emissions from NZ or any nearby territory. The data colleted from Baring Head is representative of the air from the surrounding southern ocean so applicable to the area around NZ (87 metres above sea level). Baring head is a part of a global network of stations for determining trends in GHG – greenhouse gases concentrations. Because of the unavailability of the data for the Auckland Region, the Wellington data was used in this study. None of the above gases have a significant latitudinal gradient in the extra-tropical Southern Hemisphere, so they can be taking as applying to clean air over Auckland. It should be noted that the mixing ratios of these gases

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for the Auckland Region will be essentially identical to globally averaged mixing ratios of these gases since, because they are so long lived, there is very little regional heterogeneity (Pers. Communication with Greg Bodekar, NIWA, November 30, 2006). The CO_2 concentrations were measured continuously in situ (along with meteorological parameters), but for the other gases the discrete samples were collected approximately weekly and analysed in the laboratory. CO_2 was measured by non-dispersive infrared spectroscopy, CH_4 and N_2O by gas chromatography (using flame-ionisation and electron-capture detection, respectively). The daily historical data for carbon dioxide, methane and nitrous oxide (GHG) was available for the periods 1970-2006, 1989-2006, and 1996-2006 respectively. The annual mean gas mixing ratios were determined using the daily GHG data for the reporting years. The annual mean temperature and rainfall values were estimated using the daily temperature (1900-2005) and rainfall data (1925-2005) collected at 28 sites located in the Region. The details on these sites can be found in [2]. The annual mean seal level data was also obtained from NIWA for the period 1899 to 2000. Future estimation of GHG concentrations, temperature, rainfall and sea level rise were made using the rate at which these have increased over the past years. The Auckland Regional surface and groundwater resources use and allocation data was obtained from Auckland Regional Conference (ARC). The 30 years average temperature and rainfall data were also collected from the Albert Park weather station of Auckland city for comparison purpose.

3 Results and discussions

3.1 Concentrations of CO_2, CH_4, and N_2O Gases

The results showed that the concentrations of CO_2 increased steadily by 16.6% (i.e. from 325 ppm in 1970 to 379 ppm in 2006) at an average rate of 1.50 ppm per year. If it continues to increase at the same rate then it is estimated that the amount of CO_2 will increase to 400 ppm by 2020 in the Region (Figure 1). The concentrations of CH_4 and N_2O were 1739 and 322 ppb in 2006. From 1989 to now, the CH_4 concentrations has risen about by 4.5% (i.e. 1663.4 to 1739 ppb in 2006) at an average rate of 2.1 ppb per year. The amount of N_2O has increased on average by 3.4% at an average rate of 0.3 ppb per year, since 1997. It is estimated that the level of these two gases (i.e. CH_4 and N_2O) would increase to 1768 and 326, respectively, by 2020.

The increasing concentration of CO_2 is caused by the burning of fossil fuels (such as oil, gas and coal), increasing number of cars on the road, and the destruction of forests. These activities release large amounts of CO_2 into the atmosphere. The main natural source of CH_4 is from wetlands. A variety of other sources result directly or indirectly from human activities, for example from ruminant animals (sheep and cows), rice paddies, leakage from natural gas pipelines, and from the decay of rubbish in landfill sites. These emissions continue to increase atmospheric CH_4 concentrations. Figure 1 showed that the level of these three gases has gone up at a significant rate over the past years. If extra amounts of greenhouse gases are added to the atmosphere, such as from human activities, then they will absorb more of the infra-red radiation. The Earth's surface and the lower atmosphere will warm further until a balance of incoming and outgoing radiation is reached again (the emission of infra-red radiation increases as the temperature of the emitting body rises). This extra warming is called the enhanced greenhouse effect. As the Auckland population is increasing, the energy use from fossil fuels will increase that will continue to lead to dramatic increase in the amount of CO_2 in the atmosphere. Similarly, CH_4 and N_2O emission may also increase. It should be noted that about 55% of NZ's emissions are methane (37.5%) and nitrous oxide (17.4%), largely of agricultural origin, which makes NZ GHG profile unusual among developed countries (Ministry for the Environment, 2003).

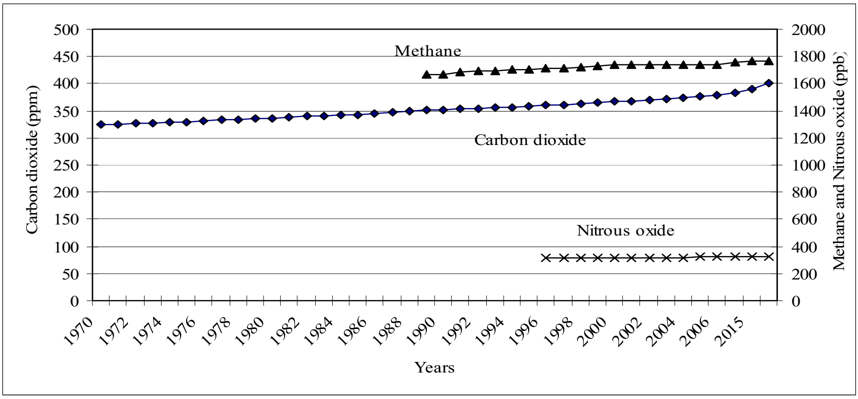

Figure 1: Carbon dioxide, methane and nitrous oxide concentrations for the reporting years. (Source: NIWA).

3.2 Temperature and rainfall

The results showed that the mean temperatures of the Region have an unstable increase in the past years. Although there were some temperature decreases but the value has increased from 14.95°C in 1900 to 16.3°C in 2000 (nearly 1.5°C increase over a century). The two temperature decreases were observed in the beginning 20 years and 1970s to 1980s of the 20th century. The highest mean temperature was 15.95°C in 1970 and the lowest mean temperature was 14.75°C in 1920. It is estimated that the future temperature of the Auckland Region will increase by 0.15°C by 2020. It should be noted that the updated 100 years (1906-2005) linear trend of global temperature showed an average increase of 0.74°C (0.56-0.92°C) [4], which is larger than corresponding trends reported by UK Meteorological office [3]. The results showed that the annual mean rainfall varied between 1235 and 1283 mm. In the last century, the average annual rainfall was over 1200 mm. The annual mean rainfall may increase up to 1300 mm (Figure 2) by 2020.

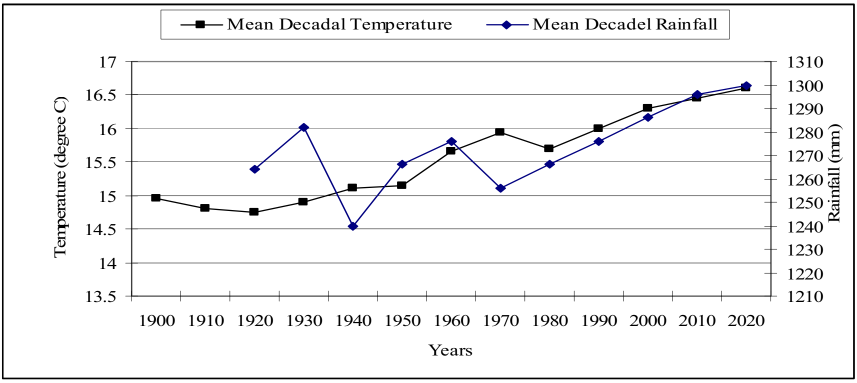

Figure 2: Auckland Region's mean decadal temperature 1900 to 2020) and rainfall (1925-2020) data. (Source: ARC).

3.3 Sea level rise

The results showed that the annual mean sea level has risen from 1710 mm in 1900 to 1900 mm in 2000. Overall annual mean sea level has risen by 0.19 m per century at Auckland port and 0.17 m per century on average across four main ports of New Zealand. The results showed that the overall trend has been 1.4 mm rise in sea level per year and it is estimated that it would increase to 1928 mm by 2020. Higher sea levels and increased storm surges could adversely impact freshwater supplies in some coastal areas of the Region. Saline water profiles in river mouths and deltas would be pushed farther inland, and coastal aquifers would face an increased threat of saltwater intrusion. The intrusion of saltwater into current freshwater supplies could jeopardize the quality of water for some domestic, industrial, and agricultural users. The sea level rise would aggravate water-supply problems in several coastal areas in the Auckland Region, including Manukau Harbour and Hauraki Gulf. It may also adversely affect the Regional groundwater levels. Changes in groundwater levels may damage drainage network and septic tank operations due to changes in groundwater pressure beyond design specifications. This may have negative public health effects, especially if groundwater wells become contaminated. Sea-level changes may also alter pressures within storm water and sewerage systems. And sea-level rise may also pose a threat to wastewater treatment facilities (e.g. oxidation ponds) located on the coast. The global average sea level rose (over the 20th Century) between 0.13 and 0.23 m, with a central value of 0.18 m per century. The total 20th century rise is estimated to be 17 cm [4]. Longer overseas records from stable ports in Europe indicate that modern sea levels began rising noticeably in the early to mid-1800s, after a 3,000-year period of relatively slow rise of 0.01 to 0.02 m per century [10]). The rise in sea level could be due to thermal expansion of the oceans and increased melting of glaciers and land ice. The Auckland sea level increased 19 cm during the past century (as mentioned earlier) which was largely due to the melting of land-based ice sheets and glaciers. The IPCC 1995 assessment report suggests that average sea level might rise another 15 to 95 cm by the year 2100, with a best guess of about 50 cm [9]. Whereas, IPCC 2007 projected globally a sea level rise between 18 and 59 cm (over the 21st century) under different model-based scenarios [4]. These projections were assessed from a hierarchy of models that encompass a simple climate model, several Earth Models of Intermediate Complexity (EMICs), and a large number of Atmosphere-Ocean Global Circulation Models (AOGCMs).

3.4 Water resources of Auckland region

The surface water of Auckland Region is composed of rivers, small streams, small lakes and wetlands. There are approximately 10,000km of streams in the Region, mostly are the small tributaries (less than two meters wide) found at the heads of catchments. This is because the catchments of the Region tend to be small and short. These streams are more sensitive to abstraction and reduced flow and so need to be closely monitored to make sure that flows are maintained at levels that do not adversely impact on their resident flora and fauna. This is particularly important during the summer when flows are at their lowest and demand for water is at its highest. Whereas, groundwater is directly abstracted for water supply and is well used in the Auckland Region. Most groundwater is abstracted from aquifers in the Franklin and Bombay basalts, Waitemata sandstone, Auckland Isthmus basalts, Pleistocene sands and Kaawa shell beds. The region also has two geothermal fields at Waiwera and Parakai. The ARC has defined those stream catchments that are under pressure from high water use as "high use stream management areas". The details of the high use streams and high use aquifers are not given here because of the length constraints of the paper, but can be found in [1]. Water (both surface and ground water) is allocated and used for a range of purposes in the Auckland Region. These are grouped into five categories; (i) irrigation: including pastures, market gardens, orchards, hot house and nurseries, (ii) community: facilities such as golf course, bowling greens, sports fields and also small community supply, (iii) industry: including quarry dewatering, industrial plants and food processing (pigs and poultry), (iv) municipal: reticulated supplies to the metropolitan area serviced by Watercare Services Ltd. and larger urban centers supplied by Franklin and Rodney District Councils, (v) other: including monitoring, emergency, stock and domestic consents, geothermal and other uses. The results showed that 136 million cubic meters (Mm^3) of water (106 Mm^3 of surface and 30 Mm^3 of groundwater) is allocated each year for all purposes (as mentioned above). The main use of the allocated water in the Region is within domestic households. It is calculated that 77.5 million cubic meters of water is used by 1.23 million people in Auckland households for uses such as drinking, cooking, washing and cleaning, and flush toilets. That equates to 173 litres being used by every person every day. This compares favorably with other cities in New Zealand with average domestic water use closer to 250 l/p/d. The Auckland population is rising at a rate of 18,000 people per year. It is estimated to increase to 1.7 million by 2020 (at a rate of 2.4% increase per annum). 92% and 5% of the allocated surface water is used for municipal and irrigation purposes, respectively. The rest is used for community, industry and other purposes. The results also showed that the amount of groundwater used by different groups varies between 38 and 72% of the allocated groundwater. The overall average water demand remains just below 54% of the total allocated groundwater and 93% of total allocated surface water (overall less than 74% use of total water allocated). This shows that the available groundwater resources are sufficient to meet the water demands in the Auckland Region. Thus it is expected that there will be no shortage of water over the next few decades. There is a broad agreement that climate change will have major impacts on water resources. Possible impacts that may especially affect water planning and project evaluation include changes in precipitation and runoff patterns, sea level rise, and land use and population shifts that may follow from these effects. Warmer temperatures will accelerate the hydrologic cycle, altering precipitation, the magnitude and timing of runoff, and the intensity and frequency of floods and droughts. Higher temperatures will also increase evapotranspiration rates and alter soil moisture and infiltration rates. It is expected that the future climate change (in terms of increase in temperature and CO_2 level) may lead to increased surface water resources of the Region, with high rainfall in the west and less in the east of the region.

3.5 General discussions

3.5.1 Potential impact of climate change in New Zealand and globally

New Zealand is likely to experience climate changes such as higher temperatures, more in the North Island than the South, (but still likely to be less than the global average); increasing sea levels that are expected to rise (globally) between 18 and 59 cm by 2100 [4], compared with an average rise of 18 cm in the 20th century); more frequent extreme weather events such as droughts (especially in the east of New Zealand) and floods; a change in rainfall patterns – higher rainfall in the west and less in the east. In the long term, if unchecked, climate change increases the risk of major and irreversible changes to Earth. For example even for relatively moderate warming, the Greenland ice sheet is expected to melt completely over the next several thousand years, which would lead to a sea-level rise of as much as 6-7 m [1]. The cost of doing nothing about climate change could be severe and the impacts on our environment, economy and society are likely to get steadily worse if greenhouse gas emissions are not reduced significantly over the coming decades. Impacts of climate change will be distributed unevenly around the world with developing nations the most vulnerable. Many leading scientists tell us that there will be both positive and negative consequences of climate change, at least in the short term (their findings are summarised in IPCC reports [1, 4]. New Zealand is heavily dependent for its economy and therefore on the climate. The severity of climate change impacts will depend on what we and the rest of the world do now to reduce greenhouse gas emissions and how we plan and prepare for the impacts of climate change. While New Zealand only accounts for 0.2% of greenhouse gas emissions globally, our climate will be affected by the total emissions of all countries not just our own. It is expected to see the flow-on effects of climate change through changing overseas market demands and competition, as well as environmental disasters such as droughts and storms becoming more severe. This could lead to increased demands on us for development aid and an increase in environmental refugees. This means it is important for us, and the rest of the world, to encourage and participate in a global agreement like the Kyoto Protocol which is aimed at reducing global greenhouse gas emissions [1].

3.5.2 New questions

The debate about climate change has moved from "is it happening?" to the much harder questions of "how many emissions are too much?", "how much is it going to cost?", "what to do about it?", and "who should pay?" On average, New Zealand is already at a concentration of 425 ppm of carbon dioxide equivalent (including the effect of other greenhouse gases) and climbing at more than 2.5 ppm per year. How much more is too much? The size of response of the climate to greenhouse gases is uncertain, so we cannot say what level of emission is safe. However, that uncertainty has been well studied and we can say that, for example, a rise of greenhouse gases to 500 ppm will imply a 70% chance of exceeding the 2°C target. A rise of 450 ppm will mean a 50% chance [9]. These percentages are themselves uncertain, but it is clear that there are big risks with any increase of emissions. A final note – delaying actions means that by the time we start cutting emissions, there will already be more carbon in the atmosphere. Thus delay requires much harsher cuts.

4 Summary and conclusions

It is clear that the Auckland Region water resources planning now and in the foreseeable future will face continued uncertainty about climate change and its impact. However, although planning under an uncertain climate is unavoidable. By identifying the likely climate changes (e.g. rise in temperature, sea level and CO_2 level) in the Region, this study provides guidance to planners in terms of possible extremes, best guesses, and more likely direct of change. For example, the greenhouse gases emissions results showed that the level of CO_2, CH_4, and N_2O increased, and will continue to rise as more and more energy being used. The total amount of CO_2 in the air is increased by 16.6% (at a rate of 1.5 ppm/year) since 1970, and this may increase to 400 ppm by 2020 in the Region. The amount of CH_4 and N_2O were 1739 and 322 ppb (parts per billion), respectively, in 2006. It is estimated that the level of these two gases may increase to 1768 and 326, respectively, by 2020. The results showed that a significant shift in the climate of the Region has occurred over the last century. The mean temperature increased by 1.5°C over the last century (i.e. from 14.95°C in 1900 to 16.3°C in 2000), and it is estimated that it may increase by 0.15°C by 2020. The average annual rainfall in the Region varied between 1241 and 1276 mm since 1925, and it may increase to 1300 mm by 2020. It is known that when climate is warmer, rainfall tends to be heavier because there is more moisture in the atmosphere. The sea level at Auckland port increased by 19 cm during the past century at a rate of 1.4 mm rise per year and it is estimated that it may increase to 1928 mm by 2020. The results showed that 136 million cubic meters (Mm^3) of water (106 Mm^3 of surface and 30 Mm^3 of groundwater) is allocated each year for all purposes. The main use of the allocated water in the Region is within domestic households. The overall average water demand remains just below 54% of the total allocated groundwater. This shows that the available groundwater resources are sufficient to meet the water demands in the Region. From a water resources perspective, the results suggested no immediate cause for alarm. So, although it is reasonable to conclude that future climate change may lead to increased surface water resources in the Region. The scale of the climate impact may vary from negligible to significant by 2020 (depending on the catchment characteristics and climate change). Thus, it is expected that there will be no substantial decrease in water supply over the next few decades, and even maximum plausible changes to 2020 are unlikely to be critical – potential benefits in terms of increased water resources availability seem more likely.

References

[1] Auckland Regional Authority. (2005), *Proposed Auckland Regional Plan: Air, Land and Water.* Auckland Regional Authority. Auckland, New Zealand.

[2] Bannister, R. Crowcroft, G. and Johnston, A. (2004), *Auckland Water Resources Quantity Statement June 2003 – May 2004.* Auckland Regional Council Technical Publication. Auckland, New Zealand.

[3] Climatic Research Unit. (2004), *Global Temperature Records*. School of Environmental Sciences, University of East Anglia, Norwich, UK.

[4] Intergovernmental Panel on Climate Change (IPCC). (2007), Climate Change 2007: The Physical Science Basis. Summary for Policymakers approved at the 10^{th} Session of Working Group I of the IPCC, Paris, February 2007. IPCC Secretariat, c/o WMO, 7bis, Avenue de la Paix, C.P.N 2300, 1211 Geneva 2, Switzerland.

[5] Ministry for the Environment. (2003). Climate Change National Inventory Report, New Zealand Greenhouse Gas Inventory 1990-2001. Ministry for the Environment, Wellington, New Zealand.

[6] New Zealand Climate Change office. (2006). *Climate Change Impacts*. Environment House, 23 Kate Sheppard Place, Thorndon PO Box 10362, Wellington, New Zealand.

[7] National Institute of Water and Atmospheric Research (NIWA). (2003). *The greenhouse effect a New Zealand perspective on climate change.* NIWA Information Series No. 29. ISSN 1174-264x. Wellington, New Zealand.

[8] National Institute of Water and Atmospheric Research (NIWA). (2006). Climate change and global warming. Retrieved July 1, 2006, from http://www.niwascience.co.nz/edu/students/faq/change.

[9] The Royal Society of New Zealand. (2006). Royal Society Alert – Issue 446. Received on October 26, 2006. Also accessible on http://www.rsnz.org/news/sciencealert.php.

[10] Wratt, D. (1998). Intergovernmental Panel on Climate Change (IPCC) Assessments. National Institute of Water and Atmospheric Research. Auckland, New Zealand.

A case study for the Ecovillage at Currumbin – integrated water management planning, design and construction

C. J. Tanner
Bligh Tanner Pty Ltd, Australia

Abstract

This paper will present a case study of the planning and development of the multi-award winning the Ecovillage at Currumbin, Queensland. The paper will focus on the integrated water management systems that have been implemented at the Ecovillage. The case study will present the vision, objectives and implementation of the Ecovillage including tracing the vision and objectives through the life of the development, comparing the outcomes to the objectives. The Ecovillage is a new type of development in which the objectives address the triple bottom line – economic, social and environmental – and result in new forms of development, for example reversing the usual ratio of private land: community land.

The integrated water management systems are described briefly as follows.

- There are no connections of either water or wastewater to municipal supplies, and it is understood that this is the first residential development in Australia to achieve this.
- All rainwater falling on house roofs is collected in rain tanks and used to supply all potable water needs to the householder.
- All site stormwater is managed by a system of swales, bio retention filters and ponds so that the development does not change either the water quality or water quantities of site runoff from pre to post development. Site stormwater collected in ponds is available for re-use to agricultural areas of the development.
- All wastewater is collected via a low infiltration sewer reticulation system, treated to Class A+ and recycled for site irrigation, household toilet flushing and household external uses eg car washing.

The lessons learned are wide ranging and include the need to develop a strong process based delivery from concepts through to construction, the need for systems thinking in design, with complex analysis supporting detailed design, and innovative construction management to achieve a world's best practice development.

Keywords: subdivision, planning processes, water sensitive urban design, integrated water management.

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1 Introduction

Located on Currumbin Creek in the Gold Coast of Queensland, the Ecovillage at Currumbin is a sustainable development in the fastest growing region of the OECD in the world's most urban nation, which is also the driest inhabited continent. The Ecovillage is a 144-lot subdivision with no mains connections to water or wastewater. The developer, Landmatters, aims to adopt world's best practice and to create the world's best Ecovillage. Sustainability objectives direct all aspects of the development. The site is self-sufficient for water supply and all wastewater will be beneficially recycled. The site has achieved a 20% residential: 80% community open space ratio – the reverse of most conventional developments. Dwellings are clustered around community greenways; substantial areas are set aside for food crops, recreational open space and conservation areas. Recycled building materials have been used wherever possible. An on-site recycling centre will be provided. A resource management system, Ecovision®, monitors and manages resource use [1].

In Australia it is clear that a more sustainable approach to the management of urban water resources is needed: the pressure of growing population, limited new water sources and climate variation all demand change. Nowhere are these factors more keenly felt than in South-East Queensland. The purpose of this paper is therefore not to discuss the need to promote sustainable integrated water management design in urban development, but rather to report on a unique project in the region that is substantially completed and demonstrates what can be achieved. The Ecovillage at Currumbin recently won the Queensland Environmental Protection Agency Award for Most Sustainable Development 2006 and the Urban Development Institute of Australia's (Queensland) Best Sustainable Development and Best Small Subdivision in 2006. This project is significant for several reasons: rainwater is used as the primary source of drinking water supply; wastewater recycling provides water for residential toilet flushing and garden watering; water sensitive stormwater design is incorporated; and it demonstrates what can be achieved where community title allows for common ownership and mutual responsibility [1].

Bligh Tanner is the principal consultant for the Ecovillage and has been instrumental in the development and delivery of the water management systems. The significant contribution of other consultants is also acknowledged, in particular Andrew King (EECO) and WBM Oceanics (part of the BMT Group).

2 Objectives and the planning process

A rigorous approach to the planning and design process was adopted. The first step was the definition of a vision and objectives specific to the development. The vision adopted was 'To create the world's best Ecovillage' [2]. To support the vision, the developer and consultant team brainstormed objectives called Desired Environmental Outcomes (DEOs) [3]. Furthermore, the DEO's reflect a number of the objectives of the local shire planning scheme, some of which are listed below:

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The local Shire Council town plan required, among other things, the following:

- to minimise disturbance to natural landscape and wildlife habitats;
- to protect areas of nature conservation, buffers, separation of areas and continuity of open space should be established around these regions;
- to minimise disturbance to surface drainage, water courses, tidal and ground water movement;
- to conserve land and soil resources;
- to protect marine and freshwater habitats and water quality;
- to link areas of habitat and provide corridors for wildlife movement.

The DEO's relate to the requirements outlined above, and specific objectives of the Developer. They encompass both local and global considerations and are expected to be sustained over the full life of the development.

2.1 Ecological objectives

Ecol. 1 Restore, maintain and enhance biodiversity through the protection and enhancement of existing significant habitat;

Ecol. 2 Minimise initial and continuing consumption of resources and energy through the application of energy efficient design principles; the reduction of private motor vehicle usage; optimizing local food and production opportunities; and the reuse and recycling of water, wastes and other materials;

Ecol. 3 Minimise impact and change to air, soil and water, thereby ensuring equity for all elements of the natural environment whether living or inanimate;

Ecol. 4 Promote awareness and understanding of sustainability, including ecological issues and reduced energy consumption. and reduced materials consumption;

Ecol. 5 Minimise impact on global environment by optimising local ecological food and material production opportunities.

2.2 Social objectives

Soc. 1 Respect and honour Indigenous and other cultural, historical and spiritual values of the land and its surrounds;

Soc. 2 Enable social equity and diversity, honouring differences and catering for the needs of individuals through the different stages of life;

Soc. 3 Maximise health, safety and comfort in the built environment to provide enduring quality of life;

Soc. 4 Foster and promote social cohesion within the village community and a deep sense of human connection to and interdependence with the land;

Soc. 5 Utilise aesthetic sensitivity to create a continuing sense of place and beauty;

Soc. 6 Facilitate integration of the village with the broader local community through the shared achievement of common objectives and the promotion of openness within the village.

2.3 Economic objectives

Econ. 1 Promote initial and ongoing ecovillage economic viability through excellence of design;
Econ. 2 Minimise operational and maintenance costs;
Econ. 3 Minimise obsolescence through enduring component life cycle design;
Econ. 4 Provide for change and re-use at minimal cost / loss;
Econ. 5 Enable economic productivity and ecological contribution to local and world systems and economies.

In keeping with the DEOs, in particular Social Objectives 1 and 6, the developers chose to advertise their intention to develop the land before any plans or fixed ideas had been determined. The land is located in a sensitive environment in which lives a close and generally conservative community. Notifying the community of the developers' intentions was courageous because it gave the public far greater opportunity to influence the local authority and other community members against the development. The developer hosted a community meeting at which the vision and DEOs were explained, and a week-long public consultation was held. The open consultative approach succeeded extremely well, with most community input aimed at positively influencing the outcomes. Community members were comforted by the vision and DEOs because they understood how these would help protect the environment in which they lived. In addition, many members of the public were interested in the water cycle, in understanding the innovative approaches which were being discussed and in public health issues. For example, there was widespread support for UV disinfection rather than chlorination of water because it was recognised as a healthier outcome that was also better for the environment.

Finally, the objectives have been significant in the management of the entire development, not only the planning process because the DEOs underpinned the strong direction of the development processes. At all stages of the design process, ie concepts through to construction specifications, the DEOs have been used to guide the overall outcomes. For example, consider how the use of rainwater tanks relates to the three DEOs below (one from each category: Ecol 3, Soc 3 and Econ 2):

- minimise impact and change to air, soil and water, thereby ensuring equity for all elements of the natural environment whether living or inanimate;
- maximise health, safety and comfort in the built environment to provide enduring quality of life;
- minimise obsolescence through enduring component life cycle design;

The use of rainwater tanks with appropriate treatment satisfies all three of these objectives in these ways:

- reducing the directly connected impervious area, which benefits stream flow;
- reducing the amount of runoff needing to be treated to manage the water quality impacts of urbanisation;
- by using with appropriate filtration and disinfection rain water provides a safe and healthy water source for the users;
- a rainwater tank has a relatively high capital cost, (although not much more than the costs of providing trunk water supplies to lots) but maintenance costs are low, and replacement life cycles are long, so life cycle costs are comparable to alternatives.

3 Design outcomes – IWM initiatives

3.1 Integrated water management systems description

This section describes the integrated water management systems. The Ecovillage has no municipal water or wastewater connections – probably the first residential development in Australia to achieve this. The decision not to have these connections has significantly influenced the site water management strategies.

- All site stormwater is managed by a system of swales, bio retention filters and ponds so that the development does not change either the water quality or water quantities of site runoff from pre to post development. Site stormwater is collected in ponds from where it is available for re-use on agricultural areas of the development. Coupled with natural vegetation, the ponds provide a highly aesthetic environment for a residential development.

Figure 1: Stormwater pond.

- All rainwater falling on house roofs is collected in rain tanks and used to supply all potable water needs to the householder. Each domestic tank will also provide a 5 kL fire-fighting volume and a stormflow attenuation zone. The water will be filtered and UV disinfected prior to use. A range of tank sizes has been determined based on house bedroom numbers: approximately 20 kL for a 1-bedroom house to 40 kL for a 3-bedroom house. These tanks will provide 99% of potable water requirements. If the tank runs low then the householder will need to purchase water by tanker. Householders will be encouraged to use water wisely and a comprehensive range of water efficient devices will be required in each house. Ecovision® will provide quick access to household water consumption data and allow the Body Corporate to restrict supply to individual gardens if use is exceeds parameters set by the Body Corporate.

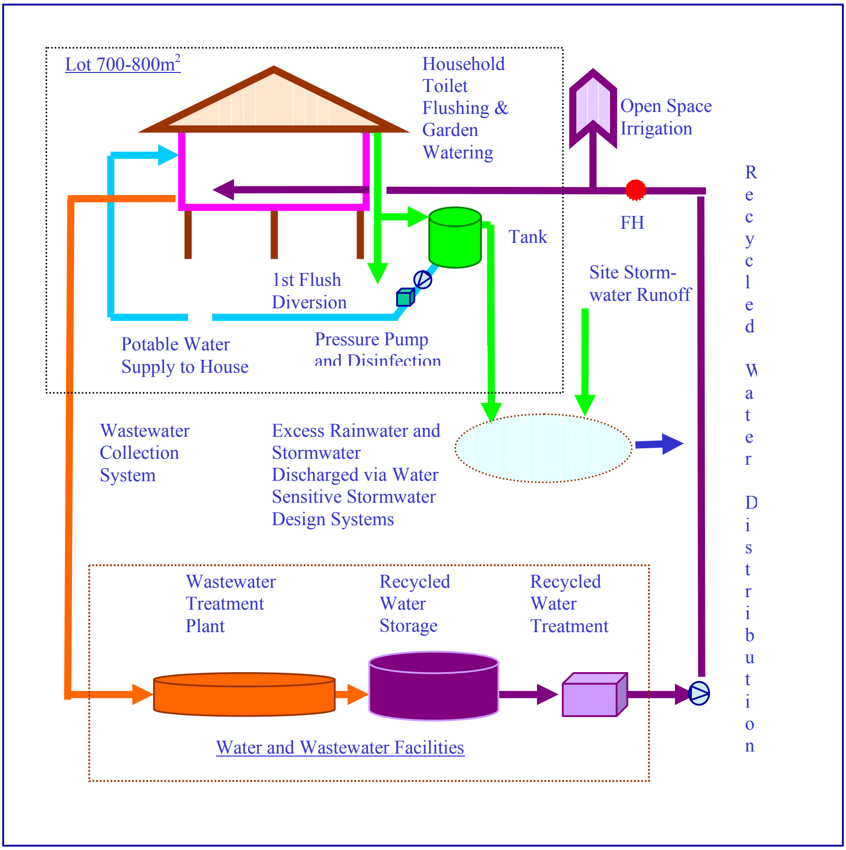

Figure 2: Integrated water management schematic.

- All wastewater is collected via a low infiltration sewer reticulation system (heat welded PE pipes; ‘Poo Pit’ access chambers instead of manholes), treated to Class A+ and re-cycled for site irrigation, household toilet flushing and household external uses such as car washing. Secondary wastewater treatment is provided in an Orenco Advantex® textile filter system incorporating primary pre-treatment and an anoxic zone for de-nitrification. Final polishing is provided by Memcor micro-filtration, UV disinfection and residual chlorination. There will be no direct discharge of recycled water from the site. The design allows for 98% beneficial reuse with the balance dispersed by over-irrigation on the site. The water and wastewater system is illustrated below.

3.2 Significance of the IWM systems used

The integration of the water systems is a key innovation. Traditionally these systems have been designed and constructed as separate systems, with stormwater and wastewater treated as disposal items and water to be sourced externally from independent systems. This approach fails to recognise the potential of each of these water systems as a resource. An IWM system is more complex to understand and model during the design process than the traditional model. It requires experienced engineering and environmental input in terms of systems and detail design.

The stormwater conveyance systems were all modelled in XP-SWMM. This model provides for hydrological routing based on synthetic or historically recorded rainfall patterns, through complex urban systems incorporating houses, rain tank systems, swale and piped drainage, bio retention filters, wetlands and the like. It also has the capability to model water quality (although that was not used in this case). The stormwater model must properly account for detention and retention in the system, because the accumulation of many small storages, in rain tanks, swales and bio retention filters is significant.

The MUSIC model was used to design the water quality management systems. This model simulates rainfalls and pollutant decay using exponential decay rate equations empirically calibrated to match experimentally measured results. The model tallies daily rainfall and evaporation, determines runoff and routes this through the water quality devices.

The MEDLI model was used to simulate the mass balance of water and nutrients for irrigation of land using recycled water. This also provided an estimation of likely frequency of discharge to the environment and the water quality of the discharge, which was then used in the MUSIC model.

Bligh Tanner developed an Excel spreadsheet to undertake a water balance for households using rain tank systems and for the overall site water balance.

During the concept design stage Multi Criteria Analysis, including life cycle cost components, was undertaken to review options in the systems against the DEOs.

3.3 The costs of IWM

The up front costs of water infrastructure are generally greater at the Ecovillage than for more traditional subdivision. However, these costs do not consider any of the benefits that the self-sustaining, environmentally sensitive systems of the Ecovillage provide. Nor do they take into account an environmental cost of the traditional systems. In the Pimpama Coomera Strategic Stormwater Study [4] a rigorous assessment of Business as Usual (BAU) costs compared to an integrated water management (IWM) approach found that the IWM costs were only marginally higher than BAU on a life cycle basis.

The water management system adopted for the Ecovillage has wide ranging benefits. It is self-sufficient, it provides the Ecovillage Community with water for agricultural uses, it provides aesthetic benefits, and it provides all the drinking water for the community. The environmental impact is considered to be low, and nutrients consumed by the community will be recycled to grow food for the same community.

4 Contracting and construction innovations

Infrastructure for the Ecovillage is two-thirds completed. Several major construction contracts have been let and completed during this process. Satisfying the Vision and DEOs, while maintaining quality and cost control of contracts and providing sufficient flexibility to accommodate site led modifications to works has required an alternative approach to standard construction contracts. The key innovations in this regard have been:

- Short-listing suitable contractors (to 2 or 3) by an extensive review facilitated by a comprehensive Expression of Interest process;
- Preliminary tender pricing by short listed contractors to select a preferred contractor;
- Negotiated contracts with the preferred contractor;

The negotiations with the preferred contractor have been extensive including reviews of:

- All works required detailed discussions of proposed work method statements, including consideration of alternatives, sometimes at greater monetary cost, but lower environmental impact;
- All material supplies, again including consideration of alternatives;
- Environmental and administrative management systems;
- Contractors Site personnel with a focus on the key site people being personable and flexible, and obviously competent at their work;
- Pricing of any unusual job scope items;

These negotiations have been completed prior to signing contracts and commencing work. However, at the time the preferred contractor is selected, a strong commitment has been made to them, so that they have felt comfortable proceeding with the time consuming, hence costly negotiations.

Key outcomes from this process have been:

- High site attendance by the owners, or senior management of the construction companies to allow quick decisions on matters of costs, timing, resources and alternative work methods. This has been specifically required and separately costed in the construction contracts;
- Introduction of a system of hold points to allow the developer and consulting team input to the procurement of materials, and work methods for the construction phase. This has enabled wide-ranging discussion of

alternative designs not necessarily considered during the design process, or of construction methods and materials to obtain better environmental outcomes.

An example of this is the construction of gravity sewer reticulation. The design (and local authority guidelines) generally called for sewers to be constructed at a 1m offset from allotment boundaries. This location often fell on land with relatively steep side slopes. Preliminary work method statements for the construction had been submitted, reviewed and approved by the developer. This was reviewed on site and alternatives were determined to shift the sewer off the side slopes, often up to 10m clear of the allotment boundaries, and to use low grade geo-fabrics over existing grass cover to protect it during machine operations. Shifting off the side slopes reduced the volume and surface disturbance of earthworks and the geo-fabrics protected the underlying grasses and demarcated an area in which the construction vehicles were to work. Protecting grasses reduced the potential for erosion and sediment transport to the creek to almost nil.

5 Conclusions

The conclusions are wide ranging and include the following points.

- A strong process is essential to maintain the direction and integrity of an ecovillage development, in particular when complex inter-related issues arise;
- Using an integrated water management system it is possible to develop a community that has a very low impact on the environment, on local and regional sources of water, and on local waterways. To achieve this designers must be systems thinkers and capable of detailed design using complex analysis;
- A flexible approach to construction contracts is required, recognising the costs and benefits to both parties (Contractor and Developer) of a collaborative approach;

References

[1] Hamlyn-Harris: Bligh Tanner Pty Ltd;
[2] Landmatters Pty Ltd;
[3] Landmatters Pty Ltd, Bligh Tanner Pty Ltd, John Mongard Landscape Architects Pty Ltd, Andrew Hall Town Planning Pty Ltd, WBM Oceanics Pty Ltd;
[4] Gold Coast City Council

Innovative design and solutions for mine water management on an alluvial floodplain

B. Tite[1] & T. Reedman[2]
[1]*Water Section, PB, Australia*
[2]*Environment and Communications Section, PB, Australia*

Abstract

The Curragh North coal mine is located on an alluvial floodplain of the Mackenzie River, in central Queensland, Australia. The catchment area at the mine site is 50,000 km^2 and the site had been inundated during previous floods. Until recently, development of the site had not proceeded due to flooding and water management risks and the difficulties involved in overcoming these risks.

PB's innovative design provided solutions to two problems which were deemed critical to the project's viability, namely a sustainable water supply for the mine and cost-effective flood protection. A holistic design approach was used to address the project's water supply and water management challenges:

- daily water balance modelling of the mine water management system to maximise on-site water harvesting;
- design of a two-way pipeline to enable exchange of water to and from an existing final void, reducing evaporative losses and on-site dam infrastructure requirements;
- creation of dams within reshaped spoil piles to maximise the water harvesting potential as the mine expands;
- design of controlled release points to allow spills to the Mackenzie River only when the river flows are between set limits.

Keywords: water management, harvesting, hydrologic modelling, mining.

1 Introduction

The Curragh North coal mine is located 200 km west of Rockhampton in central Queensland, Australia. Wesfarmers Limited won the right to develop the coal deposit and contracted PB to design the mine's civil infrastructure.

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The development concept for the mine is an open-cut mine producing up to 7 Mt/a of run-of-mine (ROM) coal over a 25-year mine life. The coal will be transported from the Curragh North Mine along a transportation corridor to the existing Curragh Mine for processing and rail load-out.

The mine site is located on an alluvial floodplain of the Mackenzie River where the catchment area is almost 50,000 km^2. The natural floodplain in the vicinity of the mine site is subject to relatively infrequent flooding during events in excess of the 1 in 10-year average recurrence interval (ARI) event.

The mine site is protected from flood ingress by a 22-km-long perimeter levee, designed to provide the dual functions of external flood protection and management/containment of internal site water. The topographic constraints, the mine layout and the dual function of the levee combine to effectively provide a water storage facility located within an area subject to flooding.

This paper describes the water management aspects of the project, including the design of the water harvesting system, which provides a reliable water supply, maximises the time when the pits are available and manages uncontrolled releases of contaminated mine water to the Mackenzie River.

2 Climate considerations

Water is a valuable, yet limited, resource throughout the region. Competing demands from industry, farming, communities and the environment have led to the preparation of governmental regulation in the form of water allocation schemes.

Allocation of water throughout the region must be in accordance with the water supply scheme for the Fitzroy Basin Water Resource Plan (WRP), the Fitzroy Basin Resource Operations Plan (ROP) and the Interim Resource Operations Licence for the Nogoa Mackenzie, all under the *Water Act 2000*.

Due to these regulations, users cannot simply extract water from rivers and streams in an uncontrolled manner. Limited availability of water allocations and the subsequent high costs makes sustainable water management critical to the success of mining projects throughout this region.

Rainfall in the project area is highly variable and typically unreliable. On average, almost half the annual rainfall falls during the summer months from December to February. Based on 100 years of historic data, the average annual rainfall for the mine site is 590 mm.

Temperatures in the area can range from 0°C to over 40°C, with the summer averages between 20°C and 31°C and the winter averages between 12°C and 26°C. Evaporation rates vary markedly throughout the year, depending on the season and the temperature. The average annual evaporation of 2,190 mm is almost four times the average annual rainfall.

3 Water management system philosophy

The philosophy behind the water management system (WMS) for the Curragh North Mine is to retain and re-use as much on site run-off as possible, inside the levee system. The WMS has been designed to:

- provide a reliable water supply for mining operations, for the entire 25-year mine life
- minimise the volume of run-off entering mine pits, thereby maximising pit availability for mining operations
- eliminate uncontrolled discharge and limit the frequency and volume of controlled discharge from the project to off-site receiving waters.

Local run-off within the levee system is directed to a series of water storage dams via overland flow paths and drains. These dams act as both sedimentation dams and as water supply dams. The water storage dams are located based on naturally occurring depressions within the mining area.

Four dams are located alongside the flood levee around the perimeter of the site. These dams collect site run-off and transfer water to a central dam via pipelines. The central dam is connected by a two-way pipeline to a final void at the existing Curragh Mine. The final void is used as a balancing storage, enabling excess water from Curragh North to be efficiently stored at Curragh and then brought back when needed.

Pit water is collected in pit floor sumps and subsequently pumped to the water storage dams for storage and site use. Controlled release points are provided at all water storage dams to ensure that excess water can be safely released to the river. Pit water is only released after it has been diluted in the water storage dams, enabling water quality target levels to be met in accordance with the Environmental Authority (EA) conditions.

4 Water management system layout

The water storage dams have been designed to satisfy the dual functions of meeting mine water demands and controlling contaminated run-off. The water storage dams comprise a lower level excavated sump storage component and a higher level overflow storage component.

The concept of providing sump storage and overflow storage for the dams was adopted to limit nuisance flooding, by storing run-off from minor events below the general ground level. Maximum excavated depths for the sump storages are typically 4 m. This concept also minimises losses to evaporation and seepage due to the reduced surface area of the excavated storage.

During major events, the sump storage will fill and water will spill out onto the overflow storage component. The overflow storage comprises the natural ground beyond the extent of the sump storage and the plan extent of the overflow storage is constrained by the flood levees, spoil dumps, pit protection bunds and the access road. Pit protection bunds provide protection to the pits and will be relocated as mining progresses. The overflow storage areas enable the retention of most of the on site run-off within the levee system by providing large storage volumes above the natural ground level.

The WMS includes five water storage dams. Four satellite dams are located around the perimeter of the mine site and are connected to a central dam via one-way pipelines. While most of the storage dams are excavated below the level of the natural floodplain, some storages are created by ponding against the flood levee and by ponding against highwall dams.

The central dam is connected by a 15 km long two-way pipeline to a final void at the neighbouring Curragh Mine. The final void is used as a balancing storage, enabling excess water from Curragh North to be stored at Curragh and retrieved when needed. The void's small surface area optimises on site water harvesting by minimising evaporative losses. Requirements for additional on-site water storage infrastructure is also reduced, allowing a smaller mine footprint.

After year five, it is proposed to shape the spoil dumps to provide additional internal drainage areas, which will allow surface run-off from the spoil dumps to be captured and stored in dams created within the spoil dumps. These spoil dams are designed to allow collected water to be harvested, transferred back to the water storage dams via pipes or chute drains and then reused on site. This transfer will be controlled with flow valves or stop logs to ensure that flow from the spoil ponds does not occur when the receiving water storage dam is full. Seepage from the spoil dams will pass through the spoil and will eventually end up in the pit sumps, from which it will be pumped to the water storage dams.

The spoil dams are designed as long, narrow storages cut into the batter of the spoil dumps, with balanced cut and fill earthworks. This shape will minimise the interference to spoil dump planning, while also allowing maximum use of the mine area within the levees. This has led to reductions in the mine footprint, the levee length and the construction costs.

A network of drainage channels direct site run-off to water storage dams thereby minimising on-site ponding. Highwall bunds are used to divert water to storages. After year five, spoil will be placed against the flood levee, enabling drainage channels to be formed in the spoil. As the spoil dumps rise in height, additional contour drains will be constructed in the batter face to allow the safe passage of run-off from the top of the spoil batter to the water storage dams

5 Water demands

Water demands for the Curragh North project are satisfied by a combination of clean water captured from site run-off over undisturbed catchment areas, dirty water captured from site run-off over disturbed catchment areas and supplemented clean water pumped from the Mackenzie River. Total water demands were determined by summing the component demands from dust suppression, vehicle wash down, potable use and construction requirements.

Water demands were calculated for six snap shots through the mine's 25-year life span. The maximum predicted water demand for the mine was calculated as 1,300 ML/a and this was expected to occur during years 2-5 of the mine's life.

6 Water sources

Rainfall falling within the perimeter levees is captured, stored and used within the site as the highest priority water supply. The balance of the water requirements is satisfied by a supplementary surface water allocation from the Mackenzie River.

An estimate of groundwater inflow to the pits from the Permian sequence indicated that the groundwater inflows to the pits would be between 100 ML/a and 300 ML/a, depending on the assumed radius of influence.

7 Daily water balance modelling

Daily water balance modelling was undertaken to assess the performance of the WMS. The modelling was used to predict the reliability of supply in satisfying the estimated water demands, to predict the pit availability and to predict the frequency and volume of releases from the WMS.

PB have developed an in-house software package called WAMAN (WAter MANagement), which performs daily water balance calculations and was written specifically for WMS analysis. The rainfall–run-off engine of WAMAN is identical to the well-known AWBM (Australian Water Balance Model).

WAMAN is capable of simulating the long-term behaviour of water management systems, which include complexities such as surface and underground water storages, variable water demands, external supply sources, variable pumping rates and flood harvesting. Model inputs include historic rainfalls, evaporation losses, seepage losses, water demands, supply sources, contributing catchment areas and volumes of on-site storages. Model outputs include catchment run-off, system yield and system spills.

8 WMS reliability

The WAMAN model was used to assess the reliability of the WMS. The reliability simulations conservatively used lower estimates of catchment yield, lower estimates of groundwater seepage and higher estimates of water demands. The model cycled through 75 climate simulations, with each simulation representing a different 25-year period of historic climate record, the starting year being incremented by one year for each simulation. If the WMS was unable to satisfy the total site water demand on any one day in the simulation, the model recorded a supply failure for that day.

For each 25-year simulation, the model then summed the total number of supply failure days and the system's reliability was calculated as the percentage of failure days over the whole simulation period. Statistical analysis was then used to identify the WMS reliability corresponding to the fifth percentile historic driest climatic period. The target reliability for this climatic period was 95%, as instructed by the mine management.

The WAMAN model was configured to ensure that water demands were prioritised. Demands were satisfied first with water captured from local site catchments, with any shortfall being supplemented by the Mackenzie River allocation. A sensitivity analysis was undertaken to determine the optimal volume of supplementary allocation needed to satisfy the target reliability. The results indicated that a supplementary allocation of 600 ML/a was required, which is just under half the total water demands for the mine site.

The time dependency of the supplementary allocation throughout the mine's life was assessed. This assessment endeavoured to identify trends associated with the frequency and volume of allocation required from the river and whether the volume could be reduced over time, as water was stored in the WMS. The results indicated that while the WMS does reduce the dependency on the allocation, the volume and frequency of supplementary water is dominated by the prevailing climatic conditions, with no clear trend being evident.

A sensitivity analysis was performed on the pumping rate from the central dam at Curragh North to the final void at Curragh. This assessment confirmed the benefits of using the final void as a large off site holding tank. Pump rates ranging between 0 and 1,000 L/s were modelled, with the return pump rate set to match the daily demand. As expected, the system reliability improved with increased pumping rate, with the optimal rate being approximately 200 L/s. A pump rate of 80 L/s was needed to satisfy the target reliability.

9 Controlled releases from the WMS

The combination of space limitations within the mine site, large historical storm events and long distance pumping considerations, meant that it was impractical to store all local runoff within the WMS. Instead, the WMS is designed to ensure that controlled releases are infrequent and will not cause environmental damage. Gated release points are provided at all dams to ensure excess water can be safely released to the river in a controlled manner. Release conditions were stipulated by the Queensland Government Environmental Protection Agency's Environmental Authority (EA) for the Curragh North project. These conditions describe the water contaminant levels that must not be exceeded in a release. The conditions further require that release must only occur while the Mackenzie River is in flood (to assist dilution).

The WAMAN model was used to assess the frequency and magnitude of WMS releases to the Mackenzie River. The release simulations conservatively used higher estimates of catchment yield, higher estimates of groundwater seepage and lower estimates of water demands. The model cycled through 75 climate simulations, with each simulation representing a different 25-year period of historic climate record, the starting year being incremented by one year for each simulation. A supplementary allocation of 600 ML/a from the Mackenzie River was assumed.

The total number of spill days and spill events from the water management system were counted for each simulation, with a spill event being defined as one or more days of consecutive spill. Statistical analysis was then used to identify the system spills occurring within the fifth percentile historic wettest climatic period. This climatic period was chosen to represent a conservative situation with respect to the potential volume and frequency of releases, as instructed by the mine management.

The layout of the WMS and the perimeter levee prevent releases during a major flood in the Mackenzie River, as external river levels will be higher than internal dam water levels. Attempted releases during a major flood would likely result in overtopping of the WMS dams and spillage into the pits, due to river ingress.

Combining the EA release conditions with the release limitations during major flood events, results in a small window of opportunity for controlled releases from the WMS, namely when there is a minor or moderate flood in the Mackenzie River, but not when there is no flow a major flood.

Control of WMS releases at times when river levels are lower than the EA threshold or higher than dam water levels, can be achieved by careful operation of the release structures. By monitoring weather forecasts and radar patterns, mine operators can be aware of expected weather patterns, both for the immediate local catchment area and for the regional Mackenzie River catchment. If forecasts show that a large storm is likely to pass over the mine site, then operators can expect local run-off to the WMS dams within a day. Conversely, if forecasts show that a large storm is likely to pass over the regional catchment, then operators can expect river levels to rise within the next three to four days.

10 Water quality

The potential pollutant sources from coal mining activities include coal processing, site maintenance, petroleum storage, acid mine drainage, saline pit water and saline groundwater inflows through the coal seam. Water quality in storage dams was predicted using information gathered from groundwater monitoring at the Curragh North and Curragh mine sites and from direct measurement of water quality in the Curragh mine existing storage dams.

For Curragh North, the poorest quality water will be derived from pit seepage from the coal seam and alluvial aquifers. The inflows to the water management system dams will be sourced from pit seepage, surface water run-off from contributing catchments and from direct rainfall on the dam surface. Saline pit water will be diluted by the freshwater in the storage dams. Settlement of suspended solids will occur in the water storage dams, prior to any controlled release of waters to the Mackenzie River.

The results of groundwater monitoring indicated that electrical conductivity (EC) measurements for all bores at Curragh North are less than the EA trigger level of 4,500 μS/cm, with the exception of one high EC value of 7,600 μS/cm, which was markedly higher than all other readings from surrounding bores.

At Curragh mine, the water storage dams predominantly receive pit water inflows and the EC readings match those from groundwater. The EC readings for Curragh North groundwater are considerably lower, due to the diluting effects of the nearby Mackenzie River. The EC measurements at both mine sites demonstrate that the groundwater quality provides a good indicator of the quality in the water management dams.

A mass balance model was constructed using a spreadsheet to track the accumulation of salt in the water storage dams at Curragh North over time. Inputs to the mass balance model were sourced from the WAMAN modelling, including daily catchment run-off volumes, pumping volumes, release volumes, evaporation/seepage volumes and dam volumes. A salt concentration was applied to each inflow stream and the model tracked the total mass of salt inflow to the dams on a daily basis. Salt was assumed to gradually accumulate in the dams over time, with the only reductions in salt content being due to WMS releases, pumping and dilution. Plots of the salt concentration were prepared to identify the salt concentration during those critical times during WMS releases.

As a worst-case scenario, the highest recorded EC of 7,600 μS/cm was assumed for inflows from disturbed spoil catchments, for inflows and transfers from the out-of-pit spoil dump and for pumping from open pits. An EC of zero was adopted for all natural catchments. The model results indicated that the ponded EC is expected to gradually increase throughout the mine life, as salt is captured in the dams. Towards the end of the mine life, the salt concentration is shown to approach 4,000 μS/cm, which is within the limits specified in the EA.

11 Pit availability

Pit availability is a significant issue affecting mining operations. Water ponding at the bottom of the pits can cause disruptions to mining because mining operations will be delayed while the water is pumped out. It is common practice to excavate a pit sump in the lowest level of the pit, enabling any water entering the pit to naturally drain to the sump, thereby keeping the pit floor relatively dry. The size of this pit sump is critical. If it is too small, then the pit floor will be inundated on a regular basis, resulting in disruptions to mining operations and poor pit availability. If the sump is too big, pit availability will be improved, but pit floor access will be limited, also resulting in disruptions to mining operations.

Pit availability is defined as the proportion of days that the pit sump was not overtopped over the mine's life. The target availability for this scenario was 98% in any one year for all pits, as specified by the mine management.

Pit availability was assessed using the WAMAN model of the WMS. The pit availability modelling was run for the historical period corresponding to the fifth percentile historic wettest climatic period. A supplementary allocation of 600 ML/a from the Mackenzie River was assumed. This climatic period was chosen to represent a conservative situation with respect to the pit availability.

The model results indicated that a 10 ML sump would provide pit availabilities ranging between 96.7% and 99.2% for all dams. Highwall pumping rates of 200 L/s were adopted for the water transfer out of the pit sumps to the adjacent water storage dams. The longest consecutive period that the pits would be expected to be unavailable ranged between three and six days.

Sensitivity testing was undertaken to optimise the sump volume and the highwall pumping rate. Sump sizes ranging between 10 ML to 40 ML were checked and the highwall pump rates were changed from 200 L/s to 100 L/s.

The results indicated that only nominal increases in pit availability would result from increased sump size, but the benefits were outweighed by disruptions to mining. The provision of high rate pumps for the high wall was recommended due to significant reductions in the longest period of pit unavailability (8 versus 28 days), increasing the time available for mining operations.

12 Conclusions

Water is a valuable, yet limited, resource throughout the region. Competing demands from industry, farming, communities and the environment have led to the preparation of governmental regulation in the form of water allocation schemes. Rainfall in the project area is highly variable and unreliable, with almost half of the annual rainfall occurring during the summer months. Average annual evaporation is nearly four times the average annual rainfall.

The philosophy behind the water management system (WMS) was to retain as much run-off as possible on site, thereby reducing the amount of additional allocation required to satisfy the mine's water demands. Total water demands for the mine were determined by summing the component demands from dust suppression, vehicle wash down, potable use and construction requirements.

PB's in-house daily water balance model (WAMAN) was used to assess the performance of the WMS of the Curragh North Mine, in satisfying the estimated water demands, in predicting the pit availability and in predicting the frequency and volume of system releases. WAMAN is capable of simulating the long-term behaviour of water management systems, which include complexities such as surface and underground water storages, variable water demands, external supply sources, variable pumping rates and flood harvesting.

The water management system comprises four satellite dams located alongside the flood levee around the mine perimeter. These dams collect site run-off and transfer water to a central dam via one-way pipelines. The central dam is connected by a two-way pipeline to a final void at the existing Curragh Mine, 15 kilometres away. The final void is used as a balancing storage, enabling excess water from Curragh North to be efficiently stored at Curragh and retrieved when needed. This concept optimises the water harvested on site by storing it in an existing deep and narrow void that minimises evaporative losses. In addition, storage of water off site reduces the need to provide additional on-site water storage infrastructure, thereby providing more area for other mining activities.

System reliability was assessed using a conservative worst-case dry climate situation combined with higher water demand estimates. The model results indicated that a supplementary allocation in the order of 600 ML/a was required to meet the target 95% reliability, as required by mine management.

Reshaping of the spoil dumps will provide additional internal drainage areas, allowing capture and storage of surface run-off. The spoil dams have been designed as long, narrow storages cut into the batter of the spoil dumps. This concept allows maximum use of the mine area within the levees, thereby minimising the mine footprint and the levee length and, consequently, reducing

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costs. The spoil dams will provide additional water harvesting capacity, while their storage volume will also attenuate peak flows during major storm events, resulting in reduced risk of system spills to the Mackenzie River.

Gated release structures were provided at the water storage dams to ensure that excess water can be safely released to the Mackenzie River in a controlled manner. The release criteria require that water can only be released from the mine site only when there is a minor or moderate flood in the Mackenzie River. This means that there is a small window of opportunity within which controlled releases are permitted to occur.

Water quality release limits were stipulated in the Environmental Authority (EA). Dam water quality was predicted using information gathered from groundwater measurements at the Curragh North and Curragh mine sites and from direct measurements of Curragh dam water. Mass balance modelling tracked the accumulation of salt in the storage dams and identified the concentration during those times when WMS releases would occur. The results indicated that the concentration will increase throughout the mine's life, as salt is captured in the dams. Towards the end of the mine's life, the concentration is predicted to approach 4,000 μS/cm, which is still below the EA discharge limit.

Pit availability is a significant issue affecting mining operations and is defined as the proportion of days that the pit sump is not overtopped. Water in the pit causes disruptions to mining operations while waiting for water removal. The target availability was 98% in any one year for all pits, as specified by mine management. The results showed only nominal increases in pit availability due to increased sump size. High rate pumps for the high wall were recommended due to significant reductions in the longest period of pit unavailability.

The project was awarded a high commendation in the Engineers Australia Queensland Engineering Excellence Awards 2006. The judges were 'impressed with the water management system designed to provide a reliable water supply for mining operations, to minimise run-off entering mining pits and controlling off-site discharges to meet Environmental Protection Agency requirements'.

References

[1] Boughton, W.C. A Hydrograph-based Model for Estimating the Water Yield of Ungauged Catchments. Presented at the Hydrology and Water Resources Symposium, Newcastle, Australia, 1993.
[2] PB. Curragh North Coal Project — Hydrology and groundwater study for the environmental impact study, Wesfarmers. PB, Brisbane, Australia, 2003.
[3] PB. Curragh North Coal Project — Water management strategy design report, Wesfarmers. PB, Brisbane, Australia, 2004.
[4] Qld Department of Natural Resources. Water Act 2000 Water Resource (Fitzroy Basin) Plan 1999. Qld DNR&M, Brisbane, Australia, 1999.
[5] Qld Department of Natural Resources and Mining. Groundwater database search. Brisbane, Australia, 2004.
[6] Qld Department of Natural Resources and Mining. Fitzroy Basin Resource Operations Plan (ROP). Qld DNR&M, Brisbane, Australia, 2006.

Water resources management – a possibility for drought mitigation in wetlands?

O. Dietrich[1], H. Koch[2] & S. Schweigert[1]
[1]*Leibniz-Centre for Agricultural Landscape Research (ZALF) eV, Institute of Landscape Hydrology, Germany*
[2]*Brandenburg University of Technology Cottbus, Hydrology and Water Resources Management, Germany*

Abstract

The water balance of many wetlands in the North-East German Lowland is dominated by water resources management systems with drainage and sub-irrigation. These systems are integrated in the water resources management system of their whole river basin. Scenario investigations show the possibilities and constraints of different water resources management options within the wetland and in the basin. For the Spreewald wetland strategies for the mitigation of negative impacts of climate change are presented as an example.
Keywords: wetlands, water resources management, drainage, sub-irrigation, water balance model, climate change.

1 Introduction

Most of the wetlands in the North-East German Lowland are fens. After the last ice age they developed in spite of the climatic conditions with a mean annual precipitation of approximately 500 mm per year, because they got sufficient recharge from their basin and the discharge was blocked by natural barriers. In the last two centuries most of the fens were drained for agricultural land use. However, because of the low precipitation, the drainage systems were completed with a number of weirs as a prerequisite for intensive agricultural production in the 1970s and 1980s. Therefore, these regions have complex water resources management systems today, which are often integrated in the water resources management system of the whole river basin.

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The last decade, with dry summers and hot temperatures, shows that there is an increasing risk of droughts in these wetlands. Climate models forecast an additional threat, with increasing temperatures and decreasing precipitation in the summer months for the next few decades in north-east Germany. On the other hand, there are a lot of ways of enabling these areas to be used once more with groundwater levels more typical for wetlands. To do so, it is necessary to develop new water management strategies taking changing climatic conditions into account. This process can be successfully planned with the help of numerical models. The model systems WBalMo Spreewald (Dietrich *et al* [1]) for the Spreewald wetland and WBalMo Spree/Schwarze Elster (Kaltofen *et al* [2], Koch *et al* [3]) for the Spree River basin are suitable tools for such a task. They were developed, tested and used for scenario investigations in the context of global change within the research project GLOWA-Elbe (www.glowa-elbe.de). In this paper some scenario results will be presented and discussed.

2 Study region

The Spreewald wetland is situated about 70 km south-east of Berlin, located within the Spree River basin (Fig. 1). It is one of the most significant wetlands in Germany. The lowland region has an area of 320 km². It is characterized by a low mean annual precipitation of about 530 mm for the period 1961-1990 (HAD [4]) and rather high potential evapotranspiration (FAO grass reference evapotranspiration) of about 610 mm for the same period.

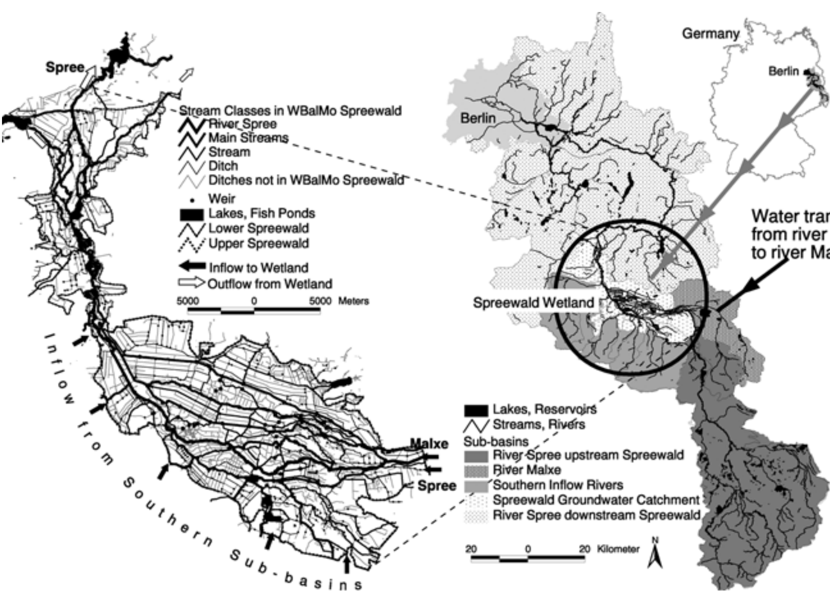

Figure 1: Running water system (classified for the water balance model WBalMo Spreewald) with weirs, main inflow and outflow of the Spreewald (left) and location of the Spreewald wetland south-east of Berlin in the Spree River basin (right).

The wetland soils are dominated by groundwater-influenced sands (49%), fens (33%) and loamy soils (18%). The land use of the wetland area is characterized by extensive grassland (44%), fields (23%) and forests (20%).

The wetland has a very dense stream and ditch system of about 1,600 km in length with more than 600 weirs to regulate ditch water and groundwater levels (Fig. 1, left). Therefore, the water balance of the Spreewald region is strongly influenced by the water management system within the wetland.

The Spreewald wetland is supplied with water from the Spree River (2,535 km²), the Malxe River (345 km²) and the southern sub-basins (1,160 km²) (Fig. 1, right). The water balance in these basins has been influenced by a number of opencast mines for more than 100 years. In the 1980s the pumping rates of mine discharges increased up to 30 m³/s (Grünewald [5]). The extensive groundwater drawdown in the mining region led to a large groundwater deficit in the basin. Today most of the opencast mines are closed. So the amount of mine discharges in the Spree River decreased to about 10 m³/s and will be reduced to 0 m³/s by 2040. Additionally, the residual mining pits have to be refilled with water and the man-made drawdown is being reverted. As a consequence of the current high water demand of the basin, water deficiency situations for the wetlands occur increasingly during the vegetation periods.

3 Method

The models used, WBalMo Spreewald and WBalMo Spree/Schwarze Elster, consider aspects of the water resources management in the wetland and in the basin. WBalMo Spreewald is a combination of a water management model (WBalMo®, WASY [6]) and a water budget model for wetlands with drainage / sub-irrigation systems (WABI, Dietrich *et al* [7]). This combination is a solution to fulfil the complex requirements of the wetland region. WBalMo Spree/Schwarze Elster is a complex water balance model with a great number of different water users in the basin.

The model system WBalMo represents the hydrological processes and the water management in a river basin. River basins are represented by simulation sub-basins, running waters, balance profiles, water users and reservoirs. The input values are stochastically generated time series. The water utilization processes are reproduced deterministically. The time step is one month.

WABI is a simple water balance model for groundwater-influenced areas with drainage and sub-irrigation systems. The study site is divided in sub-areas, the smallest area in which the groundwater level can be regulated separately. One important assumption is a horizontal groundwater level in each sub-area. The time step is also one month. The model requires target water levels and inflows for each sub-area. WABI is directly coupled to WBalMo. Each WABI sub-area is one water user in WBalMo. The sub-model WABI requires information about the elevation distribution connected with land use and soil types of each sub-area as well as storage for each sub-area.

In the model WBalMo Spreewald the wetland's complex system of streams and ditches was simplified. Only watercourses which are important for drainage and water surplus for the wetland sub-areas are considered. For 86 bifurcations of watercourses special rules were developed on the basis of expert knowledge, the current water management practice or depending on the water demand of water users downstream from the bifurcation. Changing distribution rules is one way of simulating different water management strategies with the model. The wetland area was divided into 197 sub-areas. Every sub-area is represented by one water user in the model. The calibration and validation of the model WBalMo Spreewald is described in detail in [1].

WBalMo Spreewald requires input data for precipitation, potential evapotranspiration and the inflow from the sub-basins into the wetland and the sub-areas at the boundary of the wetland. The climatic input data were prepared by project partners using the climate model STAR (Werner and Gerstengarbe [8]). The inflow from the sub-basins was calculated using the water management model of the Spree River basin WBalMo Spree/Schwarze Elster [2] based on the same STAR model climate data. Input series were made available for each month from 2003 to 2052 with 100 realizations of every year (Monte Carlo Simulation). The modelled time range of 50 years was divided in ten 5-year periods. Within each 5-year period the management options, climate trends, water demands of water users, etc. are unchanged.

4 Scenarios

The WBalMo models were used to determine the impacts of different water management options on the water balance of the Spreewald wetland under changing global conditions. Scenarios were defined as combinations of boundary conditions and water resources management options in the wetland. In this paper the boundary conditions for the wetland water balance are climate change (Wechsung *et al* [9]) and two water resources management options in the Spree River basin. The first option represents the current management practice and the second a water transfer of at most 2 m^3/s from the Odra River to the Malxe River (Koch *et al* [10]) (Fig. 1, right). The two water resources management options of the basin were combined with options in the wetland: (1) the current management practice and (2) another distribution of the inflow water within the wetland (Dietrich *et al* [11]).

Table 1: Definition of scenarios by combination of different water resources management options in the basin and in the wetland.

Scenario name		Water resources management in basin	Water resources management in wetland
1	Basis	Current practice	Current practice
2	Redistribution	Current practice	Redistribution of basin surplus within wetland
3	Transfer	Water transfer from Odra River	Current practice
4	Transfer with redistribution	Water transfer from Odra River	Redistribution of basin surplus within wetland

5 Results and discussion

The statistical evaluation of the model results was made for each 5-year period. In the following we compare and discuss the results of the period 2003-2007 (P1) with the last period 2048-2052 (P10). In the figures the first period of the basis scenario is also the reference status for the other scenarios. The bars in the figures represent the 50th percentile of 500 values per month. The caps show the range between the 20th and 80th percentiles.

5.1 Boundary condition – climate change

The climatic boundary conditions are the same in all scenarios. For a better interpretation of the model results the impact of climate change is represented by the climatic water balance (Fig. 2). Already, the balance of P1 shows a deficit for the months from April to September. But from June to August, especially, the deficit will clearly increase up to the last period (P10). This increasing deficit influences the water demand in the wetland, but also the inflow from the basin.

Because the period from April to September is most interesting for water scarcity situations in the wetland, the following figures will only show percentiles of this part of the year.

5.2 Recharge from basin into the wetland

The inflow from the basin will decrease in the summer months up to 2050 (Fig. 3). The reasons are the changed climatic conditions as well as the planned development of the mining activities in the basin. The inflow from the Malxe River, especially, will decrease in the future, because today there are two large opencast mines in this sub-basin, which are going to close by 2030. Then the pumping of mine water will stop and additional water is needed to refill the residual mining pits.

The water resources management option "water transfer from the Odra River into the Malxe River" could improve the water supply situation (Fig. 3). However, the volume assumed in the scenario is not sufficient to compensate for the decrease in the water inflow of the whole Spree River basin upstream of the Spreewald wetland due to climatic changes and a lack of mine discharges.

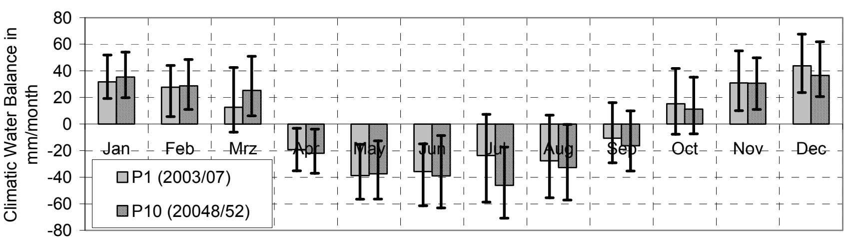

Figure 2: Comparison of the climatic water balance in the periods 2003-2007 (P1) and 2048-2052 (P10).

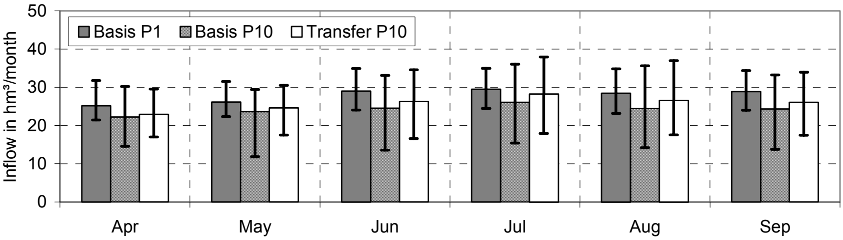

Figure 3: Change of inflow into the wetland in the periods 2003-2007 (P1) and 2048-2052 (P10) depending on different water management measures in the basin (basis, water transfer from Odra River).

5.3 Water demand of the wetland

The development of the wetland water demand is shown using the example of July for all ten 5-year periods. The values are influenced by the water balance parameters of precipitation, actual evapotranspiration and the water storage deficits of the previous month. The results in Fig. 4 show an increasing water demand of 35 mm between the median of the first and last 5-year period. The reasons for this are the changed climatic conditions with lower summer precipitation and higher potential evapotranspiration. This will also lead to higher actual evapotranspiration because of the near-surface groundwater levels. The consequence is that the wetland depends more on the basin inflow.

5.4 Water withdrawal of the wetland

Figure 5 shows the water withdrawal for all scenarios in P10 in comparison to P1 of the basis scenario. In all scenarios there is an increase in the water withdrawal from July to September because of the higher water demand. But the limited water yield from the sub-basins limits the withdrawal. Different water resources management options improve the utilisation of the existing water yield, but the increase in available water can not compensate for the overall increase in water demand (10 mm versus 35 mm in July). The consequences are decreasing groundwater levels in the wetland and decreasing outflow from the wetland.

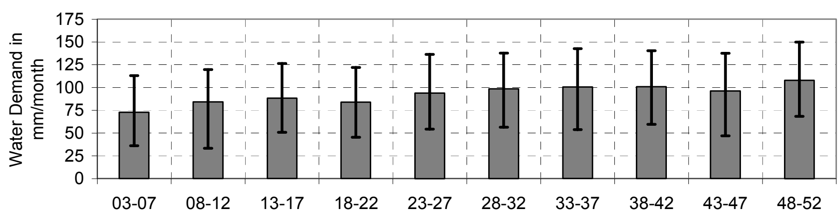

Figure 4: Water demand of the whole wetland area (basis scenario, July).

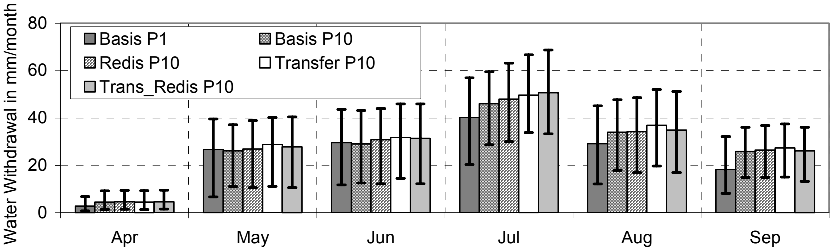

Figure 5: Water withdrawal of the whole wetland area from basin inflows in the periods 2003-2007 (P1) and 2048-2052 (P10) depending on different water management measures.

5.5 Groundwater levels in the wetland

The maps in Fig. 6 illustrate of the change of groundwater levels in July of P1 and P10 in the different scenarios. Figure 6A shows few differences in the central parts of the Upper and Lower Spreewald wetland. These parts are predominantly supplied with water from the main inflow of the Spree River. Only the sub-areas in the Upper Spreewald wetland, predominantly supplied with water from the Malxe River, have decreasing groundwater levels. The reasons are explained in chapter 5.2. The largest problems will arise in the border parts of the wetland because these sub-areas can only receive water supply from relative small sub-basins. It is difficult or even impossible to transfer water from the main inflow of the Spree River to all of these parts.

The different water resources management options (Fig. 6B-D) improve the situation in some parts of the Spreewald wetland. The redistribution of water from the Spree River inflow in the northern part of the central Upper Spreewald (in the basis scenario supplied by the Malxe inflow only) and the concentration on the central parts of the wetland, which are the most important parts for nature protection, lowers the threat of water scarcity in these parts. But it also leads to drier situations in other parts of the wetland.

Transferring water from the Odra River to the Malxe River improves the groundwater levels in the wetland parts predominantly supplied with Malxe water without a negative impact on other parts (Fig. 6C). The fourth scenario shows the largest increase in the groundwater levels (Fig. 6D). However, no management scenario leads to an improvement in the most-threatened border parts compared to the basis scenario in P10.

5.6 Discharge below the wetland

The discharge below the wetland is important for the Spree River and water users along the river downstream of the Spreewald wetland. Because of the water demand of these water users, there should be an outflow of the Spreewald wetland into the Spree River of approximately 12 hm^3 per month. Already in

July of P1 of the basis scenario this value is reached in 50% of the years only (Fig. 7). No management scenario improves this situation distinctly. The reason is the large water demand of the wetland.

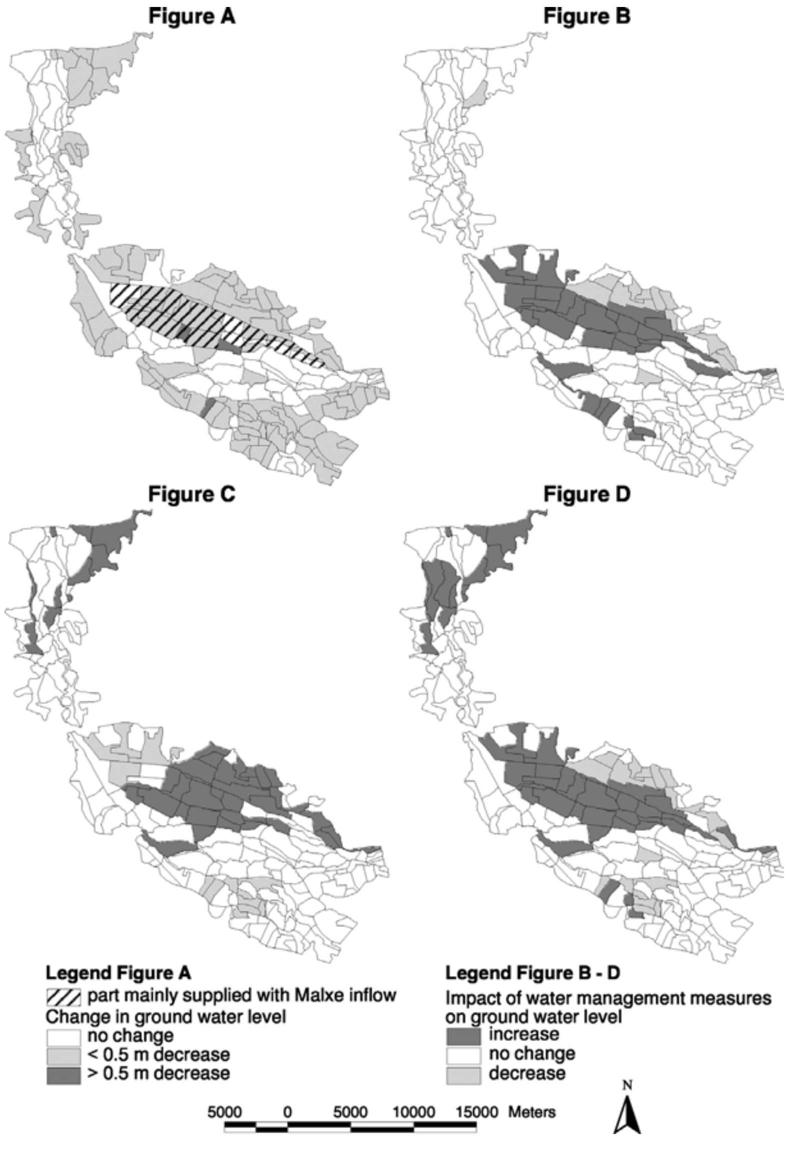

Figure 6: Difference between the July groundwater levels of the basis scenario for the periods 2003-2007 and 2048-2052 (A) as well as the difference between the basis scenario of the period 2048-2052 and the redistribution scenario (B), the transfer scenario (C) and the transfer with redistribution scenario (D) for 2048-2052.

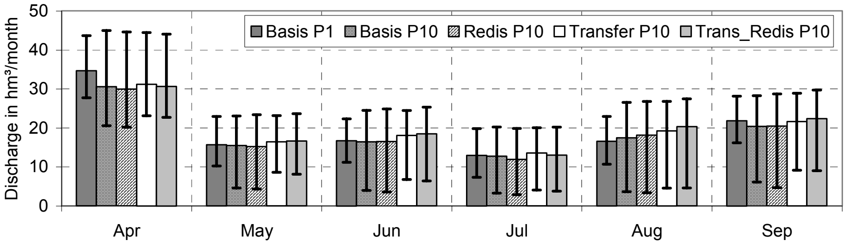

Figure 7: Discharge below the wetland in the periods 2003-7 (P1) and 2048-2052 (P10) depending on water management options.

The results show also increasing problems in the drier years. The 20th percentile values undershoot the 12 hm³ per month in P1 in the basis scenario only in July. In P10 this value will be undershot more distinctly from May to September. This could be a large problem in the Spree River basin in the future.

6 Conclusions

In connection with the WBalMo Spree/Schwarze Elster for the Spree River basin, the WBalMo Spreewald model offers a way to integrate water budget modelling in wetland areas and the water resources management of the whole basin in one model. It can be used to analyse the impacts of changing boundary conditions (meteorological, hydrological, economic conditions) as well as management options and to develop strategies to reduce unwanted impacts of global change.

The results show that climate change may produce large problems for wetlands in humid climatic zones in the future, especially if there are already water deficits under the present conditions. In the Spreewald wetland the water demand of the wetland area will increase in the future. The inflow from the basin will be insufficient to compensate for this increase in the whole region. Therefore, water deficit periods will occur more frequently. The consequences of frequent water deficits are deeper groundwater levels in summer, which differ depending on the inflow conditions and water distribution within the wetland.

The results of the scenarios are only a few examples of how water resources management strategies can help to reduce unwanted impacts and to mitigate droughts in wetlands in the future. The investigation of further strategies is necessary to find the best solution for all water users in the wetland and downstream. A socio-economic evaluation of the scenarios is also needed. The task is to find strategies which are seen by all water users as a good compromise.

Acknowledgements

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References

[1] Dietrich, O., Redetzky, M. & Schwärzel K., Wetlands with controlled drainage and sub-irrigation systems-modelling of the water balance. *Hydrological Processes*, DOI: 10.1002/hyp.6317, 2007.

[2] Kaltofen, M., Koch, H., Schramm, M., Grünewald, U. & Kaden, S., Application of a long-term water management model for multi-criteria assessment procedures – Scenarios of global change in the Spree catchment influenced by lignite mining. *Hydrology and Water Resources Management*, **48(2)**, pp. 60-70, 2004.

[3] Koch, H., Kaltofen, M., Grünewald, U., Messner, F., Karkuschke, M. Zwirner, O. & Schramm, M., Scenarios of water resources management in the Lower Lusatian mining district, Germany, *Ecological Engineering*, **24(1-2)**, pp. 49-57, 2005.

[4] HAD, Hydrologischer Atlas von Deutschland. Bundesministerium für Umwelt, Naturschutz und Reaktorsicherheit (Hrg.), 1998.

[5] Grünewald, U., Water resources management in river catchments influenced by lignite mining. *Ecological Engineering*, **17(2-3)**, pp. 143-152, 2001.

[6] WASY, WBalMo 3.0 – Interactive Simulation system for planning and management in river basins. User's Manual, WASY Gesellschaft für wasserwirtschaftliche Planung und Systemforschung mbH, Berlin, 157 p., 2006.

[7] Dietrich, O., Dannowski, R. & Quast, J., GIS-based water balance analyses for fen wetlands. In Holzmann H. & Nachtnebel, H.P. (eds.), Application of Geographic Information Systems in Hydrology and Water Resources Management. Vol. of Poster Papers, Vienna, pp. 83-90, 1996.

[8] Werner, P.C. & Gerstengarbe F.-W., Proposal for the development of climate scenarios. *Climate Research* **3**, pp. 171-182,. 1997.

[9] Wechsung, F., Becker, A. & Gräfe, P. (eds.), Integrierte Analyse der Auswirkungen des Globalen Wandels auf Wasser, Umwelt und Gesellschaft im Elbegebiet. Weissensee-Verlag, Berlin, 405 pp., 2005.

[10] Koch, H., Kaltofen, M., Schramm, M., Grünewald, U., Adaptation strategies to global change for water resources management in the Spree river Catchment, Germany. *International Journal of River Basin Management*, 4(4), 2006.

[11] Dietrich, O., Redetzky, M. & Schwärzel K., Modeling Water Balances of Wetlands with controlled Drainage and Sub-irrigation Systems. In: Kotowski, W., Maltby, E., Miroslaw-Swiatek, D., Okruszko, T. & Szatylowicz, J. (eds.), Wetlands: modelling, monitoring, management. Taylor & Francis, The Netherlands. A.A. Balkema Publisher (in press), 2007.

Considering salinity effects on crop yields in hydro-economic modelling – the case of a semi arid river basin in Morocco

C. Heidecke & A. Kuhn
Institute for Food and Resource Economics, Bonn University, Germany

Abstract

Agricultural production, especially date palm cultivation, is the major food and income source for people in the Drâa basin in Southern Morocco. However, the semi-arid river basin faces very low rainfalls and has suffered from a continuing drought over the last years. River water, as the principal source for irrigation, has been increasingly substituted by groundwater mining. This has led to an unsustainable downing of the groundwater table, increased salinisation problems, and has posed further constrains on the agricultural production potential. Without targeted water resources management, water available for irrigation will soon be depleted or too saline to be used for most crops. Consequently, farmers will not be able to maintain their production levels, and subsequently lose an important source of family income. The relationship between water use and agricultural production is represented using an integrated hydro-agro-economic simulation model with a spatial water distribution network of in- and outflows, balances and constraints. The model results are driven by profit-maximising water use by agricultural producers which are primarily constrained by both water availability and quality. Crop yields are influenced by quantitative irrigation water application deficits and by the salinity of irrigation water. Results show considerable differences depending on whether salinity is incorporated or not. When salinity is considered, yields tend to be much lower despite increased irrigation water needs to enable a reduction of soil salinity through leaching.
Keywords: nonlinear programming, water allocation, water quality, salinity.

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1 Introduction

The Drâa river basin is located in South-East Morocco at the edge to the Saharan desert. The area faces pre-Saharan climatic conditions with naturally low rainfalls. The precarious water situation has been aggravated by subsequent droughts and due to increasing salinity of both ground- and surface water in recent decades. Water salinity adversely affects the yet poor agricultural production potential. During the last years the salt content of irrigation water has further increased, [1] leading to very low agricultural output levels and the need for the farm households to identify additional sources of income. Hence, a holistic water management should take into consideration the impact of salinity on agricultural yields.

Since 1972 a centrally managed reservoir, the Mansour Eddahbi reservoir, supplies a belt of six oases along the middle Drâa River basin with irrigation water. Due to increasing surface water scarcity, farmers progressively established wells with motor pumps and are using groundwater instead of river water for irrigation. However, groundwater use has the drawback of very high salt contents especially in the two most southern oases, Ktaoua and Mhamid. The average values for salt content are shown in table 1. It should be noted that groundwater salinity is markedly higher than that of surface water.

Table 1: Salt content of ground and surface water in the Drâa basin.

Oasis	*Groundwater (g/l)*	*River water (g/l)*
Mezguita	1.5	0.64
Tinzouline	2.2	0.79
Ternata	2.5	1.04
Fezouata	4.0	1.04
Ktaoua	5.0	1.32
Mhamid	5.0	1.32

Source: Bouidida, A. 1990, Ministère du Commerce, de l'Industrie, des Mines et de la Marine Marchande, 1977.

For the Drâa basin irrigation water salinity is tremendously high (locally sometimes up to 10 milliohms/cm in the South), but so far seems not to have been sufficiently considered in water management in the region.

Water quality, especially salinity, has been addressed in various simulation models dealing with irrigated agriculture. Lee and Howitt [2] use a Coob-Douglas production function according to Dinar and Letey [3] to account for water salinity in a nonlinear programming model. Also, Cai et al. [4] use a production function taking the water deficit, salinity rates, and technology levels for yield formation into account. By contrast, the integration of water quality aspects in the hydro-economic model MIVAD (Modèle integrée du Valée du Drâa) presented in this article is formulated as a yield function containing factors reflecting both seasonal water deficits and salinity levels.

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2 The role of salinity in the Drâa basin model

The hydro-economic river basin model MIVAD is a nonlinear water allocation model that consists of a node-link network representing the six oases along the Drâa River. MIVAD is similar to the class of river basin models as designed by Rosegrant et al. [5]. The spatial structure of the model is presented in figure 2 where the interconnection between supply and demand is represented with arrows. The objective of the model is to maximize agricultural profits taking into account various constraints and balances. In MIVAD, farmers can make choices in cropping on two levels: the absolute area to be cultivated with a certain crop mix that is kept constant, and the yield levels for the different crops which depend on water application of different quantity and quality (i.e. salt content).

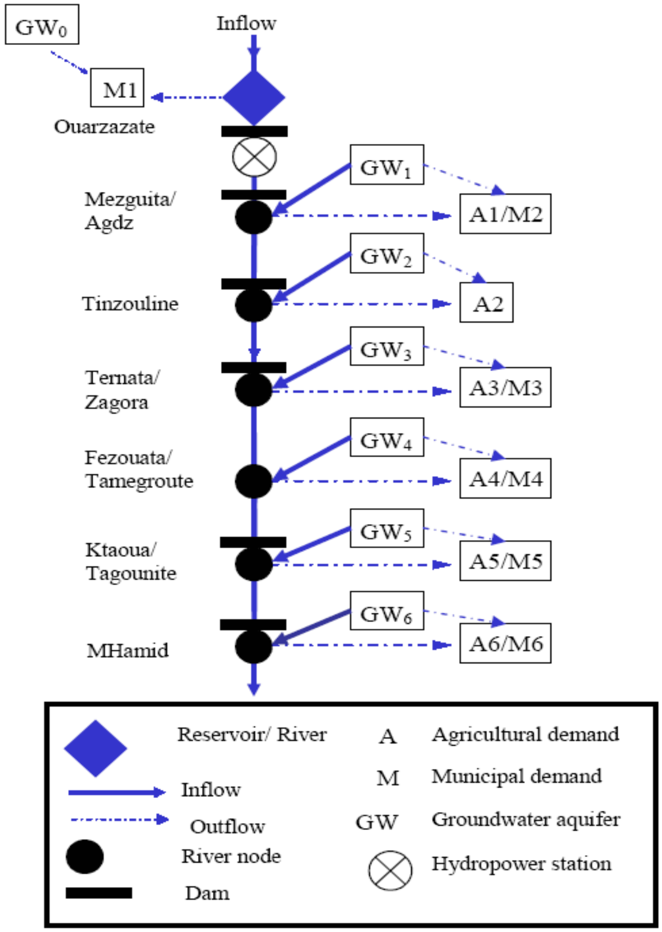

Figure 1: Spatial structure of the MIVAD model (Source: Kuhn et al. [6]).

More precisely, actual yields are calculated by reducing the maximum yield of a crop by a water deficit factor and a salinity reduction factor. This has been applied by Dinar and Letey [3] to a seasonal crop water production model. It is assumed that there is a maximum crop yield *pmaxyield* to be achieved with average technology (seed variety, fertiliser use, chemicals, seedbed preparation etc.). The actual yield in a certain year may be lower than the maximum due to insufficient water supply to the crop and salinity response. The yield function is based on the following relation:

$$vcropyiel_{dma,crop} = pmaxyield_{crop} \cdot vdef\_maxi_{dma,crop} \cdot vyie\_sali_{dma,crop} \qquad (1)$$

with *pmaxyield*, maximum yield for the different crops (per ha), *vdef_maxi*, yield reduction factor due to periodically or generally, insufficient water application (crop water deficit), *vyie_sali,* yield reduction factor due to salinity.

In MIVAD it is assumed that water application to crops is a decision that is made by farmers for the entire cropping season based on a-priori information on the amount of irrigation water available. The yield reduction factor due to crop water deficit (*vdef_maxi*) is calculated as a non-smooth approximation of the seasonal water deficit *vdef_seas*.

$$vdef\_maxi_{dma,crop} = \left(1 + \exp\left(\alpha \cdot \left(-vdef\_seas_{dma,crop} + \beta\right)\right)\right)^{-1} \qquad (2)$$

with *vdef_seas* being the seasonal water deficit as calculated by using seasonal ky-values (FAO 1986 [7]), α a slope coefficient of the approximation curve, and β a coefficient determining the position of the curve.

Monthly evapotranspiration consists of two components: total irrigation water applied to a crop (*v__w_a_cr*, which the farmer can choose to take from surface or groundwater sources) reduced by a leaching factor (to be explained later on), and the effective rainfall in the area.

$$\begin{aligned} & veta\_stag_{dma,crop,pd} = \\ & v\_\_w\_a\_cr_{dma,crop,pd} \cdot vleachfct_{dma,crop} \\ & \quad + vcroparea_{dma,crop} \cdot peff\_rain_{dma,pd} \end{aligned} \qquad (3)$$

with *v__w_a_cr i*rrigation water available to a crop both from surface water and groundwater, *vleachfct* leaching factor, *peff_rain* effective rainfall in mm, where the leaching factor (see formula 8) is calculated according to Ayers and Westcot [8] as:

$$vleachfct_{dma,crop} = 0.01 \cdot \exp\ (\delta \cdot vet\_ratio_{dma,crop}) + pirr\_effy \qquad (4)$$

with *vet_ratio* actual evapotranspiration (ETa) divided by maximum evapotranspiration (ETm), *pirr_effy* irrigation efficiency factor (constant).

The leaching factor not only determines the amount of irrigation water which percolates into deeper soil layers, but also plays an important role for the level of soil salinity. Salt concentration in the soil is a result of the fact that evaporation of irrigation water leads to an accumulation of salt in the topsoil. This is especially the case in the most southern oases, Ktaoua and Mhamid, as evapotranspiration in the area is high and insufficient leaching leads to an accumulation of salt on the surface. During and after irrigation days, leaching into deeper soil layers might occur and help to keep soil salinity in check, while during the rest of the time plants may still suffer from irrigation water deficit. This is why the leaching factor used in MIVAD contains a constant additive component (*pirr_effy*) reflecting the leaching losses of furrow irrigation.

The salt content of water consumed by crops (salinity) is another important factor for yield formation. The yield reduction factor due to salinity is calculated on the basis of a modified discount function (Steppuhn et al. [9]). The salinity of soil water (*vyie_sali*) is calculated as:

$$vyie\_sali_{dma,crop} = \left(1 + \left(vsoilsali_{dma,crop} \big/ psal\_thre_{crop}{}^{psal\_slop_{crop}}\right)\right)^{-1} \qquad (5)$$

with *vsoilsali* being the salinity level of the soil water consumed by crops, *psal_thre* the crop-specific salinity level at which the yield is depressed by 50%, and *psal_slop* a slope parameter. The effect of the slope parameter is displayed in figure 3.

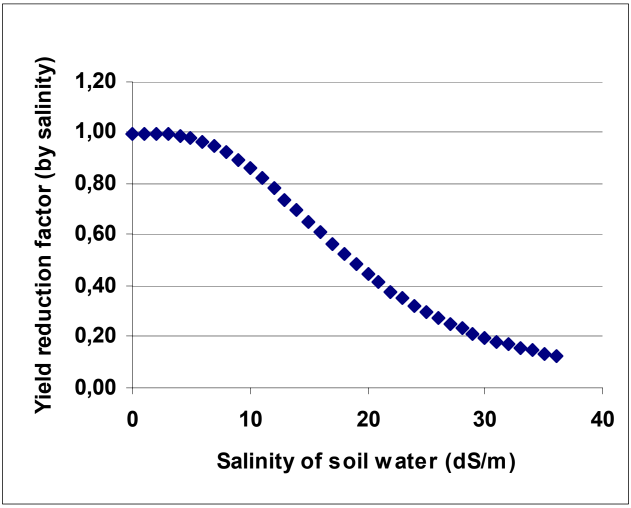

Figure 2: Effect of salt reduction factor on yields.

The soil water salinity level can be derived from the salinity level of the irrigation water multiplied by a concentration factor specific for each crop and oasis.

$$vsoilsali_{dma,crop} = vsalinity_{dma} \cdot vcon\_fact_{dma,crop} \tag{6}$$

with *vsalinity* being the salinity level of irrigation water and *vcon_fact* the concentration factor. Salinity of irrigation water is the average of the salinity levels of surface (= river) and groundwater used for irrigation, respectively.

The concentration factor describes the ratio of salinity in irrigation water to the salinity of soil water and can be calculated as a function of the variable 'leaching factor' (*vleachfct*) that has already been mentioned in equation 5. On the basis of results from Ayers and Westcot [8], the leaching concentration factor VCON_FACT is calculated as:

$$vcon\_fact_{dma,crop} = \left(\beta \cdot vleachfct_{dma,crop}\right)^{-\rho} \tag{7}$$

with β a level parameter, and ρ a slope parameter .

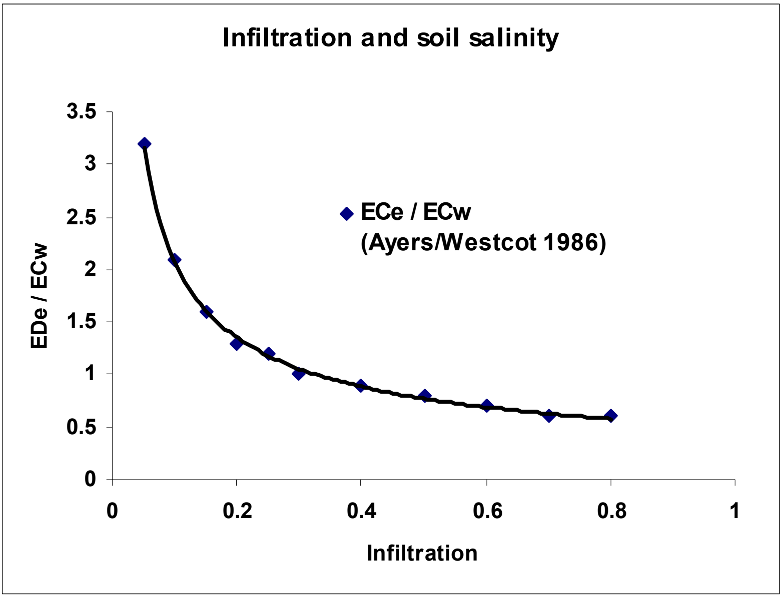

Figure 3: Soil salinity as a nonlinear function of the leaching factor.

3 Results of simulations involving salinity

To evaluate the effect of salinity on crop yields and agricultural profits, comparisons with and without a salinity effects on crop yields have been carried out for three different water supply scenarios. Table 2 summarises simulation results for different levels of surface water availability for the whole Drâa basin: a normal year, a medium and a dry year. The normal year relates to an average of inflows into the surface water network of the basin from 1972 to 2002, the dry year presents the average of the ten driest years over the same period (23% of the water amount of a normal year), and the intermediate year is an average of the other two (62%, respectively). If salinity is not considered, surface water for irrigation is more and more substituted by groundwater the more surface water becomes scarce. As total water use is nevertheless decreasing, agricultural profits are decreasing as well, primarily because the total cultivated area is decreasing, but also because yield levels are lowered, as is shown in table 3.

Table 2: Basin-wide simulation results for normal, medium and low water availability without and including salinity effects.

	Without salinity			*With salinity*		
	Normal	*Medium*	*Low*	*Normal*	*Medium*	*Low*
Ag. river water use (mio cbm)	165.3	89.4	16.9	188.8	118.4	17.9
Ag. groundwater use (mio cbm)	63.3	92.0	76.7	23.9	14.0	6.5
Total ag. water use (mio cbm)	228.6	181.4	93.5	212.7	132.4	24.4
Total water use (mio cbm)	233.8	186.6	98.7	218.0	137.6	29.6
Use of available crop area (%)	63.9	50.7	26.0	47.7	32.0	6.1
Agric. profits total (mio DH)	260.4	189.6	79.6	171.0	119.4	20.5

Results look completely different when the yield-decreasing effect of salinity is considered. As surface water is free of charge for farmers and less saline than groundwater, surface water is strongly preferred for irrigation of agricultural crops. However, when surface water becomes scarcer, for example in the intermediate and dry water supply scenarios, it would be increasingly substituted by groundwater, even though groundwater pumping is costly for the farmers. This is not the case when salinity is considered: groundwater is by far not used as extensively particularly in the dry year due to the fact that its use would not contribute to keep yields per hectare at profitable levels. This ultimately leads to a far more pronounced decrease in crop areas to only 6% of the maximum area available to farmers.

When water scarcity alone is taken into account, farmers will probably decrease crop areas, but also crop yields to a minor extent to deal with the scarcity situation. But in a situation which combines water scarcity and high salinity of the water available, farmers face a more complicated dilemma, as a reduction of the amount of irrigation water per hectare as in the scenario without salinity would swiftly increase soil salinity and depress yields by far more. The reason this is that the leaching effect of irrigation would decrease by more than the pure water reduction, an effect which is explained by the non-linear relation between water application and leaching as shown above.

A closer look at the individual effects of water scarcity and salinity reveals that salinity effects are indeed much higher than the impact of water scarcity (see tables 3 and 4). The scarcer the water gets, the more intense are the effects of salinity, as more groundwater is used, and as leaching to keep soil salinity down becomes more expensive. It is no surprise that crops that have both a high drought and salinity tolerance (see table 4, first column) such as wheat, barley or date palms suffer the smallest yield reduction effects as compared to the scenario without salinity (see table 3).

Table 3: Yield levels (in % of maximum yield levels) for normal, medium and low water availability.

	Without salinity effects			*With salinity effects*		
	Normal	*Medium*	*Low*	*Normal*	*Medium*	*Low*
Wheat	95.9	94.1	92.5	96.2	92.7	91.6
Barley	82.5	68.2	66.3	87.6	74.4	74.9
Pulses	97.9	97.2	95.3	91.0	83.3	75.5
Vegetables	98.5	99.1	99.7	58.8	64.4	69.2
Henna	80.2	85.4	86.1	67.3	72.8	72.3
Date palms	77.5	82.2	83.9	70.5	75.6	77.5
Alfalfa	71.1	77.3	78.9	37.8	39.5	38.7

Table 4 decomposes the yield reduction effect under salinity into the water deficit and the salinity effect which together constitute the yield function (see equation (1)). Moreover, the sensitivity of the different crops with respect to water deficit and salinity as used in the model are reported in the first column. Water needs of crops (and implicitly the sensitivity to water stress) are expressed as the evapotranspiration at the maximum yield level under local climate conditions in millimetres per annum. The higher the water need of a crop, the higher the crop is assumed to be prone to water stress. The sensitivity regarding salinity is expressed as an index calculated by dividing the level parameter *psal_thre* by the slope parameter *psal_slop* (see equation (5)). The lower the index, the more sensitive is the crop to the salt content in the soil water.

Table 4 shows that for most crops yield reduction originates from salinity (the reduction factors are much smaller) and not from the 'pure' irrigation water deficit. Moreover, it is difficult to predict the yield reduction on the basis of the sensitivity to water stress and salinity alone. The profitability of crops might still justify a high water input level, which is exemplified by vegetables: the overall salt content of irrigation water does hardly allow yield levels above 70% of the maximum yield. Nevertheless, vegetables are heavily leached in order to allow reasonable yields. Alfalfa yields, by contrast, are allowed to drop, as this crop generates less profit than vegetables.

Table 4: Decomposing the yield reduction under salinity into a water deficit and a salinity effect (figures denote the share of the maximum yield).

	Sensitivity of crops to yield-reducing factors	*Normal*	*Medium*	*Low*
Water deficit effect	*Max. water need*			
Wheat	513	1.00	0.99	0.97
Barley	509	0.96	0.88	0.83
Pulses	431	1.00	1.00	1.00
Vegetables	659	1.00	1.00	1.00
Henna	1848	0.90	0.92	0.93
Date palms	1786	0.83	0.87	0.88
Alfalfa	1848	0.77	0.76	0.76
All crops		0.92	0.92	0.91
Salinity effect	*Salinity tolerance*			
Wheat	6.35	0.96	0.94	0.95
Barley	4.61	0.95	0.93	0.95
Pulses	1.80	0.89	0.82	0.78
Vegetables	2.03	0.57	0.63	0.70
Henna	3.78	0.72	0.79	0.77
Date palms	6.70	0.85	0.87	0.88
Alfalfa	3.30	0.50	0.51	0.51
All crops		0.78	0.78	0.79

4 Discussion

Accounting for salinity in yield formation and production models has enormous effects on simulation results regarding resource use, which is highly relevant for basin-wide water management decisions. Most importantly, the on-farm effects (water use from different sources, cropping choice, yield levels) become more difficult to predict when salinity comes into play. The decision situation facing the farmers is indeed highly complex, even when simulated in a deterministic setting with perfect foresight as in this article. Moreover, if the salinity of irrigation water were to further increase in the coming years, the trend towards using groundwater for irrigation could perhaps be reversed. This effect could be demonstrated in more detail by employing a salt flow balance, which so far has not been addressed due to a lack of empirical data. As to resource management aspects, both groundwater availability and salinity should be considered when deciding on the optimal allocation and distribution of surface water among the oases, as far as it this in the domain of a central water distribution agency.

Furthermore, the cropping mix cultivated is likely to shift to more salt-resistant crops with increasing salinity. The model version on which the results in this article are based is keeping the crop mix fixed and only adapts total cropping area and crop yields. A suitable calibration method allowing for a more flexible cropping mix needs to be further refined.

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References

[1] ORMVAO (1996): Etude de salinité de sols dans la vallée du Drâa, Province de Ouarzazate. Final Report. Vol 1.
[2] Lee, D. J, Howitt, R.E. (1996): Modeling regional agricultural production and salinity control alternatives for water quality policy analysis. American Journal of Agricultural Economics. 78: 41-53.
[3] Dinar, A., Letey, J. (1996): Modeling Economic Management and policy issues of water in irrigated agriculture. Westport: Praeger Publishers
[4] Cai, X, Rosegrant, M.W., Ringler, C. (2006): Modeling Water Resources Management at the Basin Level: Methodology and Application to the Maipo River basin. Research Report No. 149. International Food Policy Research Institute. Washington D.C.
[5] Rosegrant, M.W. et al (2000): Integrated economic-hydrologic water modelling at the basin scale: the Maipo river basin, Discussion Paper No. 63. International Food Policy Research Institute. Washington, D.C.
[6] Kuhn, A. Schmidt, T., Heidecke, C. (2005): Economic Aspects of Water Management in the Drâa Region (Southeast Morocco). Deutscher Tropentag 2005: The Global Food and Product Chain - Dynamics, Innovations, Conflicts, Strategies University of Hohenheim, Stuttgart, October 11 - 13.
[7] FAO (1986): FAO Irrigation and Drainage Paper 33: Yield response to water, Rome.
[8] Ayers, R.S. Westcot, D.W. (1985): Water quality for agriculture. FAO Irrigation and Drainage Paper No. 29.
[9] Stepphuhn, H., van Genuchten, M.Th. Grieve, C.M. (2005): Root-zone salinity II. Indices for Tolerance in Agricultural Cops. Crop Science 45. 221-232.

Low-pressure drip system in reduced tillage cotton

B. Marsh[1], M. Dowgert[2], R. Hutmacher[1] & C. Phene[3]
[1]*University of California Shafter Research and Extension Center, Shafter, CA, USA*
[2]*Netafim, USA, Fresno, CA, USA*
[3]*SDI+, Fresno, CA, USA*

Abstract

Research has shown the soil and water conservation advantages of subsurface drip irrigation. Low-pressure drip system (LPS) technology has shown a high potential for economically improving application efficiency of irrigation systems under sandy soil conditions in areas where water is scarce and/or expensive. Energy costs are reduced as less than 70 grams per square cm is needed for system operation. The low-pressure system is installed just below the soil surface, it operates at very low flow and pressure, and it can stay on for longer periods of time without generating runoff or deep percolation. This study is designed to assess LPS under a reduced tillage system without the use of any other irrigation method for stand establishment. This combines the benefits of increased water use efficiency and lower energy costs for improved irrigation efficiency and fewer tillage operations resulting in lower production costs and less airborne dust. Since the drip tape was installed two years ago, only 3 cultivation passes have been made. No major tillage operations, the kind that generate lots of dust, have been performed. LPS water usage was 15% less than furrow irrigation and yields of cotton (*Gossypium hirsutum*) and blackeye beans (*Vigna unguiculata*) were comparable to yields from furrow irrigation. This system does present some challenges in stand establishment on very sandy soils and with weed control, which continue to be investigated. Herbicides requiring incorporation were not used. Weeds can be controlled in cotton using glyphosate and other herbicides. Fewer chemical weed control options are available for blackeye beans. The LPS technology has many potential technical, energy and economic advantages over standard drip and subsurface drip irrigation.
Keywords: cotton, low-pressure drip irrigation.

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1 Introduction

Recently, water, energy, fertilizer, pesticides, labor cost and the capital investment in modern irrigation systems have risen dramatically and at a rate greater than farmer returns. Studies have demonstrated that drip irrigation can improve water use efficiency, reduce fertilizer losses and reduce application of pesticides and fungicides, particularly when compared with flood, furrow and sprinkler irrigation [1–7]. As drip irrigation knowledge has evolved, Netafim Irrigation has developed Low Pressure Systems (LPS) that operate at 70 grams per square cm pressure while achieving a distribution uniformity of 90% or better. The conversion of leveled furrow irrigated fields to LPS using pressurized district water eliminates additional energy expenditures. It also conserves significant water and energy and allows the use of low-pressure components, thus reducing the capital inputs of LPS. Soil moisture wetting patterns and resultant rooting patterns are affected by drip irrigation frequency and amount of water applied [8].

The objective of this project was to evaluate the use of LPS drip in a reduced tillage cotton system with the shallow buried tape remaining intact for three years with no other irrigation method used to germinate the seed. Research results will be used to validate LPS irrigation design and management, and to demonstrate on-farm water, energy, chemigation, and labor savings in a reduced till system.

2 Materials and methods

This project consists of two low-pressure irrigation treatments on undisturbed seedbeds replicated four times in a randomized block design. The drip system operates at 70 grams per square cm. Each system delivers approximately an equal amount of water on an area basis. The treatments are LPS-200: 2 drip lines on 200 cm beds (75 cm lateral spacing, 60 cm emitter spacing, 100 cm row spacing) and LPS-150: 1 drip line on 150 cm beds (45 cm emitter spacing, 75 cm row spacing). The row spacing represents typical bed configuration for multiple rotation crops.

Drip lines were installed 10 cm below the soil surface in the spring of 2005. Bed shaping in the early spring and planting have been the only tillage operations since the tape was installed. Each plot was 8 beds wide by 100 m. The plots were on a Wasco sandy loam (coarse-loamy, mixed, nonacid, thermic Typic Torriorthent) soil.

Cotton (*Gossypium hirsutum*) was grown in 2005 and blackeye bean (*Vigna unguiculata*) was grown in 2006. N-P-K fertilizers were added to meet the crop requirement and were injected in the irrigation water as needed to maintain optimal petiole tissue levels (measured weekly). Acid (N-pHURIC, 10/55) was injected in all LPS irrigation water to maintain the solution pH at 6.5+/-0.04. Plots were mechanically harvested.

Soil moisture sensors were installed at 15, 40 and 60 cm deep in one row of each treatment. In-season irrigation was determined by calculating crop evapotranspiration (ETc), using on-site CIMIS weather station measurements (ETo) and a generic crop coefficient for this area (Kc), where ETc = ETo x Kc. Feedback from the rate of change of soil moisture measurements was used to adjust irrigation schedules.

Uniformity testing was conducted on a single row in each replication. Emitter output was measured every 10 m down the row. A hole is dug deep enough to uncover the drip tape and place a small cup under the emitter to collect water. The drip tape was cleaned off and a piece of black electrical tape was wrapped around the drip tape about 3 cm from each side of the emitter. The tape prevents water from travelling down the drip tube. A small cup is placed underneath the emitter to collect water for fifteen minutes. The water collected is measured in a 100-mL graduated cylinder and recorded.

Soil and root samples were collected in a grid pattern from each plot. A 4 cm diameter tube was driven 3 cm deep into the wall of a pit and extracted. Soil samples were weighed and dried for moisture content. Root samples were washed free of soil and measured for length.

3 Results

Emitter uniformity across the field was very good exceeding 90% combined over years, see Figure 1. In 2005 uniformity was 94%, however it dropped to just less than 10% in 2006. System water pressure was less in 2006, at 55 grams per square cm. This was due to filtration problems and was lower than desirable for optimum system operation although emitter uniformity was still very good. Multiple field and laboratory studies show similar results in tape that was either unused or having been buried for up to eight years [9–11]. It is predicted that the system could remain in place for extended years without a reduction in performance.

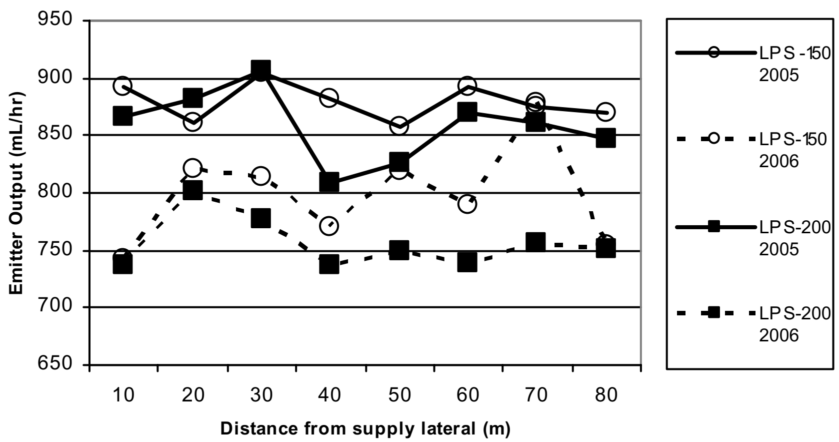

Figure 1: Emitter output.

Control of irrigation water was easily accomplished using a combination of predicted ET, a changing crop coefficient according to plant growth stage and soil moisture feedback. To accomplish the objective of not using any other irrigation method to germinate the seed, the soil was well wetted prior to planting, see Figure 2. Tops of the bed were removed to plant into moisture. Plant population was not significantly different between the treatments and was within the acceptable range for the Southern San Joaquin Valley [12]. Soil moisture was allowed to dry down following planting and as the plants developed through mid-May with only one irrigation needed in early May. Irrigation duration increased and thus soil moisture also increased in mid to late May to stimulate plant growth. Irrigation management then utilized deficit water status reducing soil moisture from mid June to August to control vegetative plant growth before increasing soil moisture during the critical period of boll development. There was not an excess delivery of water as moisture readings at 60 cm remained fairly constant. Root growth and water uptake was minimal at that depth.

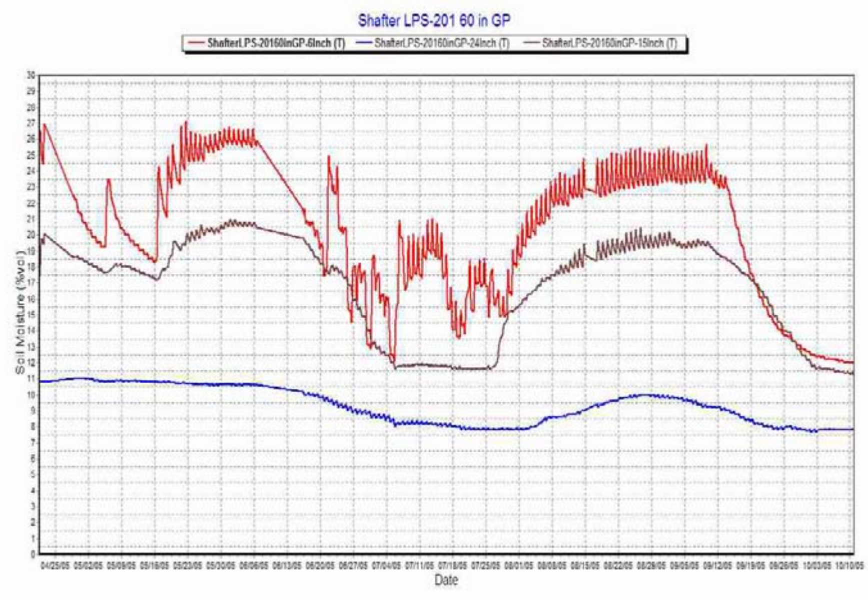

Figure 2: Soil moisture, LPS-150.

Plants became infected with fusarium wilt (*Fusarium oxysporum*) which had a limiting effect on plant growth and yield. The amount of water delivered was appropriate for the size of plants in the field. Water delivered generally remained directly below the drip lines and did not move below 50 cm deep. There was very little lateral water movement from the drip lines. As approximately equal amounts of water were delivered in each treatment, soil moisture around the single drip line was about twice the level of soil moisture around the two drip lines. In either case sufficient moisture was available for the desired growth pattern for cotton and blackeye beans.

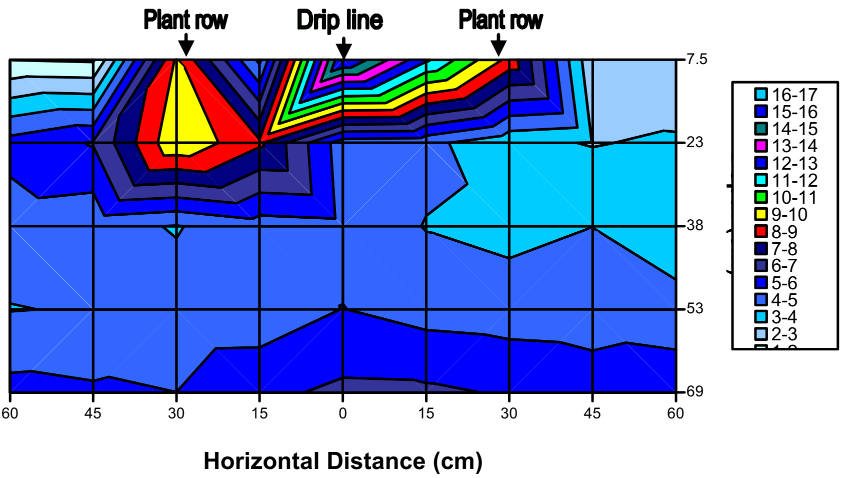

Figure 3: Soil moisture content, LPS-150.

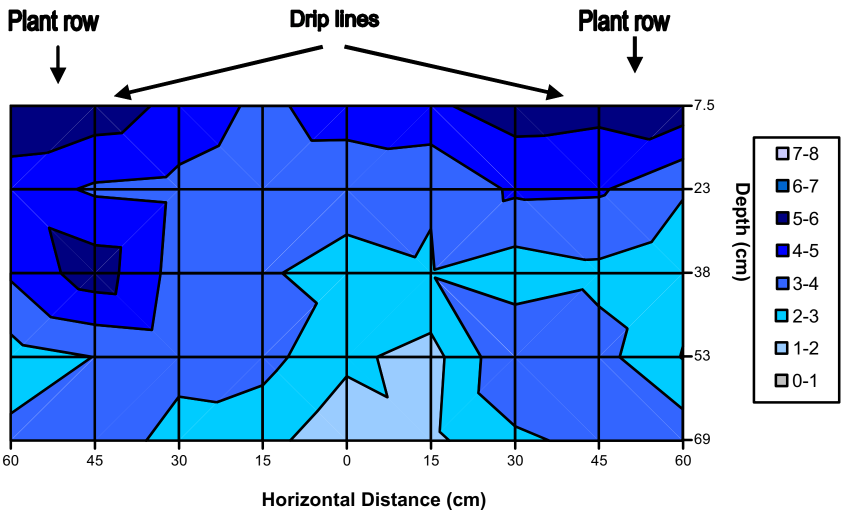

Figure 4: Soil moisture content, LPS-200.

Root growth responded to where soil moisture was and was generally confined to the upper 40 cm. In the LPS-200 treatment, where the drip lines are 10 cm from the plant row, roots grew only in that area. In the LPS-150 treatment, where only one drip line is placed between the rows, root length density was greater toward the drip line than at depth directly below the plant row. Root length density within the wetted zones was sufficient for uptake of all available water which was ample for desired crop growth. It exceeded the 0.3 to 1.0 cm cm^{-3} requirement put forward by van Noordwijk [13]. However with the limited area of water availability and root exploration, supplemental fertilization was required. Cotton and blackeye bean yields were not significantly different between the treatments and similar to furrow irrigated yields.

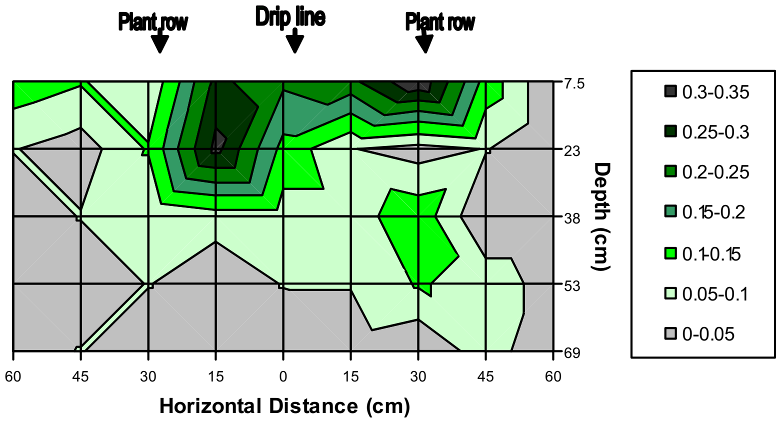

Figure 5: Root length density, LPS-150.

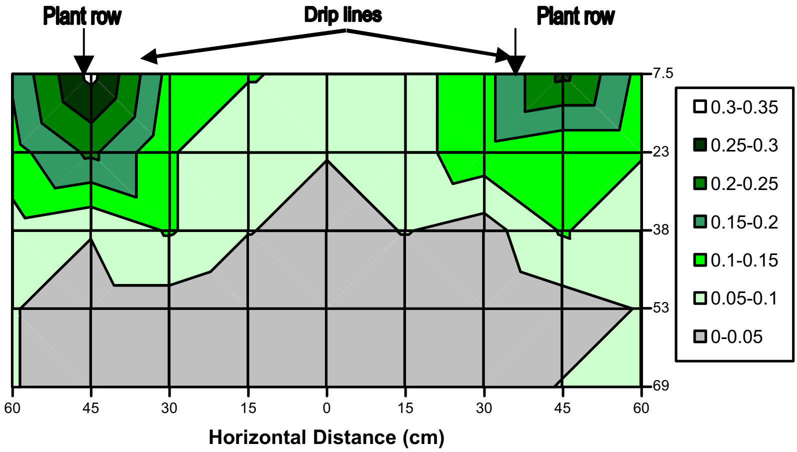

Figure 6: Root length density, LPS-200.

Table 1: Agronomic responses to drip tape configuration.

Treatment	2005 Cotton Lint Yield (kg ha^{-1})	Plant Population (# ha^{-1})	Applied Water (cm)
LPS 200	940	115,000	58.9
LPS 150	1080	110,000	60.7
	ns	ns	
	2006 Blackeye Bean Yield (kg ha^{-1})	Plant Population (# ha^{-1})	Applied Water (cm)
LPS 200	2576	147,200	69.6
LPS 150	2601	148,100	72.1
	ns	ns	ns

4 Conclusions

The combination of a low pressure drip system and reduced tillage was effective in reducing water usage, energy requirements and fugitive dust while maintaining comparable yields with furrow irrigation. Extra attention to irrigation management, to sufficiently wet the soil where seeds were to be placed, and removing the tops of the bed were required for good seed germination. This was an issue with this project because of the limited lateral movement of water in the sandy soil. The drip system had good water delivery uniformity throughout the field. The durability of the drip tape will allow it to remain in place for several years. This irrigation system provides water, energy, chemigation, and labor savings in a reduced till system.

References

[1] Bucks, D.A., L.J. Erie, and O.F. French. Quantity and frequency of trickle and furrow irrigation for efficient cabbage production. Agronomy Journal 66:53-57, 1974.

[2] Camp, C.R., Thomas, W. M., and Green, C. C., Micro irrigation scheduling and tube placement for cotton in the southeastern coastal plain. Transactions of the ASAE 36:1073-1078, 1993.

[3] Hanson, B.R., Schwanki, L. J., Schulbach, K. F. and Pettygrove, G. S., A comparison of furrow, surface drip, and subsurface drip irrigation on lettuce yield and applied water. Agricultural Water Management 33:139-157.

[4] Hodgson, A. S., Constable, G. A., Duddy, G. R., and Daniels, I. G., A comparison of drip and furrow irrigated cotton on a cracking clay soil. Water use efficiency, waterlogging, root distribution and soil structure. Irrigation Science 11:143-148, 1997.

[5] Lamm, M. C., Davidson Jr., J. I., and Pitts D. J., Revision of EP-458: field evaluation of microirrigation systems. ASAE Paper No. 972. St. Joseph, Mich. 1997.
[6] Ottis, B., Henggeler, C. and Vories, E., Low-pressure, Drip-irrigation for Rice. American Society of Agronomy, 2006.
[7] Sammis, T. Comparison of sprinkler, trickle, subsurface and furrow irrigation methods for row crops. Agronomy Journal 72:701-704, 1980.
[8] Kang, Y, Wang, F., Ping, S., Effects of Drip Irrigation Frequency on soil Wetting Pattern and Root Distribution of Potato in North China Plain. ASABE Annual Meeting 2002.
[9] Camp, C. R., Sadler, E. J., and Busscher, W. J. A comparison of uniformity measures for drip irrigation systems. Trans. ASAE. Vol. 40. pp. 1013-1020. 1997.
[10] Hanson, B. R., Fipps, G., Martin, E. C. Drip irrigation of row crops: What is the state of the art? www.oznet.k-state.edu/sdi/Abstracts/Drip Irrigation of Row Crops.htm
[11] Weynand, V. L. Evaluation of the application uniformity of subsurface drip distribution systems. M.S. thesis. Texas A&M University. 2004.
[12] Johnson Hake, S, Hake, K. D, and Kerby, T. A., Planting and Stand Establishment (Chapter 4). *Cotton Production Manual*, eds. S. Johnson Hake, T. A. Kerby, K. D. Hake, University of California, pp 21-28, 1996.
[13] van Noordwijk, M., Functional Interpretation of Root Length Densities in the Field for Nutrient and Water Uptake. Root Ecology and its Practical Application. Int. Symp. Gumpenstein, A-8952, pp. 207-226. 1982.

Drought risk management in the Mediterranean under the Water Framework Directive – the example of Algarve (Portugal)

A. Do Ó
e-GEO, Faculdade de Ciências Sociais e Humanas, Universidade Nova de Lisboa, Portugal

Abstract

Recent experience and developments on drought knowledge are encouraging modern societies to shift from a traditional crisis-based management to a risk management approach. In southern Portugal, new EU legislation (and namely the Water Framework Directive) has set the framework for several policies and juridical tools covering drought and water scarcity issues, now driven by the principles of integrated and sustainable use of water resources.

This paper aims to assess the role and effectiveness of all major legal tools covering drought risk management and drought impact mitigation, as well as their level of integration and coordination. The tools currently enforced in the region that were thoroughly analysed were the National Water Plan, two River Basin Plans (Guadiana River and Algarve Streams), the 1998 Portuguese-Spanish Convention on the use of shared waters, the ad-hoc Commission for Drought 2005, and several activity regulations, namely on domestic water supply and agricultural irrigation.

Results point out the dispersion and lack of coordination within this wide range of legal instruments, especially under scarcity conditions as occurred in 2004 and 2005, calling for improved linkages between such tools under an integrated regional drought plan. Such plan should be consistent with the Water Framework Directive, integrating resources, policies and institutions, and elaborated in close collaboration with the neighbouring Iberian water regions.
Keywords: drought plans, risk management, mitigation policies, Algarve.

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1 Introduction

Recent experience and developments on drought knowledge are encouraging modern societies to shift from a traditional crisis-based management to a risk management approach (Figure 1).

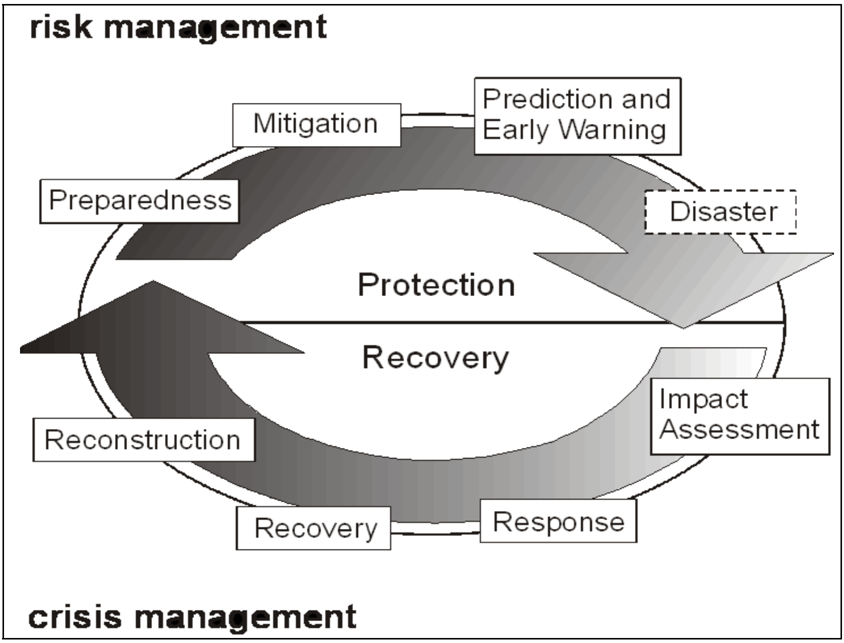

Figure 1: Crisis vs. risk management (Wilhite [1]).

This new approach is focused on preventive planning and pro-active measures, rather than reactive actions which are usually taken after the event and its impacts are already onset. Such approach calls for an integration of policies affecting water management and scarcity issues, as drought impacts are scattered among different water uses and different time and space scales.

The risk management approach to drought has been developed and implemented mainly in the USA, Australia, and South Africa, and only recently became an issue in Europe. Nevertheless, Southern Portugal, with its fully Mediterranean climate, landscape and culture, has long been facing drought impacts as a crucial component of the regional interface between society and environment. Recent severe events (1980-1983, 1991-1995, 2004-2006) with increasing impacts at supra-national scales (namely southern Iberian and western Mediterranean), have strengthened the need for targeted policies and actions, as well as for a common Mediterranean and European strategy on water scarcity and drought issues.

This paper aims to assess the role and effectiveness of all major legal tools presently covering drought risk management and drought impact mitigation, as well as their level of integration and coordination.

2 European policy context and regional legal framework

In Southern Portugal, new European Union (EU) legislation (and namely the Water Framework Directive, WFD, EC [2]) has set the framework for several policies and juridical tools covering drought and water scarcity issues, increasingly driven by the principles of integrated and sustainable use of water resources.

The WFD has no specific article on drought issues, and only makes reference to "prolonged droughts" on Article 4, while defining the exceptionality regime in face of the Directive objectives, and on Article 11, as a condition that may call for supplementary measures of water demand management. Nevertheless, as it aims at reaching a good state of European waters by 2015, both in quality and quantity, it does provide some cover to drought related policies and other legal tools that might be conceived.

The growing severity of economic, social and environmental impacts during recent drought events has provided the grounds for increasing pressure on the EU to promote a common European drought policy (EEA [3], EurAqua [4], WWF [5]). The EU has responded to these signs of public concern, and significant political action is being developed as the European Commission (EC) is preparing a policy development on Water Scarcity and Droughts (WS and D), which may lead to a Communication being issued by mid-2007.

This has been considered by political analysts as the Iberian counterweight (or even compensation) to the flood policy recently introduced, which was promoted by central and northern European countries (and namely Germany) and soon will be approved as an EU Directive, in close relation with the WFD, also called "mother-directive".

Behind the political scene, technical work has been prepared. Early actions were taken in 2003 by the Member States Water Directors, by creating an Expert Group on WS and D. Based on its work, and pressed by the severe Iberian event of 2004 and 2005 (also affecting large parts of France and Italy severely), a number of Member States informally led by Portugal and Spain requested to initiate a European Action on WS and D, during the Environment Council of March 2006. The EC agreed to analyse this request, and to present a first report to the Environment Council in June 2006. At that stage, the EC presented a first analysis based on available data, and proposed to strengthen the diagnosis and plan for further actions to be taken at the EU level. In parallel, a Mediterranean Working Group, set up in the framework of the MED-EU Water Initiative, was in charge of producing a specific report on Mediterranean specificities and examples in the region.

The technical work of the Expert Group on WS and D can be divided in two main working modules:

a) An interim report of existing data on impacts of WS and D, which was discussed with the Water Directors in November 2006, and should be updated with new data by 2007;
b) The identification of pending issues (exemptions, drought management plans) dealing with the WFD implementation process, to be further analysed.

At the time of writing this article, no further information was available on this complex policy building process.

Although this European legal "umbrella" is only now being set on, Mediterranean countries such as Portugal have considerable experience dealing with drought events and its impacts. In most cases, this has only meant that each of the affected activities has developed its own contingency planning, but little or no integration policy or action was taken by public authorities.

In fact, there is no specific integrated drought policy or plan which is active in the Algarve, either at the European, national, or regional level. Drought management is scattered through different activity regulations, namely domestic water supply and irrigated agriculture, and ad-hoc emergency relief actions. All of these regulations are usually planned and set on at the national level, with little or no cross boundary integration, and very little attention to specific water basin or regional issues.

Nevertheless, several planning tools are enforced in the Algarve region, with references to drought and water scarcity management issues. These include the National Water Plan, two River Basin Plans (Guadiana River and Algarve Streams), the 1998 Portuguese-Spanish Convention on the use of shared waters (Albufeira Convention), the Commission for Drought 2005, and specific regulations on domestic water supply and agricultural irrigation, all of which were thoroughly analysed in terms of its scope and effectiveness on drought impact mitigation.

3 Permanent drought related policies and plans

The key legal instrument for water issues in Portugal is the Water Law, which was only recently approved (Law 58/2005), transposing the contents of the WFD to the national legal framework. The law itself has more of a guidance scope than a regulatory one, which is only given by specific legislation still being produced (such as the economic and financial regime, the property and public domain regulation, and others).

In this context, it is understandable that references to drought and drought impact mitigation are scarce and rather vague. Nevertheless, the Water Law refers specifically to the protection of society against drought effects on its article 41, through eventual "drought intervention programs" which should state the goals to be achieved, specific measures to be adopted by each of the economic activities affected, and description of its implementation mechanisms. Such measures should specify any changes or limitations foreseen to regular uses and procedures, such as water pressure in supply systems, or water prices. Another positive aspect is that general priority in water uses is clearly defined:

first is domestic supply, and secondly vital activities within agriculture (permanent crops) and industry (energy production and infrastructure maintenance), with the remnant not being listed.

More importantly, the Water Law defines the planning structure and juridical context for water resources management, comprising therein the National Water Plan (approved by DL 112/2002) and the Water Basin Plans (WBPs).

The National Water Plan is mostly a sum of the 15 WBPs composing the Portuguese continental territory, and the 2 Regional Water Plans referring to the archipelagos of Azores and Madeira. It sets the conceptual framework for drought definition and drought impacts, and features a national survey of drought vulnerability based on a supply-demand balance. It concludes that agriculture and domestic water supply became increasingly vulnerable over the last couple of decades, especially in the southern regions, and proposes a “Drought Effects Mitigation Plan”, and a set of measures designed to ensure 80% of the water demand for agriculture, 95% for livestock, and 100% for domestic households (Program 6, measures 1 and 2). These measures also include increasing the efficiency of water use, and the reduction of losses in supply systems (Program 7, measures 1 and 2).

But the practical implementation of such programs and measures is dependent on a national budget capacity, which has been extremely limited over the last few years, as well as its inclusion in the respective WBPs, which are the cornerstone for effective measures and actions to be taken. In the case of the Algarve, the region is split between two Plans:

a) the Guadiana Basin Plan (approved by DR 16/2001), shared with the northern neighbouring region of Alentejo, while the basin itself is also shared with Spain, where 80% of the total basin area lies, although no common Plan is legally active;
b) the Algarve Streams Basin Plan (approved by DR 12/2002), which includes most of the region, along a network of small temporary streams flowing within it, similar to conditions of an island system.

Both Plans were produced before the Water Law was approved, based on the former legal framework for water resources management (approved by DL 45/1994), but already including the guidelines defined under the Portuguese-Spanish Convention of 1998, discussed further ahead. The two WBPs include several components related to drought issues, such as a more detailed regional diagnosis of drought occurrence, and the framework for the set-up of an early warning system, and for the elaboration of multiple planning tools, such as intervention plans, contingency plans, emergency plans, impact mitigation and impact prevention plans.

This excessive number of proposed plans, coupled with some lack of objectivity, suggests that its applicability will be nearly impossible. In fact, none of them was elaborated, and when drought stroke in 2004, only ad-hoc and emergency actions were taken (as analysed in chapter 4 further ahead), with little or none input coming from WBPs. Such lack of effectiveness results from several major operational and policy design handicaps as follows:

a) The effort put on the elaboration of these Plans was mostly concentrated on the current status analysis, instead of defining adequate strategies and operating schemes;
b) Its ambiguous and broad scope, between the strategic political plan and the operational technical project, has resulted in the lack of interest, knowledge, participation and reconnaissance from public officers and end-users in general, which was a major cause for the inefficient linkage with other planning tools as well;
c) The main results achieved under these Plans were in the field of public sanitation, domestic sewage and water supply systems, which were foreseen under other planning tools (specifically PEASAAR, Strategic Plan for Water Supply and Wastewater Treatment), and received financing priority from EU funding;
d) Lack of follow-up, assessment and updating practices, as well as poor project implementation, is due to the absence of a permanent planning structure, including both human and financial resources;
e) The strict guidelines defined under the National Water Plan generated a set of WBPs far too uniform, regardless of the strong internal regional differentiation in terms of water resources and drought vulnerability;
f) Last, but possibly most important for the Algarve, these plans sanction groundwater resources as a complementary and emergency source, suppressing one of the primary principles of integrated water resources management, promoted under the WFD, of a joint strategic use of both surface and groundwater resources. This vision set the fundaments for the second largest national public investment ever made in the Algarve (after the motorway network): the domestic water supply system, exclusively based on surface resources, which was built in the late 1990s at a total cost of approximately 1.000M€, largely supported by the EU Cohesion Fund.

Nevertheless, some positive aspects should be pointed out as well, such as the quality and extent of the basic diagnosis, the increasing sensibility of decision-makers and end-users to the principles of sustainable and integrated management of water resources, and the definition of some critical framing regulations, such as the protection of aquifers in sensitive areas, and the use of wastewater on golf courses.

According to the timing enforced for the implementation of the WFD, this first generation of WBPs should be replaced until 2009 by a new generation of Plans called Water Region Management Plans, to be elaborated in full accordance with the principles and guidelines defined in the WFD and the Portuguese Water Law.

In the specific case of the Guadiana Basin Plan, it was already elaborated under the orientation and guidance provided by the so-called Albufeira Convention, which was signed in 1998, and defines the framework for cooperation between Portugal and Spain in what concerns the management of the common water basins, which cover most of the Iberian Peninsula.

The Convention establishes an annual flow regime, defining mandatory flow volumes in sections upstream of the border, for Spain, and on the respective estuary or mouth for Portugal. It includes an article (19) specifically on “drought and water scarcity”, covering only generic aspects. Therefore, a large part of the innovative regulations within the Convention are included in the flow regime mentioned above, which was the object of an additional Protocol to the Convention.

The Protocol defines, in its article 5, the flow regime to the Guadiana River, as well as the conditions for defining an exception regime, usually associated with drought periods. Although the Guadiana flow regime has little meaning in terms of regional water supply, the Convention holds the importance of its strategic thinking and political design. Significant examples can be found in the definition of priorities among economic activities (domestic supply, livestock, permanent crops, ecologic functions), bilateral compliance with European and international laws and regulations, set-up of permanent information exchange circuits, and the promotion of a sustainable and frugal use of water (Serra [6]), since any significant increase on water consumption results on increasing risk of non compliance with the flow regime defined.

The Convention has a technical follow-up unit (CADC, Commission for the Application and Development of the Convention), subdivided into four working groups, one of which is focused on “flows, droughts and emergency situations”. As the Convention is to be revised by the end of 2007, this group has the crucial task of adapting the annual flow regime to a monthly one, as well as redefining the criteria defining exception situations.

4 Emergency plans and tools during the drought of 2004-06

As this institutional and planning panoply is only recently being created and established, practical action in emergency situations still relies on ad-hoc reactive measures. This was clear during the severe drought event of 2004-06, when public authorities (in this case, the Government) created a “Drought Commission 2005”, and the response from the two major users, agriculture and domestic supply, was completely separated.

Irrigation is the main user of water in the Algarve (Table 1), concentrated in large public infrastructure schemes that rely on surface water from reservoirs, and on groundwater in countless individual schemes.

Table 1: Distribution of major water uses in the Algarve, Do Ó and Monteiro [7].

Activity	Agriculture	Domestic	Golf	Industry	Total
volume (hm^3)	230	70	10	9	319
% of total	*72*	*22*	*3*	*3*	*100*

These farmers seldom have any systematic monitoring or preventive procedure in face of droughts, and farmers simply seek for alternative sources which, in consequence, might become exposed to overexploitation. If these sources run dry, farmers can only reduce or even stop their activity, eventually with dramatic social and economic consequences.

Each of the large public irrigation schemes has its own users' regulation under scarcity conditions. However, the customary approach is just to distribute eventual supply reductions among users. In 2004, when water resources were already below average, no action was taken, but in 2005 two were forced to impose 30% cuts in supply, and the other one had cuts of over 90%, with available resources being used to maintain livestock and perennial crops only. The key issue is that these schemes only represent about 14% of the total water volume used for irrigation, and therefore such regulations have a relatively small impact on overall regional water balance during drought events. Thus, it remains quite unknown what are the real impacts and responses of drought on agriculture in the Algarve.

Domestic water supply is, since 2001, exclusively based on surface resources and managed by a semi-private company (Águas do Algarve, AdA), which was the first large user in the region to formally react to the increasingly intense drought event. By the end of summer 2004 AdA had drafted a Contingency Plan, reinforced in March 2005 after one of the driest winters ever recorded in the region (as in most of the Iberian Peninsula). It pointed, not surprisingly but in paradox with the system design, to the need to use groundwater resources in addition to those stored in reservoirs. To achieve this, emergency boreholes and pipelines had to be constructed, as the system had not planned for any other connections.

The Plan also suggested some long-term solutions to the chronic regional water-deficit, which have since been publicly discussed, such as desalination plants, water basin transfers, dam construction, water reuse, and others, but the discussion itself faded when average rains returned during the winter of 2006.

On a higher level of political decision, the national Government created (in March 2005 only) a Drought Commission that gathered most of the public authorities involved and, to a limited extent, some of the end-users. The Commission put considerable effort in producing regular information on the situation and its impacts, and launched a set of emergency response actions. Unfortunately, as the drought receded with the average rains of winter 2005-2006, the Commission was simply disbanded, and few plans or actions were made to respond more effectively to future events.

The last report, produced in March 2006, proposed significant contributions towards drought risk management and impact mitigation, such as:

a) Contingency plans for each supply system, both in domestic supply and in agriculture;
b) Educational campaigns for water saving during drought events;
c) Information system on water uses;
d) Criteria and resources to provide technical and financial support to drought affected institutions;
e) Institutional framework for the creation of a permanent drought prediction and monitoring system.

None of these had any publicly announced developments. Furthermore, two working groups were to continue their tasks: one was to review the legal framework regulating the functioning of the Reservoir Management Commission, where most key decisions were taken (including the creation of the Drought Commission), and another one was to create a permanent drought prediction and monitoring system. Up to the present, neither of these working groups has produced any known results.

5 Results and conclusions

Results of the analysis point out the dispersion and lack of coordination within the wide range of legal instruments in the Algarve region, especially under scarcity conditions as occurred in 2004-06, and summarized in Table 2. An effective risk management approach to drought requires improved linkages between existing policy and planning tools, and may suggest the need for an integrated regional drought plan. Nevertheless, integration between surface and groundwater on the supply side, between different activities on the demand side, and between managing institutions on a permanent basis, are key issues (Nunes *et al* [8]) to be addressed before any drought-specific plan is elaborated, as it may be just intended action rather than an effective mitigation tool.

Table 2: List of major policy tools active in the Algarve covering drought issues.

Policy tool	Thematic scope	Geographic scope	Drought relevance
Water Framework Directive	Water resources	European Union	Policy guidance
Water Law	Water resources	Portugal	Policy guidance, top level planning
National Water Plan	Water resources	Portugal	Thematic analysis, WBPs guidelines
Water Basin Plans	Water resources	Guadiana and Algarve Streams River Basins	Regional objectives and drought planning structure
Albufeira Convention	Surface water resources	Joint Iberian River Basins	Flow regime, strategic policies
Drought Commission 2005	Drought affected activities	Portugal	Coordination and decision on emergency actions
AdA Contingency Plan	Domestic supply system	Algarve (90%)	Alternative and emergency sources
Irrigation schemes regulations	Public irrigation perimeters	Algarve (14%)	Management of supply reduction

Such plans should be consistent with the WFD principles, and could well be elaborated at the water basin scale, in close collaboration with the neighbouring

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Iberian water regions. Portuguese authorities should in fact seek to learn from recent policy improvements in Spain, where Contingency Plans for large urban areas (over 20.000 inhabitants), and integrated Drought Plans for water basin districts, are currently being implemented.

In this context, there is high expectancy on the forthcoming revision of the Portuguese-Spanish Convention of Albufeira, during 2007. This might be of particular interest since both countries are currently leading the efforts for a common European policy on drought and water scarcity, and this has already been identified as one of the key environmental priorities to be assumed during the Portuguese presidency of the EU during the second semester of 2007.

References

[1] Wilhite, D.A., Drought as a natural hazard: concepts and definitions (part I). *Drought: a global assessment*, ed. D.A. Wilhite, Routledge: London and New York, pp. 3-18, 2001.

[2] European Commission (EC), Directive 2000/60/EC of the European Parliament and of the Council of 23 October 2000 establishing a framework for Community action in the field of water policy, *Official Journal of the European Communities*, **L327**, pp. 1-72, 2000.

[3] European Environment Agency (EEA), *Sustainable water use in Europe – Part 3, extreme hydrological events: floods and droughts*, Office for Official Publications of the European Communities, Luxembourg, 84 pp., 2001.

[4] EurAqua, Towards a European Drought Policy, November 2004. http://www.euraqua.org/download/Drought%20brochure%20992kb.pdf

[5] World Wildlife Fund (WWF), Drought in the Mediterranean: WWF Policy Proposals, July 2006. http://assets.panda.org/downloads/wwf_drought_med_report_2006.pdf

[6] Serra, P., A cooperação Luso-Espanhola na aplicação da DQA. *Proc. of the 4th Iberian Congress on water management and planning*, Coord. C.I. Martí & N.P. Fornels, Fundação Nova Cultura da Água, Zaragoza, pp. 35-44, 2006.

[7] Do Ó, A. & Monteiro, J.P., Estimação da procura real de água no Algarve por sectores. *Proc. of the 5th Iberian Congress on water management and planning*, Fundação Nova Cultura da Água, Faro, 4-8 December 2006 (in press).

[8] Nunes, L., Monteiro, J.P., Cunha, M.C., Vieira, J., Lucas, H., Ribeiro, L., The water crisis in southern Portugal: how did we get there and how should we solve it. *Management of Natural Resources, Sustainable Development and Ecological Hazards, WIT Transactions on Ecology and the Environment, Vol. 99*, eds. C.A. Brebbia, M.E. Conti & E. Tiezzi, WIT Press, pp. 435-444, 2006.

Indices of water availability assessment on hydrological basins: a case in Mexico

I. Velasco
Hydrology Department, Mexican Institute of Water Technology, Mexico

Abstract

Water use and demand are increasing sensitive; all the productive sectors require bigger volumes with a minimal quality to complete the expectations in the requirements of food, goods, services and manufactured products.

The hydrological systems are under increasing pressure and in Mexico City this need has led to severe crises owing to the inadequacy of the natural water supply to meet this demand. Particularly, water over-assignments phenomenon is present; the assignment to some users with volumes that overcome the natural capacity, due mainly to ignorance about the real watershed capacity, so these volumes lead to stressing of the hydrological systems, creating dangerous situations because the conflicts and social unbalances are increased, especially during shortage periods. Given the complexity and uncertainty on hydrologic phenomenon and the variability in time, if it is not properly assessed it can drive to over evaluating the water supply capacity and then to the creation of false expectations that potentially cause a crisis and negative impacts because of water inadequacy.

In the face of this dilemma, it is essential to carry out hydrological balances keeping in mind that watershed is the unit of analysis, and considering the diverse water uses, individually and as a whole in order to evaluate the potentiality, as objectively as possible, of the water available related to water demand. This allows us to estimate the level of relative availability, and on this basis, to improve water planning with a reasonable risk of falling under conditions such that imply over exploitation and committing their sustainability. This acquires special importance for basins where hydrometric information is scarce or null, and then one has to apply indirect methods for water availability estimation; the results are expressed graphically as a "semaphore plane".
Keywords: hydrologic balance, water availability, water resources assessment.

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1 Introduction

Water demand has a continued growth, but not its availability.

To achieve a regional balanced development of the activities that demand water as a basic input, it is necessary that water demands are in agreement with water availability. To balance both components is not always an easy task, because of the multiple water uses (agriculture, domestic use, aquaculture, industry, energy generation, etc.), and because frequently the users do not have a registration or official assignment with the correct physical location and the volumes to use; on the other hand, the offer is also frequently only supposed, since measurements are not made, neither controlled due to, among other factors, the lack or inadequacy of the hydrometric network, the lack of economic resources to assist this activity, and the rural and isolated control points, etc.

Nevertheless, the environmental sustainability, considering the water as the main axis, depends on the knowledge about the certainty of the availability of the resource, as well as its demand, in order to achieve an exploitation and rational use without causing unnecessary water stress and without committing the stability and future development.

2 Technical and institutional basis

Estimating the available water volumes in Mexico on an annual basis, is based on an official standard [1, 2], relative to the water conservation, which establishes the specifications and the methodology for determining the annual average availability of the national waters on a hydrological basin as a unit of analysis.

Briefly, the method is based in estimating the balance between demand and supply, when both components are calculated separately, with the considerations for each case. For the supply, available hydrometric information is used, registered as monthly runoff volumes measured in specific places, usually at the basin or sub basin exit, where there is a reservoir, a diversion dam or a hydrometric station. To these volumes are added the upstream used volumes within the basin, to obtain a total runoff volume. On the other hand, the demand is considered from the last or the lowest delivery point toward upstream, in order to accumulate the partial volumes and then to obtain the whole demand to the hydrological system.

Starting from knowing the assigned water and the runoff in a final point of the sub basin *i*, one has that [2]:

$$A_{Bi} = C_{P_i} + A_{R_i} + R_i + I_i - (Uc_i + E_{V_i} + E_{X_i} + \Delta V_i) \tag{1}$$

where:

A_{Bi} runoff toward downstream of the sub basin

C_{Pi} runoff or water contribution within the sub basin

A_{Ri} runoff from upstream sub basin(s)

R_i water returns within the sub basin
I_i water importation from other basin(s)
Uc_i water use (consumptive use) within the sub basin
Ev_i water evaporation from reservoirs
E_{xi} water exportation to other basin(s)
ΔV_i annual volume variation in stored water (reservoirs) (V_2 - V_1)

This leads to estimating the volume that is available in the final point of the sub basin, which is the same that enters to the immediately downstream basin.

2.1 Reserved volumes estimation within the sub basin

The downstream reserved volume, R_{XY}, for a given sub basin *X*, is the fraction of the runoff that comes out at the end of the area, such that it contributes to satisfy the extractions and demands of the next downstream sub basin *Y*; and reserved volume for sub basin *X*, R_{XX}, is that which contributes to the satisfaction of demands inside the same sub basin *X*. The demand estimation is carried out from downstream toward upstream.

2.2 Available volumes in each sub basin

Water volumes available at the end of a sub basin (D_{XY}), can be estimated as the difference of A_{Bi}, the runoff downstream, minus the reserved volumes, R_{XY}, those volumes which sub basin *X* contributes to satisfy the demands of sub basin *Y*. This way, the available volumes from sub basin *X* to sub basin *Y* are:

$$D_{XY} = A_{BX} - R_{XY} \tag{2}$$

Similarly, the remainder water volumes, available for sub basin X itself are:

$$D_{XX} = C_{PX} - R_{XX} \tag{3}$$

In order to classify the sub basins according to water available, the term relative availability coefficient *(Dr_i)* is used, which is expressed by the equation:

$$Dr_i = \frac{Cp_i + Ar_i}{Uc_i + Vc_i} \tag{4}$$

where Vc_i is the committed water volume that is equal to the reserved volume from sub basin *X* for a sub basin *Y* (R_{XY}), downstream; although at the moment it is not contemplated as such, future versions of R_{XY} should include the environmental flow (or ecological volume) focused to the preservation of natural flora and fauna, especially on the river beds.

In accordance with the range where estimated Dr_i is located, the respective sub basin will be classified following the distribution shown in Table 1. The Dr_i

values are conventional and they show the proportion between water available (the offer) and water requirements (the demand).

Table 1: Conventional characteristic of the relative availability coefficient Dr_i.

Range	code	Color	Description
$Dr_i < 1.4$	1	Red	**D**(eficit)
$1.4 < Dr_i < 3.0$	2	Yellow	**E**(quilibrium)
$3.0 < Dr_i \leq 9.0$	3	Green	**R**(eserve)
$9.0 < Dr_i$	4	Blue	**A**(bundance)

Thus, Dr_i is a conventional measure of water stress that is estimated for a sub basin, and means the commitment level of water assignments related to natural water volumes given by rain and runoff.

3 Application case

The case here is presented is related to the San Pedro River Basin, located in the Center-West of Mexico (figure 1), that includes approximately 28,563 km^2, and is formed by 11 well defined sub basins, two of which are closed, with no connection or exit, and the other nine are connected to each other downstream, having a final point at the discharge to the Pacific Ocean (figure 2).

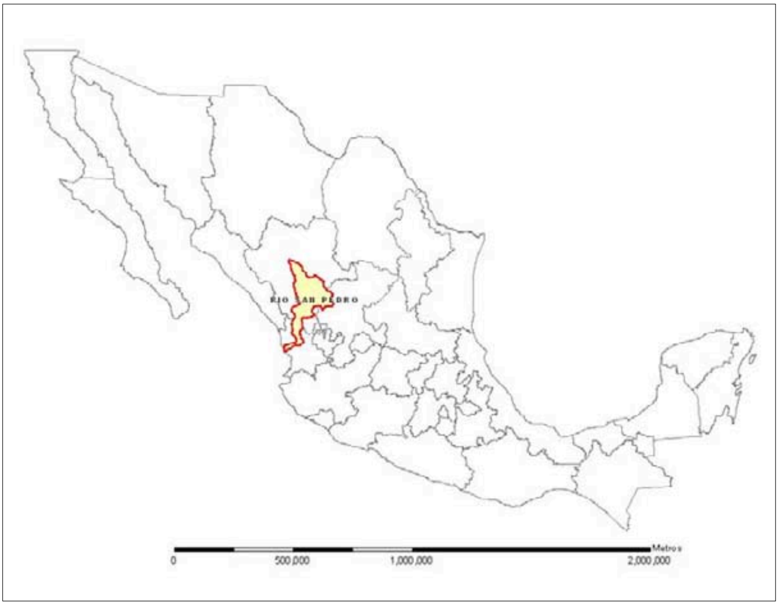

Figure 1: Location of the San Pedro River Basin in Mexico.

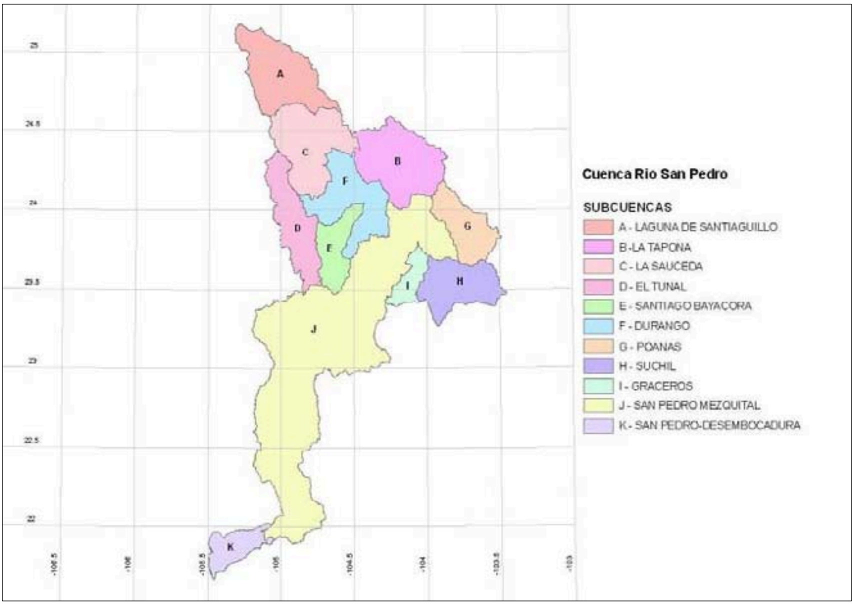

Figure 2: Sub basins within the San pedro River Basin.

Although a defined hydrographic network exists, which includes main and tributaries streams in all the sub basins that discharge to San Pedro River, which finally ends in the sea, the hydrometric information is not as complete as would be desirable, neither has it the record longitude and homogeneity since the dynamics of the basin has changed now that reservoirs, diversion works, and other works have been built which alter the old mesurements sites, or they have disappeared or been cancelled, which all results in that the data cannot be used efficiently as a direct information source.

In those cases, it is necessary to use the indirect method to estimate the surface water contributions; it consists basically on transporting the runoff coefficients, *CE*, from physiographical neighbor and similar sub basins, which have enough hydrometric records. The *CE* is the relationship *measured (runoff)/rainy volume*, both for the same period. That gives a dimensionless fraction as a result, in the interval 0.03 at 0.85, which indicates the proportion of the rain volume that flows and is measured in the gauging site or hydrometric point; so, the flatter and covered (vegetation) the surface, the smaller the *CE;* then, this parameter is also an indirect measure of the covering and slope.

Certainly, homogeneity among sub basins is something relative and subjective, since the differences can be minimum but lead to different effects, for what the analyst judgement and experience are decisive.

For this basin, the application refers exclusively to surface water, because groundwater is not significant, although the Mexican Standard also understands groundwater and its interrelation with surface water.

For many Mexican river basins, mainly the most exploited, for many years the water authorities have established and sent ordinances which prohibited and/or limited the water extraction and/or assignment in bigger volumes in order to avoid demand exceeding availability, with a possible crisis for water inadequacy. For this basin, such an official ordinance has existed since 1955 [3]; the value and importance of these ordinances have been decisive to avoid or to palliate the conflicts, especially when water scarcity periods have occurred. These water regulations are very effective because in Mexico the water (and in general all the natural resources) are the nation's property, and not particularly those of the separate States which are part of the country.
Table 2 shows the main characteristics of the eleven sub basins that compose the San Pedro River Basin, as well as the basic outcomes of the water balance, expressed by the Dr_i.

Table 2: Main characteristics of the sub basins and the hydrological balance.

Sub basin	Area, km^2	Water Volumes, hm^3				Dr_i	Code
		U_C	C_P	A_B	D_{XY}		
A	2,362	8	125	35	35	1.39	D
B	2,594	3	88	85	85	31.09	A
C	2,452	16	129	76	50	1.64	E
D	1,800	0.297	139	130	87	2.66	E
E	1,092	24	82	51	34	1.71	E
F	2,171	97	94	254	63	3.01	R
G	1,400	35	44	2	1.6	1.04	D
H	1,733	13	36	23	21	2.45	E
I	597	0.144	10	9	9	10.96	A
J	11,521	15	2,509	2,782	2,314	12.92	A
K	842	221	272	2,833	253	13.79	A
TOTAL	28,563	433	3,528		2,953		

The graphical presentation of these results is shown in figure 3, called the *semaphore plane*, where using contrasting and conventional colors it is simple to appreciate more qualitatively and objectively the concept of relative water availability.

This way, in a first approach, this graphical result is useful so that the analysts confirm the truthfulness of the results with the reality, and also to improve the perception and sensibility of the water users in order to inform them that this is a limited and scarce resource and that its use should be made with efficiency and respecting the rights of all the users to water access, and the downstream committed volumes as well as the priorities in water use.

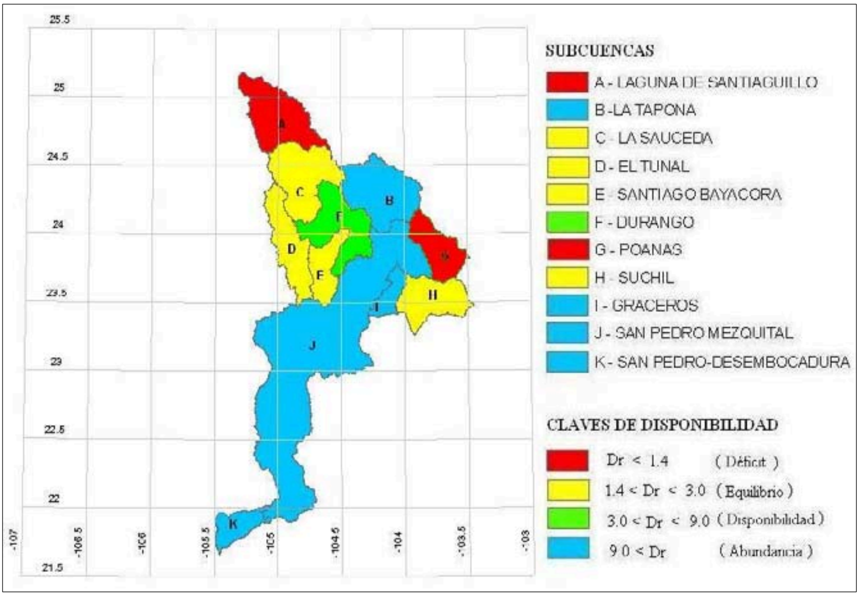

Figure 3: Semaphore map on relative water availability.

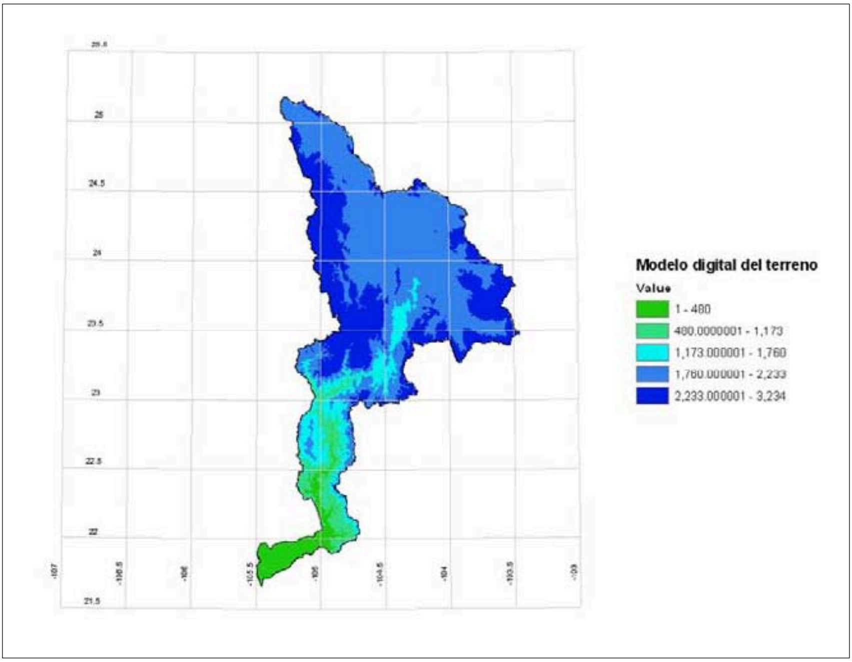

Figure 4: Digital terrain model, in meters above sea level.

This perception is supplemented and improved if the water availability results are related to other physical aspects of the basin such as topography expressed, for example, through digital terrain models (figure 4), as well as isohyetal maps (figure 5); both concepts help for a better understanding of the occurrence and physical rain distribution on the area, as well as the reason why unauthorized or bigger water volumes are not be used because downstream commitments and other users exist and have the same water rights.

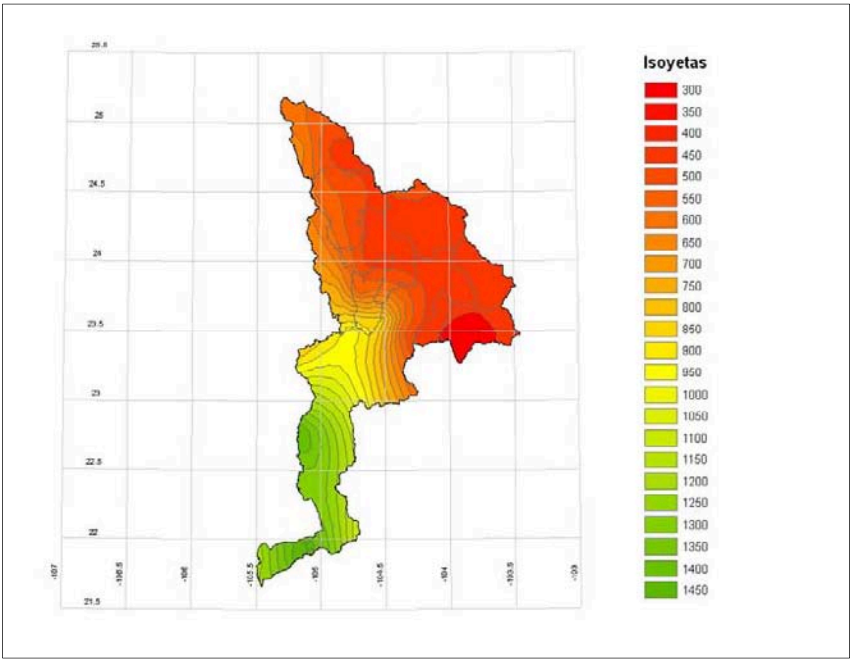

Figure 5: Annual isohyetal map, in millimeters, during the 1980-2001 period.

It should be kept in mind that Dr_i is just a conventional index and it seeks to indicate the degree of water stress or use, commitment or exploitation of the available resource; therefore, it is important to consider that mainly in the lower ranges (deficit, equilibrium, and reserve) the index evaluation is very sensitive to the change in one or both components (supply and demand), so still having a dimensionless numeric value, it is not definitive, since if the water balance evaluation or upgrade is made as is recommended i.e. every three years, it is very possible that there should be some changes from one range to another.

That is why it is important to have good hydrometric, meteorological information, and details of the demands.

Also, regarding the relative Dr_i index, its interpretation should be critical and analytic, because its value can be deceiving or confused: when the results show an abundance of water and indicate that the supply is greater than demand, but, due to the marked regional low water periods, if there is not way to retain and conserve the water through reservoirs or lakes to use it when it is required and there is no rain season, then such abundance is fictitious because monsoon lasts a short period, no more than four months.

To obtain the isohyetal lines, monthly data there analyzed the rain records in meteorological stations that cover the basin during the period 1980-2001, which made it possible to homogenize the information. So, the used meteorological information is appropriate because, besides its homogeneity, it covers 20 years as the Standard indicates, and is representative of the real basin conditions.

Although the hydrological network is well defined, it was not always possible to obtain the suitable information due to the heterogeneity of the records. There are in the basin several reservoirs with their respective hydrometric records: inputs and extractions [4], and in those cases it was possible to estimate the total runoff (including the used volumes upstream of the reservoir), as well as to obtain the runoff coefficients *CE* which were used because of the fact that in the neighboring sub basins hydrometric records do not exist, or they are not enough nor appropriate. In this case, homogeneity among sub basins is considered appropriate since the vegetation, topography, and climate conditions are similar, so there are no significant differences; and then, transferring runoff coefficients from one area to another is considered a reasonable process.

Nevertheless, there is an aspect that requires special attention, and its carefulness will be very useful for the next Standard upgrades [1]: water demand and commited volumes. In Mexico, since some years ago, the Water Rights Public Registration (REPDA in Spanish), has been implemented as the legal instrument that allows to register all the water users and uses, which works as an inventory, evaluation and management tool. The potential of this tool is very positive, although the first version has some frequent flaws, like the water demand volumes or water rights location because the registered geographical coordinates, frequently not verified, as well as the possible typing errors, lead to wrong data of the distribution and geographical location; that means that they fall outside of the true sub basin, and that contributes to uncertainty and errors in the water demand accounting which affects the balance giving possible errors in the results.

4 Conclusions

The obtained results of this application are appropriate with the observed reality, so this fact allows us to affirm that the procedure is correct. The results are annual, but we are already working in adapting them to a monthly scale, which is important because the basin, as almost the whole country, has a summer season rain regime of a monsoon type, from July to October in which 80% of total rain occurs, so its intensity is high and frequently torrential. Then, there is a marked period of low water runoff during which the available water volumes can be lower than demands.

In complement to the purely hydrological calculation, complementary aspects such as the topographical, meteorological, demographic, communication network, and agrological aspects are of high utility in order to make the results more valuable, and then to enrich the vision and to improve objectively the natural hydrological environment vision as a whole.

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References

[1] CNA, 2003. Programa Hidráulico Regional 2002-2006, Pacífico Norte, Región III. (National Water Commission, Hydraulics Regional Program 2002-2006, Northern Pacific Region) Pp. 166.

[2] CNA, 2002. Norma Oficial Mexicana NOM-011-CNA-2000: Conservación del recurso agua que establece las especificaciones y el método para determinar la disponibilidad media anual de las aguas nacionales. Publicada el 7 de abril de 2002. (National Water Comisión. Official Mexican Standard NOM-011-CNA-2000, Published on April 7, 2002)

[3] DOF, 1955. Diario Oficial de la Federación, donde se publicaron los decretos de veda en la concesión y uso de aguas superficiales para la cuenca del río San Pedro, 08 de febrero de 1955. (Mexican Federal Official Journal, Legal Ordinance related to prohibition on use and assignment of surface water, on the San Pedro River Basin. Published on February 8, 1955)

[4] SRH. 1968. Boletín Hidrológico No. 30 Región Hidrológica 11, Presidio-San Pedro. Tomo I. (Ministry of Water Resources, Hydrologic Bulletin No. 30, Hydrologic Region 11, Presidio-San Pedro Rivers, Volume I).

Determinants of domestic water demand for the Beijing region

D. Karimanzira, M. Jacobi & C. Ament
Institute of Information Technology and Automation,
Department of System Analysis, Ilmenau, Germany

Abstract

The analysis of demand for water, including realistically forecasting future levels of demand, is an important and critical step in the economic analysis of water supply projects. The results of demand analysis will enable to determine the service levels to be provided, determine the size and timing of investments, estimate the financial and economic benefits of projects, and assess the ability and willingness to pay of the project beneficiaries. Furthermore, the surveys carried out during the demand assessment will provide data on cost savings, willingness to pay, income and other data needed for economic analysis. In this paper, methods of statistical analysis (correlation, regression, etc) will be used to determine the factors, which influence water demand. Each model region may have its own set determinants for domestic water demand and the importance of a given factor may vary from one region to another. Therefore, this paper focuses on the major determinants of domestic water demand for the Beijing region. Several models analyzing the determinants were compared. The models based on a feasible generalized least squares (FGLS) analysis.
Keywords: decision support system, water demand models, statistical analysis, correlation and significance tests, regression analysis.

1 Background

Increasing population growth and the associated process of urbanization in the semi arid city of Beijing, China requires a reliable source of water. Although the city currently has an inexpensive and abundant supply of water, it is imperative that the city faces the challenge associated with providing safe drinking water. This work is part of the Chinese – German joint project "Towards Water-

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Scarcity Megalopolis' Sustainable Water Management System"[6]. This project takes the challenge of water shortage, the outstanding conflict between water supply and demand. It aims at a decision support system (DSS) for the sustainable development of economics and community in Beijing. An essential requirement for such a DSS is a simulation model of the water resources/supply system. Part of the simulation model is shown in Figure 1.

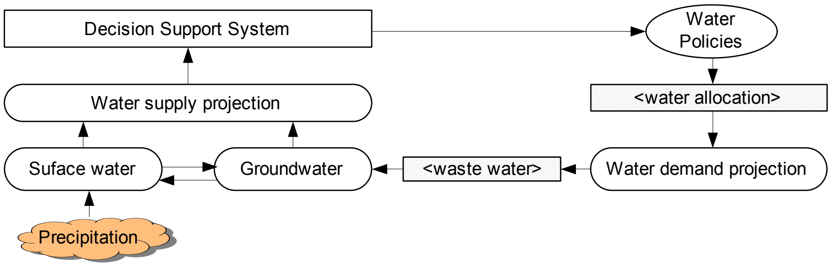

Figure 1: The graphical user interface of the simulation model.

The simulation model comprises the water supply, optimization and the water demand systems. On focus in this paper is part of the water demand system, namely the domestic water demand. Compared to the agricultural and the industrial water demand, the domestic water demand is very difficult to model, because it is determined by several subjective factors.

2 Introduction

Recently, several models for domestic water forecasting have been developed and published in literature. The model from Archibald is a simple component model for calculating the domestic water demand per household. One of the components in the model is "bathing and showering" [2]. The problem with this type of model is that every component has several factors, which should be determined and in most applications there is not enough information to every component and if one component cannot be modelled the whole model would be insufficient. Eqns (1) and (2) describe some of the most used models for domestic water demand forecasting.

$$\Phi_{dwd} = \Phi_{pop} + \eta\Phi_{gdpc}\left[m^3 / Jahr\right] \quad (1)$$

$$DSWI = DSWI_{\min} + DSWI_{\max}\left(1 - e^{-\gamma_d GDPC^2}\right) \quad (2)$$

Eqn (1) describes the IMPACT-Model [1], where ϕ_{dwd} is the growth rate of the domestic water demand, ϕ_{pop} is the growth rate of the population and ϕ_{dwd} is the growth rate of the gross domestic product per capita. Eqn (2) expresses the WATERGAP-Model [4], where $DSWI$ is the domestic structural water intensity and γ_d is the curve retardation.

After analysing several models in literature [1–4] a general model is obtained, which can be expressed as follows:

$$W = f\left(W_{prev}, p, y, d, g, k, v, gdpc, prec, E, Eva, Inc, fam\right) \tag{3}$$

where W is the domestic water demand, W_{prev} is the previous domestic water demand, p is the water price elasticity, y is the income elasticity, d is the residence density (population), g is the individual preferences (e.g. bathing habit), k is the number of individuals per household, $prec$ is the precipitation, Eva is the evapotranspiration, E is the employment rate and v is the weather.

In this work the influence of the different factors on the domestic water demand is analysed using methods of statistical analysis i.e., correlation, regression, multicollinearity and significance tests. In this way we obtain the most important factors, which should be included in every domestic water demand model for reliable results.

It is known that each model region may have its own set of domestic water demand determinants and the importance of a given factor may differ from one project to another. Therefore to prove generality and robustness of the resulting determinants, the same tests were applied to different data sets of different regions of different development status (Canada, Germany).

3 Test for determinants

3.1 Correlation and significance tests

To test the determinants, correlation tests are used in this paper. A correlation describes the strength of an association between variables. An association between variables means that the value of one variable can be predicted, to some extent, by the value of the other. A correlation is a special kind of association: there is a linear relation between the values of the variables. A non-linear relation can be transformed into a linear one before the correlation is calculated. For a set of variable pairs, the correlation coefficient gives the strength of the association. The square of the size of the correlation coefficient is the fraction of the variance of the one variable that can be explained from the variance of the other variable. The relation between the variables is called the regression line. The regression line is defined as the best fitting straight line through all value pairs, i.e., the one explaining the largest part of the variance. The correlation coefficient is calculated with the assumption that both variables are stochastic (i.e., bivariate Gaussian). If one of the variables is deterministic, e.g., a time series or a series of doses, this is called regression analysis. In regression analysis, the interpretation of the correlation coefficient is different from that of correlation analysis. In regression analysis, tests on statistical significance can only be used when the conditional probability distribution of the other variable is known or can be guessed. However, the regression line can still be used.

If the aim is only to prove a monotonic relation, i.e., if one variable increases the other either always increases or decreases, like in most of our cases, then the rank correlation test is a better test.

The normal procedure for performing correlation and significance tests is as follows: First the hypothesis is H_0 made: "The values of the members of the pairs are uncorrelated, i.e., there are no linear dependencies". It is also assumed that the values of both members of the pairs are normal (bivariate) distributed.

Procedure:

The correlation coefficient r_{xy} of the pairs (x,y) is calculated as:

$$r_{xy} = \frac{\mathrm{cov}(x, y)}{\mathrm{var}(x) \cdot \mathrm{var}(y)} \tag{4}$$

$$r_{xy} = \frac{\frac{1}{N}\sum_k (x(k) - \bar{x}) \cdot (y(k) - \bar{y})}{\sqrt{\frac{1}{N}\sum_k (x(k) - \bar{x})^2} \cdot \sqrt{\frac{1}{N}\sum_k (y(k) - \bar{y})^2}} \tag{5}$$

The regression line y = a * x + b is calculated as:

$$a = \frac{N\left(\sum xy\right) - \left(\sum x\right) \cdot \left(\sum y\right)}{N\left(\sum x^2\right) - \left(\sum x\right)^2} \tag{6}$$

$$b = \frac{\sum y - a\sum x}{N} \tag{7}$$

Level of Significance:

The value of $t = r_{xy} \cdot \mathrm{sqrt}((N-2)/(1-r_{xy}^2))$ has a Student-t distribution with degrees of freedom N-2. If the degrees of freedom are greater than 30, the distribution of t can be approximated by a standard normal distribution.

Remarks:

This could be called the most misused statistical procedure. It is able to show whether two variables are connected. It is not able to show that the variables are not connected. If one variable depends on another, i.e., there is a causal relation, then it is always possible to find some kind of correlation between the two variables. However, if both variables depend on a third, they can show a sizable correlation without any causal dependency between them. A famous example is the fact that the position of the hands of all clocks is correlated, without one clock being the cause of the position of the others. Another example is the significant correlation between human birth rates and stork population sizes. To overcome this problem the spearman rank correlation is used in combination with regression analysis is applied in this paper.

3.1.1 Spearman's rank correlation test

This is a test for correlation between a sequence of pairs of values [7, 8]. Using ranks eliminates the sensitivity of the correlation test to the function linking the pairs of values. In particular, the standard correlation test is used to find linear relations between test pairs, but the rank correlation test is not restricted in this way. Given N pairs of observations (x_i,y_i), the x_i values are assigned a rank value and, separately, the y_i values are assigned a rank. For each pair (x_i,y_i), the corresponding difference, d_i between the x_i and y_i ranks is found. r_{xy} is:

$$r_{xy} = \sum_{i=1}^{N} d_i^{\,2} \tag{8}$$

For large samples the test statistic is then:

$$z = \frac{6r_{xy} - N\left(N^2 - 1\right)}{N\left(N+1\right)\cdot\sqrt{N-1}}, \tag{9}$$

which is approximately normally distributed. A further technique is now required to test the significance of the relationship. The *z* value must be looked up on the Spearman rank significance curves (see Figure 3).

4 Determinants for Beijing domestic water demand

Possible influencing factors for the domestic water demand for the Beijing region were selected and are listed in Figure 2 for the period from 1996-2003 [5]. They are factors from the weather, population and economy, which are thought to be obviously linked to the domestic water demand. The main objective of this study is to find out, which factor has the greatest influence and which ones are negligible in the models, and therefore simplify them.

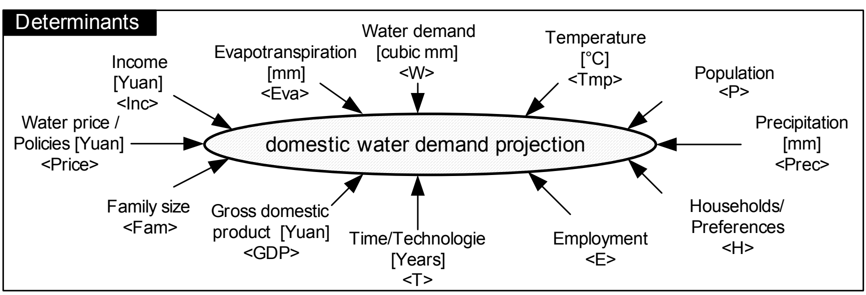

Figure 2: Possible determinants for the domestic water demand for Beijing.

Correlation, regression and significance analysis described in the previous section were performed for the domestic water demand with respect to all input variables including the previous domestic water demand W_{prev}.

5 Results

All estimated cross-correlation coefficients r_{xy} which were significant at the 5% level according to the three-step procedure described above are summarized in Table 1 (bold) and Figure 3. Surprisingly, the magnitude of the total cross correlation coefficient of the domestic water demand and the number of employment is quite large (-0.777). A decreasing domestic water demand by increasing employment cannot be explained logically. The partial correlation coefficient (a measure for the dependence of two variables after switching of the linear influences of other variables) between employment and the domestic water demand confirms this. After switching of the linear influences of other variables the remaining partial correlation coefficient is only -0.065, which practically shows no linear dependence between the two variables. Also unexpected is the minimal correlation of the domestic water demand and the population (correlation coefficient of -0.016). Normally, one would think that the domestic water demand increases with increasing population growth.

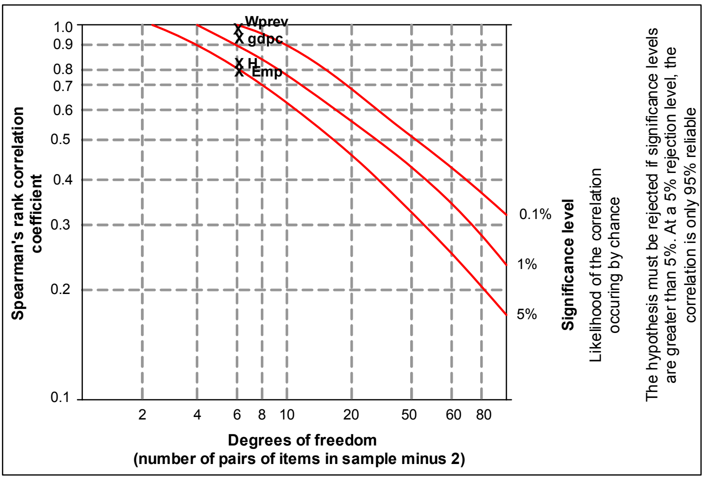

Figure 3: Significance of the Spearman's rank correlation coefficients.

Table 1: Correlation coefficients of the variables.

	T	**P**	**GDP**	**H**	**Tmp**	**prec**	**E**	**W**	**gdpc**
T	1.000	**0.934**	**0.991**	**0.993**	0.047	*-0.546*	*0.406*	0.183	-0.294
P	**0.934**	1.000	**0.928**	**0.947**	-0.199	*-0.413*	*0.548*	-0.016	-0.537
GDP	**0.991**	**0.928**	1.000	**0.996**	0.079	*-0.489*	*0.501*	0.078	-0.373
H	**0.993**	**0.947**	**0.996**	1.000	0.002	*-0.486*	*0.499*	**0.676**	-0.384
Tmp	0.047	-0.199	0.079	0.002	1.000	-0.243	-0.133	0.332	0.382
prec	*-0.546*	*-0.413*	*-0.489*	*-0.486*	-0.243	1.000	0.069	-0.500	-0.199
E	*0.406*	*0.548*	*0.501*	*0.499*	-0.133	0.069	1.000	**-0.777**	**-0.899**
W	0.183	-0.016	0.078	**0.676**	0.332	-0.500	**-0.777**	1.000	**0.824**
gdpc	-0.294	-0.537	-0.373	-0.384	0.382	-0.199	**-0.899**	**0.824**	1.000
Eva	—	—	—	—	—	—	—	**-0.726**	—

Only the previous water demand, gdpc, employment, time and number of households (H) were significant at the 5% level (see Figure 2 and Table 1). Therefore, it is recommended to include these variables as inputs in reduced forecasting models. Regression results also show that several combinations of these variables are possible to obtain a reliable model. Most of the coefficients of the explanatory variables have expected signs. The positive value of temperature suggests domestic consumers use more water when the weather is relatively warm. Precipitation contributes negatively to water consumption, meaning that households tend to use less water when there is enough rainfall. Family size and water price (not shown) are not significant at any level, which may be due to the fact that both variables vary little with time.

Important was also to find the robustness and the generality of the influencing parameters. Therefore, the correlation coefficient for the previous water demand, price, employment, time and number of households to domestic water demand were calculated for other regions of different nature where data could be obtained. Data for Germany and Canada was present and the calculated correlation coefficients were almost similar to that of Beijing, *T, E* and *H* had the highest correlation with the domestic water demand, which suggests that models for forecasting domestic water demand that include these variables are quite reliable. To avoid multicollinearity in the models, the population density *P* is excluded to be an explanatory variable as it is highly correlated with *GDP* and *H*.

To prove our results models, which included combinations of the different variables were implemented and tested. Results of some selected models, described by equations 10 and 14 are shown in Figure 4. All the models are compared with the simple linear model, where the entire yearly water requirement of the households is the product of the estimated domestic water demand per capita and the estimated population.

$$\text{Model 1}: W = \alpha + \beta T + \gamma E + \delta W_{prev} + \Delta Temp + \nu P \quad (10)$$

$$\text{Model 2}: W = \alpha + \beta T + \gamma E + \delta W_{prev} + \Delta Temp \quad (11)$$

$$\text{Model 3}: W = \alpha + \beta T + \gamma E + \delta W_{prev} \quad (12)$$

$$\text{Model 4}: W = \alpha + \beta T + \gamma E + \mu GDP \quad (13)$$

$$\text{Model 5}: W = \alpha + \beta T + \gamma E + \varphi H \quad (14)$$

The parameters in Table 2 were estimated for the different models. Their standard deviation and coefficient of variation in % were also calculated. In Table 3 the results of the correlation tests of the covariance between the calculated parameters are listed. The models could be reduced accordingly.

The results of all the models show a very good adjustment of the model values up to the real water requirement, and also the future development follows a smooth, realistic process. Due to the fact that there could be some effects that are correlated to some explanatory variables the OLS is biased and inconsistent.

Therefore, the parameter estimation was done using feasible generalized least squares analysis.

Table 2: Estimated parameters and their standard deviation.

	Best Estimate (Model)					Standard Deviation (Model)				
	1	2	3	4	5	1	2	3	4	5
α	265.7	-697	-77.947	-1063	-3480	1294	525	532	162	1948
β	-0.0139	0.0363	0.0411	0.055	0.1793	0.0674	0.0271	0.0274	0.0815	0.0998
γ	0.0012	-0.0039	-0.0042	-0.0031	-1.9E-3	0.0064	0.0015	0.0015	9.5E-6	0.0010
δ	0.7722	-0.2478	-0.333	—	—	13.709	0.5554	0.5623	—	—
Δ	-0.013	0.0924	—	—	—	0.1564	0.0849	—	—	—
ν	0.0039	—	—	—	—	0.0047	—	—	—	—
μ	—	—	—	-0.0186	—	—	—	—	0.0512	—
φ	—	—	—	—	-0.0202	—	—	—	—	0.013

Table 3: Correlation coefficient of the parameters.

	α	β	γ	δ	Δ	ν
α	1.0	-0.9999	0.9746	0.9938	-0.7033	0.9029
β	-0.9999	1.0	-0.9757	-0.994	0.6997	-0.9045
γ	0.9746	-0.9757	1.0	0.9744	-0.7655	0.9688
δ	0.9938	-0.994	0.9744	1.0	-0.7044	0.9034
Δ	-0.7033	0.6997	-0.7655	-0.7044	1.0	-0.8183
ν	0.9029	-0.9045	0.9688	0.9034	-0.8183	1.0

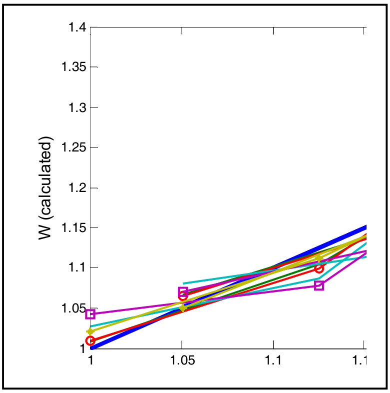

Figure 4: W using different models (calculated vs measured).

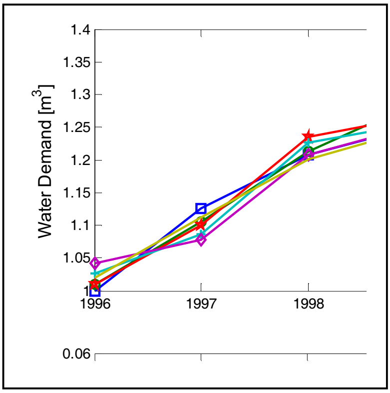

Figure 5: Domestic water demand forecasting using different models.

Figures 3, 4 and Table 4 show the qualitative and quantitative results of the models, respectively. The sum of the squares of the residuals, the multiple correlation coefficients for I/O data and the linear correlation coefficient of measured output and the calculated output show the best results in the following order: 1, 5, 2, 3 and 4. Due to the high correlation of the domestic water demand and the number of households the reduced model (Model 5) that include the number of households shows better results compared to Model 2.

Table 4: Statistical results of the models.

	Model 1	Model 2	Model 3	Model 4	Model 5
Sum of squares of residuals	0.0037	0.0049	0.0069	0.0072	0.0047
Correlation: x - y data					
Multiple Correlation Coefficient (MCC)	0.9756	0.9672	0.9539	0.9514	0.9688
MCC F-test ratio	147.908	192.969	252.363	238.411	382.102
MCC F-test probability	0.0257	0.0077	0.0024	0.0028	0.00093453
Correlation: y(measure) - y(calculated)					
Linear Correlation Coefficient (LCC)	0.9756	0.9672	0.9539	0.9514	0.9688
LCC Probability	3.6625E-6	1.248E-5	3.3178E-5	3.9827E-5	8.5582E-6
Degrees of freedom	2	3	4	4	4
Number of data points	8	8	8	8	8
Number of estimated paramaters	6	5	4	4	4

6 Conclusions

The applied study is an important starting point for the development of simple and robust models. For the residential sector the variables of the economic water use models include income, household size, housing density, air temperature, rainfall, marginal price, and fixed charges for water and wastewater. In most regions there is no data available for most of these parameters. It is therefore very important to know which parameter, can be used at the minimum to produce some reliable results. In this study several variables have been tested for their influence on the domestic water demand. It has been shown that to predict domestic water reliably at least the gdpc, the previous water demand, employment rate, the time and the number of households must be included. The estimation can be improved by using panel data covering a longer time period or more disaggregated sub-regional level analyses. It would also be useful to extend the study with more adequate data especially regarding time series water prices for the domestic sector. Well-designed household surveys would provide richer information and greater insights into the factors influencing domestic water demand. The results of the comparison between Canada, Beijing and Germany shows that in water abundant areas more water will be used and also the increased positive correlation coefficient of income (Beijing 0.345; Canada 0.578; Germany 0.623) implies that consumers who have a high income tend to consume more water.

References

[1] Rosegrant, M., Ringler, C., Msangai, S., Cline, S. *International Model for Policy Analysis of Agricultural Commodities and Trade (IMPACT-WATER): Model Description*. Int. Food Policy Research Inst, Washington, D.C., 2005.

[2] Archibald G.G. *Forecasting Water Demand – A Disaggregated Approach*. Journal of Forecasting, 1983.

[3] Zena Cook, Scott Urban. *Domestic, Commercial, municipal and Industrial water Demand; Assessment and Forecast in Ada and Canyon Counties, Idaho*. Idaho department of Water resources IDWR, 2001.

[4] Petra Döll, Joseph Alcamo, Thomas Henrichs, Frank Kaspar, Bernhard Lehner, Thomas Rösch and Stefan Siebert. *The Global Integrated Water Model WaterGap 2.*1, 2001.

[5] *Beijing Municipal Bureau of Statistics. Beijing Statistical Information net*, Mai 2005. http://www.bjstats.gov.cn/english/index.html.

[6] Rauschenbach T. The Introduction of Water Resources in Beijing and the Demand Analysis, July 2005.

[7] Hartung J. *Statistik. Lehr- und Handbuch der angewandten Statistik*. 13. Auflage, Oldenbourg Verlag München Wien, 2002

[8] Wernstedt J. Experimentelle Prozessanalyse. 1. Auflage. VEB Verlag Technik Berlin, 1989

[9] Brockwell, P.J. and Davis R. A. Time Series: Theory and Methods. Springer Verlag, New York. 1991

Non-elastic matrix model for hydraulic networks calculation

V. E. M. G. Diniz[1], P. A. Souza[1] & E. L. Junior[2]
[1]*Department of Sanitary and Hydraulic Engineering of Polytechnic School, São Paulo University, Brazil*
[2]*Campinas State University, Brazil*

Abstract

This paper presents a non-elastic matrix model to calculate hydraulic networks, based on a method created by Nahavandi and Catanzaro (*Journal of Hydraulics Division*, 99(HY1), pp.47-63, 1973). It is a method that calculates the discharges and pressure heads in hydraulic networks for the steady state, for the extended period and for the transient state. This method has advantages concerning the Cross method, because the latter does not allow the calculation of transient situations such as the settings of valves, the starting and stopping of boosters, the branch ruptures, etc. The applicability of the method created by Nahavandi and Catanzaro was enhanced, because the programming and input data to consider the presence of valves, reservoirs or boosters in the hydraulic network were developed. Furthermore, the mathematical formulation and programming to calculate the extended period and transient state were also developed. The matrix method is working well, because the model was applied to calculate some hydraulic networks used as examples and the values calculated by the model are similar to the ones obtained from the technical literature.
Keywords: hydraulic networks calculation and operation, software.

1 Introduction

In the water resources field, the unbalance between water supply and water demand obliges more and more elaborated solutions from the engineer. As countries develop, problems related to water, like cities supply, water transference among watersheds and mainly the lack and the difficulty to obtain

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financing founds to build new hydraulic works, demand the existing systems are more and more efficient.

The operational control of hydraulic networks to attend population demands during the day is a problem that has been searched for many years and until nowadays the solutions are not always optimized, resulting in flaw risks for water supply.

The operational control of hydraulic networks has many variables that must be controlled and optimized to obtain the best efficiency in operation, such as: a) water level in reservoirs; b) pressure heads all over the hydraulic network; c) number of valve settings; d) supply discharge; e) booster set ups; f) operations to avoid hydraulic transients.

The proposition of this paper is to develop a hydraulic network calculation model more efficient than the Cross method to calculate the pressure heads and discharges for the steady state and for the extended period and that can be applied to calculate slow transients without using the characteristic method.

2 Literature review

Ormsbee and Wood [2] proposed an algorithm that used a truncated expansion of Taylor's series to linearize the energy equations and the conservation of mass equations (written in terms of pipe diameter and velocity) for all network pipes and nodes respectively. This method is a modified version of the linear method *apud* Wood [9].

Jowitt and Xu [3] developed an algorithm to determine the values of flow control valve settings to minimize leakage. The non-linear basic hydraulic equations of the network, which describe the node heads and the flow rates in the pipes, are augmented by terms that explicitly account for pressure-depended leakage by terms that model the effect of valve actions. These equations were linearized using the method *apud* Wood [9].

Todini et al [4] apud http://www.dha.lnec.pt/nes/epanet/downloads /EN2Pmanual.pdf developed the "Gradient Method". This method solves the energy equations and the conservation of mass equations and the relation between discharge and head loss, which feature the conditions of hydraulic balance of the network in a given moment. The Gradient Method is used by computer programs as EPANET and WATERCAD.

Vairavamoorthy and Lumbers [5] developed an optimization method to minimize leakage in water distribution systems through the most effective settings of flow reduction valves. This problem was formulated as a nonlinear programming problem and solved using a reduced sequential quadratic programming method. The method showed advantages compared to previously published techniques in terms of robustness and computational efficiency. A feature of this approach is the use of an objective function that allows minor violations in the targeted pressure requirements. This allows a much greater improvement in the violations of minor pressure requirements that would be achieved otherwise.

Filion and Karney [6] developed a hybrid model that combines the modeling sophistication of a transient simulator and the time-stepping efficiency of a quasi-steady state model and can simulate steady and unsteady interactions in a system over an extended period. The model's procedure consists of running water hammer simulations at the start and end of an extended time step to track the rate of filling of a system's reservoirs and then use this information to update reservoir levels at the end of the time step. Extended period and worst-case simulations presented in a case study suggest that the hybrid model has a high routing accuracy and can be used to identify the critical state, which will produce the most severe transients in a system.

Goulter [7] showed that the system analysis techniques, and in particular optimization, used to design water distribution networks have not been accepted into practice although the component design models are quite robust, versatile and capable of handling relatively complicated design problems. According to Goulter [7], it happens mainly because of the lack of suitable packaging of the models for ease of use in a design environment. There's also a lack of a network reliability measure due to the complexity of the reliability problem in water distribution networks. There is a need for development of decision support systems for design of water distribution networks. These systems should be able to combine optimization and simulation models and to use an interactive graphical basis to assist in the inclusion and interpretation of reliability in the network solutions and to develop alternative solutions.

Mpesha et al [8] used a frequency response method to determine the location and rate of leakage in open loop piping systems. A steady-oscillatory flow, produced by the periodic opening and closing of a valve is analyzed in the frequency domain by using the transfer matrix method, and a frequency response diagram at the valve is developed. Several piping systems were analyzed for all practical values of the friction factor (0.01-0.025) to detect and locate individual leaks of up to 0.5% of the mean discharge. The method, requiring the measurement of pressure and discharge fluctuations at only one location, has the potential to detect leaks in real life open loop piping systems conveying different kinds of fluids, such as water, petroleum and others.

3 Method

The non-elastic matrix model for hydraulic networks calculation is based on a method created by Nahavandi and Catanzaro [1]. It is a method that calculates the discharges and the pressure heads distribution in hydraulic networks for the steady state, for the extended period and for the transient state.

This method has great advantages concerning the Cross method, because the latter doesn't allow the calculation of transient situations such as the settings of valves, the starting and stopping of boosters, the branch ruptures, etc.

It will be adopted the following simplifying hypotheses: (1) incompressible fluid; (2) turbulent and isothermal flow; (3) non-elastic pipe; (4) it will be used the same friction factor "f" value to calculate the head loss in the transient state and in the steady state.

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3.1 Mathematical formulation for the steady state

A connection matrix [C], formed by the elements 1, -1 and 0 is defined by the following way: each branch of the hydraulic network corresponds to a row in the matrix and each node of the hydraulic network corresponds to a column in the matrix. An element Cij of the connection matrix may have the following values:

Cij = 0 → If a branch i is not connected to a node j.

Cij =-1 → If a branch i is connected to a node j and the flow of branch i goes to a node j.

Cij = 1→ If a branch i is connected to a node j and the flow of branch i comes from a node j.

The method is based on 3 (three) equations written in matrix form. Eqn. (1) relates the pressure head difference on a branch to the pressure heads on the nodes at the beginning and at the end of the same branch.

$$\left\{\frac{\Delta P}{\rho g}\right\} = [C]\left\{\frac{P}{\rho g}\right\} \tag{1}$$

Eqn. (2) of the method is the continuity equation written to the nodes in matrix form:

$$\left[C^T\right]\{Q\} + \{Q_{es}\} = \{0\} \tag{2}$$

Eqn. (3) of the method is the momentum equation written in finite differences form (matrix form):

$$\left\{\rho\frac{\pi D^2}{4}L\left(\frac{v-v^0}{\Delta t}\right)\right\} = \left\{\Delta P\frac{\pi D^2}{4} + \rho g\frac{\pi D^2}{4}L\left(\frac{\Delta Z}{L}\right) + \rho g H\frac{\pi D^2}{4}\right\}, \tag{3}$$

$$-\left\{\rho g \Delta H_v\frac{\pi D^2}{4} + F_{av}\right\}$$

After some algebraic operations in eqns. (3) an (2) and solving these equations to $\{Q\}$ and $\{P/\rho g\}$ respectively, the result is:

$$\{Q\} = \left\{Q^0\right\} + \left\{\beta\frac{\Delta P}{\rho g} + \beta\Delta Z + \beta H - \beta\Delta H_v - \beta P_c\right\} \tag{4}$$

$$\left\{\frac{P}{\rho g}\right\} = \left[M^{-1}\right]\left[C^T\right]\left\{-\beta\Delta Z - \beta H + \beta\Delta H_v + \beta P_c - Q^0\right\} - \left[M^{-1}\right]\{Q_{es}\} \tag{5}$$

The numerical solution of the problem can be accomplished by solving eqns. (5), (1) and (4) using a computer. The input data to accomplish this analysis are: the hydraulic network topology, the geometric dimensions, the hydraulic properties, the initial conditions of the problem and the control variables.

Although the equations for the steady state consider the presence of boosters and valves, the software accomplishes the calculations considering only pipes, nodal demands and the presence or not of reservoirs with constant water levels. This procedure is adopted to give the user an idea of how the hydraulic network will work considering only the action of gravity acceleration.

3.2 Mathematical formulation for the extended period

After the software finishes calculating the steady state, it begins to calculate the extended period during the day. The day is divided in 24 time steps of 1 hour each one to calculate the extended period and the user may decide the number of time steps to be calculated, varying from 1 to 24 time steps.

The extended period has the same equations of the steady state because any given time step corresponds to the steady state of that moment with its own features. The differences between the steady state and the extended period are that the user may decide if there will be boosters and/or valves operating during the day on the hydraulic network or not, if there will be the presence of reservoirs or not, the nodal demands may vary from a period to the next one and the reservoirs may vary the water levels. The reservoir water level variation is calculated by the continuity equation:

$$\{\Delta H\}\{A\} = \Delta t\{Q_e\} - \Delta t\{Q_s\} \tag{6}$$

To calculate the pressure heads on the hydraulic network nodes, $\{DH\}\{A\}$ is divided by Dt and the result is multiplied by $[M^{1}]$ and added to eqn. (5):

$$\left\{\frac{P}{\rho g}\right\} = \left[M^{-1}\right]\frac{\{\Delta H\}\{A\}}{\Delta t} + \left[M^{-1}\right]\left[C^T\right]\left\{-\beta\Delta Z - \beta H + \beta\Delta H_v + \beta P_c - Q^0\right\} \tag{7}$$
$$-\left[M^{-1}\right]\{Q_{es}\}$$

After it, the numerical solution of the problem is reached solving eqns. (1) and (4).

3.3 Mathematical formulation for the transient state

After calculating the discharges and pressure heads to any given time step of the extended period, the coefficients C_{es} to the nodes that have nodal demands are calculated and then a diagonal matrix $[C_{es}]$ is built having the C_{es} values in its diagonal. The following equation is used to it:

$$\left\{\left(\frac{P}{\rho g}\right)^{0.5}\right\}[C_{es}] = \{Q_{es}\} \tag{8}$$

In the transient state, the nodal demands values ($\{Q_{es}\}^*$) will be calculated using the hydraulic theory for discharge calculation through orifices and nozzles. The following equation is used:

$$\{Q_{es}\}^* = [C_{es}]\left\{\left(\frac{P_{es}}{\rho g}\right)^{0.5}\right\} \tag{9}$$

The coefficients C_{es} values used in eqn. (9) are the same calculated at the end of the extended period time step because the variation of C_{es} values is small and the C_{es} values are also small.

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To calculate the pressure heads on the hydraulic network nodes in the transient state, it is used eqn. 10, developed from eqn. 5:

$$\left\{\frac{P}{\rho g}\right\}^{*} = \left[M^{-1}\right]\frac{\{\Delta H\}^{*}\{A\}}{\Delta t} + \left[M^{-1}\right]\left[C^{T}\right]\left\{-\beta\Delta Z - \beta H^{*} + \beta\Delta H_{v}^{*}\right\}, \tag{10}$$

$$+\left[M^{-1}\right]\left[C^{T}\right]\left\{\beta P_{c} - Q^{0}\right\} - \left[M^{-1}\right]\left[C_{es}\right]\left\{\left(\frac{P_{es}}{\rho g}\right)^{0.5}\right\}$$

To calculate the pressure head differences on the hydraulic network branches in the transient state, the connection matrix $[C]$ multiplies the pressure heads calculated in eqn. (10) using eqn. (11):

$$\left\{\frac{\Delta P}{\rho g}\right\}^{*} = [C]\left\{\frac{P}{\rho g}\right\}^{*} \tag{11}$$

To calculate the discharges in the hydraulic network branches in the transient state, the pressure head differences calculated in eqn. (11) are used in eqn. (12):

$$\{Q\}^{*} = \left\{Q^{0}\right\} + \left\{\beta\frac{\Delta P}{\rho g}^{*} + \beta\Delta Z + \beta H^{*} - \beta\Delta H_{v}^{*} - \beta P_{c}\right\}. \tag{12}$$

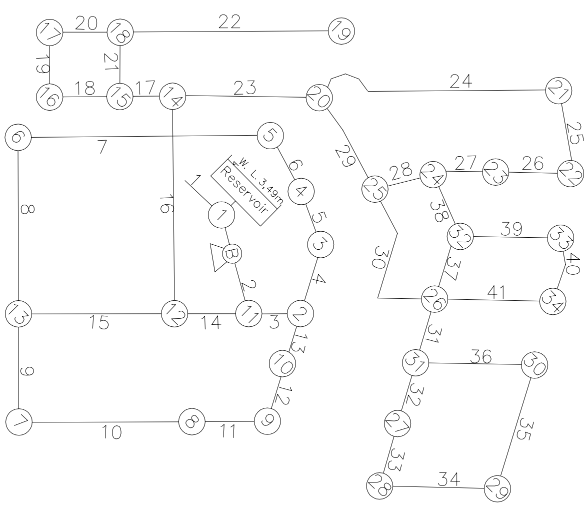

Figure 1: Hydraulic network scheme for the upper part of the neighbourhoods.

4 Results

The non-elastic matrix model has already been tested to calculate many hydraulic networks. Two of these hydraulic networks are located at Paulínia – SP – Brazil. These hydraulic networks supply water to two neighbourhoods of Paulínia. One of them supplies the upper part of the neighbourhoods (fig. 1) and the other one supplies the lower part of the neighbourhoods. It was decided to use the hydraulic networks of Paulínia because the calculated pressure heads of some nodes of the hydraulic network were compared to the pressure heads gauged in situ on the same nodes of the hydraulic network.

Table 1: Input data for the hydraulic network of the upper part of the neighbourhoods for the steady state flow.

node	nodal demand (l/s)	node elevation (m)	pipe	length (m)	diameter (m)
1	-4.77	640.20	1	0.10	0.200
2	0.05	641.17	2	14.38	0.200
3	0.05	640.60	3	32.55	0.200
4	0.07	640.01	4	49.82	0.150
5	0.23	636.47	5	59.29	0.100
6	0.26	624.64	6	112.99	0.075
7	0.22	627.28	7	426.13	0.050
8	0.19	640.79	8	200.00	0.050
9	0.04	641.37	9	100.00	0.050
10	0.04	641.54	10	411.60	0.050
11	0.05	641.06	11	48.28	0.075
12	0.32	640.59	12	58.13	0.100
13	0.30	625.96	13	46.72	0.150
14	0.14	632.68	14	47.22	0.150
15	0.12	632.50	15	411.60	0.050
16	0.11	629.02	16	293.97	0.100
17	0.11	626.39	17	27.06	0.075
18	0.19	628.89	18	188.90	0.050
19	0.08	624.55	19	66.13	0.050
20	0.23	632.74	20	189.01	0.050
21	0.20	623.61	21	65.38	0.050
22	0.14	635.90	22	199.24	0.050
23	0.09	639.48	23	19.00	0.100
24	0.05	640.01	24	311.34	0.050
25	0.18	640.04	25	164.29	0.050
26	0.26	640.90	26	167.62	0.050
27	0.06	640.15	27	36.39	0.075
28	0.12	638.21	28	6.98	0.075
29	0.14	635.06	29	218.31	0.100
30	0.14	630.09	30	208.16	0.075
31	0.14	640.89	31	66.70	0.075
32	0.16	640.54	32	66.72	0.075
33	0.13	630.58	33	66.39	0.050
34	0.14	623.98	34	209.78	0.050
			35	132.74	0.050
			36	209.93	0.050
			37	135.95	0.075
			38	69.47	0.075
			39	179.63	0.050
			40	131.72	0.050
			41	209.85	0.050

Although the comparison between the calculated pressure heads of some nodes and the pressure heads gauged in situ on the same nodes will be shown for both hydraulic networks, it was decided to show the results obtained for the steady state and for the extended period for the hydraulic network of the upper part of the neighbourhoods schemed in fig. 1 for being more complex than the other one.

It is necessary to say the hydraulic network results were calculated for 6 periods and that the absolute roughness of all pipes is 0.1 mm (PVC).

Table 2: Nodal demands to calculate the extended period for the hydraulic network of the upper part of the neighbourhoods.

node	period 1 (l/s)	period 2 (l/s)	period 3 (l/s)	period 4 (l/s)	period 5 (l/s)	period 6 (l/s)
1	-2.39	-5.25	-6.80	-6.44	-5.49	-3.58
2	0.03	0.06	0.08	0.07	0.06	0.04
3	0.02	0.05	0.07	0.06	0.05	0.03
4	0.04	0.08	0.10	0.10	0.08	0.05
5	0.11	0.25	0.32	0.31	0.26	0.17
6	0.13	0.29	0.38	0.36	0.30	0.20
7	0.11	0.24	0.31	0.29	0.25	0.16
8	0.10	0.21	0.28	0.26	0.22	0.15
9	0.02	0.05	0.06	0.06	0.05	0.03
10	0.02	0.05	0.06	0.06	0.05	0.03
11	0.02	0.05	0.07	0.06	0.05	0.03
12	0.16	0.35	0.45	0.43	0.36	0.24
13	0.15	0.33	0.43	0.41	0.35	0.23
14	0.07	0.16	0.20	0.19	0.16	0.11
15	0.06	0.13	0.17	0.16	0.14	0.09
16	0.05	0.12	0.15	0.15	0.12	0.08
17	0.05	0.12	0.15	0.15	0.12	0.08
18	0.10	0.21	0.27	0.26	0.22	0.14
19	0.04	0.09	0.12	0.11	0.10	0.06
20	0.12	0.25	0.33	0.31	0.27	0.17
21	0.10	0.22	0.29	0.27	0.23	0.15
22	0.07	0.15	0.20	0.19	0.16	0.10
23	0.04	0.09	0.12	0.12	0.10	0.06
24	0.02	0.05	0.07	0.06	0.05	0.04
25	0.09	0.20	0.26	0.25	0.21	0.14
26	0.13	0.29	0.37	0.35	0.30	0.20
27	0.03	0.06	0.08	0.08	0.06	0.04
28	0.06	0.13	0.17	0.16	0.13	0.09
29	0.07	0.16	0.21	0.19	0.17	0.11
30	0.07	0.16	0.21	0.20	0.17	0.11
31	0.07	0.16	0.21	0.20	0.17	0.11
32	0.08	0.18	0.23	0.22	0.19	0.12
33	0.07	0.14	0.19	0.18	0.15	0.10
34	0.07	0.16	0.21	0.19	0.17	0.11

Ahead, it is shown in tables 4 and 5 the comparison between the calculated pressure heads of some nodes and the pressure heads gauged in situ on the same nodes of the hydraulic networks of the upper and of the lower part of the neighbourhoods respectively. Due to the hour of the day the pressure heads were gauged in situ, the calculated pressure heads correspond to the nodal demands used in period 3 of the extended period for both hydraulic networks. As already mentioned before, the input data and the results for the hydraulic network of the lower part of the neighbourhoods were not shown in this paper.

Table 3: Obtained discharges for the hydraulic network of the upper part of the neighbourhoods.

pipe	discharge steady state (l/s)	discharge period 1 (l/s)	discharge period 2 (l/s)	discharge period 3 (l/s)	discharge period 4 (l/s)	discharge period 5 (l/s)	discharge period 6 (l/s)
1	-4.77	-2.39	-5.25	-6.80	-6.44	-5.49	-3.58
2	4.77	2.39	5.25	6.80	6.44	5.49	3.58
3	1.20	0.60	1.32	1.71	1.62	1.38	0.90
4	0.60	0.30	0.66	0.85	0.80	0.69	0.45
5	0.55	0.28	0.60	0.78	0.74	0.63	0.41
6	0.48	0.24	0.52	0.68	0.64	0.55	0.36
7	0.25	0.13	0.27	0.36	0.34	0.29	0.19
8	-0.01	-0.01	-0.02	-0.02	-0.02	-0.02	-0.01
9	-0.05	-0.02	-0.05	-0.07	-0.07	-0.06	-0.04
10	-0.26	-0.13	-0.29	-0.38	-0.36	-0.30	-0.20
11	-0.46	-0.23	-0.50	-0.65	-0.62	-0.53	-0.34
12	-0.50	-0.25	-0.55	-0.72	-0.68	-0.58	-0.38
13	-0.55	-0.27	-0.60	-0.78	-0.74	-0.63	-0.41
14	3.53	1.76	3.88	5.03	4.76	4.06	2.65
15	0.27	0.13	0.29	0.38	0.36	0.31	0.20
16	2.95	1.47	3.24	4.20	3.98	3.39	2.21
17	0.61	0.30	0.67	0.87	0.82	0.70	0.46
18	0.17	0.10	0.19	0.25	0.24	0.20	0.13
19	0.07	0.04	0.07	0.09	0.09	0.08	0.05
20	-0.04	-0.01	-0.05	-0.06	-0.05	-0.05	-0.03
21	-0.32	-0.15	-0.35	-0.45	-0.43	-0.36	-0.23
22	0.08	0.04	0.09	0.12	0.11	0.10	0.06
23	2.19	1.10	2.41	3.13	2.96	2.52	1.65
24	0.24	0.12	0.26	0.34	0.32	0.27	0.17
25	0.04	0.02	0.04	0.06	0.05	0.04	0.02
26	-0.10	-0.05	-0.11	-0.14	-0.14	-0.12	-0.09
27	-0.19	-0.10	-0.21	-0.27	-0.26	-0.22	-0.15
28	-0.98	-0.49	-1.07	-1.39	-1.32	-1.13	-0.74
29	-1.73	-0.87	-1.90	-2.45	-2.33	-1.99	-1.30
30	0.57	0.28	0.62	0.80	0.76	0.65	0.43
31	0.61	0.30	0.67	0.86	0.82	0.70	0.45
32	0.28	0.14	0.31	0.40	0.38	0.33	0.22
33	0.23	0.11	0.25	0.32	0.31	0.26	0.17
34	0.11	0.05	0.12	0.16	0.15	0.13	0.09
35	-0.03	-0.02	-0.04	-0.05	-0.04	-0.04	-0.02
36	-0.18	-0.09	-0.20	-0.25	-0.24	-0.20	-0.13
37	-0.41	-0.21	-0.45	-0.58	-0.55	-0.47	-0.31
38	-0.74	-0.37	-0.81	-1.06	-1.00	-0.85	-0.55
39	0.17	0.08	0.19	0.24	0.23	0.19	0.12
40	0.04	0.02	0.04	0.05	0.05	0.04	0.02
41	-0.11	-0.05	-0.12	-0.15	-0.14	-0.13	-0.09

Table 4: Comparison between the calculated pressure heads and the gauged pressure heads of the network of the upper part of the neighbourhoods.

node	7	16	28	29	31
calculated pressure heads (mH_2O)	31.43	29.06	19.41	22.50	16.81
gauged pressure heads (mH_2O)	30.00	29.00	18.00	20.00	16.00

Table 5: Comparison between the calculated pressure heads and the gauged pressure heads of the network of the lower part of the neighbourhoods.

node	4	7	8	15
calculated pressure heads (mH_2O)	28.64	48.00	31.97	42.25
gauged pressure heads (mH_2O)	25.00	46.00	28.00	38.00

5 Conclusions

The matrix method is working well, because the values calculated by the model for the hydraulic networks used as examples are similar to the ones obtained from the technical literature. The model was applied to calculate two real hydraulic networks. The calculated pressure heads of some nodes of the two hydraulic networks were compared to the pressure heads gauged in situ on the same nodes of the hydraulic networks and the results were close. The applicability of the method created by Nahavandi and Catanzaro [1] was enhanced, because the programming and the input data to consider the presence of valves, reservoirs or boosters in the hydraulic network were developed. Furthermore, the mathematical formulation and the programming to calculate the extended period and the transient state also were developed. The input data entry is easier, because in the method created by Nahavandi and Catanzaro [1] the user had to build himself the connection matrix and now the software itself builds the connection matrix. It was developed a technique to calculate the pressure heads on the nodes of the hydraulic networks (if being designed) in order to make the pressure heads stay below the maximum pressure head limit (input data) admitted by technical norms of the countries.

Symbol list

$\{DP/rg\}$ pressure head differences on the branches (m)
$\{P/rg\}$ pressure heads on the nodes (m)
$[C^T]$ transposed matrix of the connection matrix
$\{Q\}$ discharges of the branches (m^3/s)
$\{Q_{es}\}$ nodal demands (m^3/s)
$\{r\}$ specific mass of the fluid (kg/m^3)
$\{D\}$ branch diameter (m)
$\{L\}$ branch length (m)
$\{v^0\}$ present velocity (m/s)
$\{v\}$ future velocity (after Δt) (m/s)
$\{Dt\}$ time gap calculated by Courant's condition (s)
$\{DP\}$ pressure difference on the branch (m)
$\{g\}$ gravity acceleration (m/s^2)
$\{DZ\}$ elevation difference between the nodes that limit a branch (m)
$\{H\}$ manometric head of an installed booster in a branch (m)
$\{DH_v\}$ head loss of an installed valve in a branch (m)

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$\{F_{av}\}$ viscous friction force (N)
$\{Q^0\}$ present discharges (m^3/s)
$[b]$ diagonal matrix defined as: $[b] = [(pD^2gDt)/(4L)]$
$[M]$ square matrix defined as: $[M]=[C^T]*[b]*[C]$
$\{P_c\}$ head loss on a branch (m)
$\{DH\}$ water level differences of the reservoirs (m)
$\{Q_e\}$ filling discharges of the reservoirs (m^3/s)
$\{Q_s\}$ depletion discharges of the reservoirs (m^3/s)
$\{A\}$ reservoir base surfaces (m^2)
$\{(P/rg)^{0.5}\}$ square roots of the pressure heads on the nodes for the steady state (m$^{0.5}$)
$\{P/rg\}^*$ pressure heads on the nodes for the transient state (m)
$\{DH\}^*$ water level differences of the reservoirs for the transient state (m)
$\{H\}^*$ manometric head of an installed booster in a branch for the transient state (m)
$\{DH_v\}^*$ head loss of an installed valve in a branch for the transient state (m)
$\{(P_{es}/rg)^{0.5}\}$ square roots of the pressure heads on the nodes for the transient state (m$^{0.5}$)
$\{DP/rg\}^*$ pressure head differences on the branches for the transient state (m)
$\{Q\}^*$ discharges of the branches for the transient state (m^3/s).

References

[1] Navahandi, A.N. & Catanzaro, G.V., Matrix method for analysis of hydraulic networks. *Journal of Hydraulics Division*, **99(HY1),** pp.47-63, 1973.

[2] Ormsbee, L.E. & Wood, D.J., Hydraulic design algorithms for pipe networks. *Journal of Hydraulic Engineering*, **112(12),** pp.1195–1207, 1986.

[3] Jowitt, P.W. & Xu, C., Optimal valve control in water-distribution networks. *Journal of Water Resources Planning and Management*, **116(4)**, pp.455-472, 1990.

[4] User's handbook of EPANET 2 (PDF format) in Portuguese, http://www.dha.lnec.pt/nes/epanet/downloads/EN2Pmanual.pdf

[5] Vairavamoorthy, K. & Lumbers, J., Leakage reduction in water distribution systems: optimal valve control. *Journal of Hydraulic Engineering*, **124(11),** pp.1146-1154, 1998.

[6] Filion, Y.R. & Karney, B.W., Extended-period analysis with a transient model. *Journal of Hydraulic Engineering*, **128(6),** pp.616-624, 2002.

[7] Goulter, I.C., Systems analysis in water-distribution network design: from theory to practice. *Journal of Water Resources Planning and Management*, **118(3),** pp.238-248, 1992.

[8] Mpesha, W., Gassman, S.L. & Chaudhry, M.H., Leak detection in pipes by frequency response method. *Journal of Hydraulic Engineering*, **127(2),** pp.134-147, 2001.

[9] Wood, Charles *Journal of the Hydraulics Division*, 98(HY7), pp. 1157-1170, 1972.

www.witpress.com, ISSN 1743-3541 (on-line)

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Siting and sizing the components of a regional wastewater system: a multiobjective approach

J. A. Zeferino, M. da Conceição Cunha & A. P. Antunes
Department of Civil Engineering, Coimbra University, Portugal

Abstract

This paper describes a multiobjective approach for the siting and sizing of the components of a regional wastewater system. This approach can be particularly helpful for the coherent and harmonious implementation of the Water Framework Directive. Three criteria are considered for finding efficient solutions. A simulated annealing algorithm improved by a local search algorithm is used and the results of three case studies are presented and compared.
Keywords: wastewater systems, multiobjective models, simulated annealing.

1 Introduction

The worldwide concern about water and sanitation has been expressed in initiatives like the United Nations Millennium Development Goals. The target to reduce by half the population without sustainable access to safe drinking water and basic sanitation is incorporated in the goal to ensure environmental sustainability. If this target is to be met appropriate wastewater systems have to be implemented. These systems are often designed at a local level. However, better solutions both from the economic and the environmental points of view can be obtained with regional planning.

This work describes a multiobjective approach to regional wastewater system planning. In this type of system, global cost is usually the criterion that is optimized, making it a single-objective problem. The other indicators for achieving a sustainable development are often included as problem constraints, considering some upper and lower limits. However, for some indicators, these limits may be difficult to establish, which can make them easier to handle as criteria. Since the criteria to be optimized are usually incommensurable, it is impossible to find a solution where all these criteria are optimized

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simultaneously, given that the improvement of one results in the deterioration of another. This means that no optimal solution can be found. Still, there are efficient solutions that can be reached, these are non-dominated solutions also known as Pareto solutions.

2 Literature review

The first studies on wastewater system planning were carried out in the 1960s. The great majority of these studies tackle the waste load allocation (WLA) problem, which comprises the determination of the required pollutant removal level at a number of point sources of a stream (Loucks *et al* [15], Katapodes and Piasecki [10]). Another type of wastewater planning problem is the siting and sizing of the different components of a regional wastewater system (Leighton and Shoemaker [13], Tyteca [18]). Improved computational capacities and optimization methods made it possible to solve complex models incorporating all the features occurring in the cost minimization of siting and sizing the different components of a regional wastewater system (Wang and Jamieson [20]). Sousa *et al.* [16] presented a model and the respective computational application to solve this type of problem, called Regional Wastewater Systems Planning (RWSP). The water quality in the receiving stream was taken into account by the introduction of constraints for the maximum wastewater discharge in each treatment plant. In Cunha *et al.* [7], the RWSP model was improved by incorporating a water quality model to allow the determination of water quality parameters after discharge from wastewater treatment plants. This improvement enables the model to guarantee the water quality in the river, besides optimizing the network system design.

Other indicators that could be considered as criteria apart from the minimum cost configuration, particularly for WLA problems, came later: Bishop *et al.* [2] and Lohani and Adulbhan [14] attempted to minimize the deviation of the water quality goals and Tung [17] proposed new indicators in order to optimize four different criteria. These criteria were the maximization of the total waste load allocation, the maximization of the dissolved oxygen (DO) in the stream, the minimization of the equity measure between the various dischargers and the minimization of the major risk of breaching the water quality standards. With the aim of reaching efficient solutions considering the uncertainties, Burn and Lence [4] formulated models for the minimization of different criteria corresponding to a deviation measure of the DO levels. Cardwell and Ellis [5] used a criterion consisting of the number of violations of the DO standards, which was minimized together with the cost of the system. In Lee and Wen [12] the objective of maximization of the assimilative capacity in a multiobjective approach was introduced for the first time. The same work presented a list of previous studies, with the criteria used, and also showing the tendency for change from single-objective to multiobjective approaches. Multiobjective decision analysis under uncertainty has been proposed by Chang *et al.* [6] to solve potential conflicts between safeguarding water quality and economic development. Based on the concept of sustainability, Balkema *et al.* [1] defined three dimensions for multiobjectivity: economic, environmental and socio-cultural. They thus indicated different criteria capable of being optimized, such as energy use, land use, nutrient loss, waste production and social acceptance. Since there are various criteria that can be optimized, a multiobjective approach brought important advantages to the analysis. In water resources problems, Burn and Yulanti [3] were the first to use a genetic algorithm in order to find a Pareto set of solutions in a three-objective problem (the objectives were the balance between the various dischargers, the cost of the system, the water quality in the receiving stream, expressed by the number of standard DO violations). More recently, the non-dominated sorting algorithm II was used by Yandamuri *et al* [21]. Ghosh and Mujumdar [8] used a fuzzy multiobjective model for minimizing the risk in a river water quality management problem. A new global search algorithm developed recently, the Probabilistic Global Search Lausanne, was used to solve the model. Jia and Culver [9] applied a robust genetic algorithm to total maximum daily load allocations.

This paper follows previous work by the authors. It describes a multiobjective approach used to solve the siting and the sizing of the different components of a regional wastewater system. The implementation of this approach makes use of the decision-aid model presented in Cunha *et al.* [7].The simulated annealing algorithm (SA) described in Sousa *et al.* [16] improved by a local search algorithm and including the parameters calibrated by Zeferino *et al.* [22] is used.

3 Multiobjective approach

Multiobjective analysis consists either of the generation of solutions from an infinite number of alternatives, using systematic methods, or of the selection of a solution from a finite set of alternatives, also known as multiattribute analysis, using outranking methods. Since there are an immensurable number of alternatives in wastewater system planning, the solution has to be found by a multiobjective analysis based on the generation of solutions.

Three objectives for planning the wastewater system were established for this study. These objectives match the indicators that usually need to be optimized: the minimization of the capital cost of the system (Ci); the minimization of the operating cost (Ce); and the maximization of dissolved oxygen in the river (DO).

The first indicator is related to the initial investment in the wastewater system, and includes equipment and construction costs. The second concerns the cost incurred during the lifetime of the system, consisting of the recurrent costs of the facilities and the equipment, including energy costs. These operating costs are also related to the initial cost. The last indicator is related to the water quality in the river, measured in dissolved oxygen, since this is one of the most important indicators of water quality.

The approach most often used to solve models with more than one objective is based on the utility theory, turning multiple objective problems into a single objective problem prior to optimization. This is done by means of a weighted summation of the individual objectives. But it would be useful to the decision-maker if there were a set of non-dominated solutions that would allow him to note the trade-offs between the objectives when deciding on a solution. Non-dominated solutions are also called Pareto solutions. This set of solutions represents the frontier with the best solutions that can be achieved. This happens because no enhancements can be found, since the improvement of one objective result in the deterioration of another.

Following the weighted summation approach and considering the objectives previously defined, the objective function will be:

$$\text{Minimize F} = \sum_{i=1}^{3} w_i \times \tilde{f}_i\left(x_j\right) \tag{1}$$

where F: aggregate objective function; w_i: weighting values; $\tilde{f}_i\left(x_j\right)$: normalized criteria to optimize.

Since the three objectives correspond to different units with variations of different magnitudes, their scores are standardized (2). This standardization makes the objectives dimensionless, while transforming the value of the objective to a proportion contained in the interval between the lowest and highest score.

$$\tilde{f}_i(x) = \frac{f_i(x) - f_i^{min}}{f_i^{max} - f_i^{min}} \tag{2}$$

The weights w_i set the priorities for the decision criteria, indicating the relative importance of each objective. An accurate distribution of weights is one of the bigger challenges in a multiobjective optimization. This usually requires a specific process involving different stakeholders (Lahdelma *et al.* [11]). However, the process of finding the right weights to attribute is not within the scope of this work. The weights must be strictly positive for at least one objective, and have a total sum equal to one.

The single-function F (3) to be minimized is thus expressed by the sum of the weights multiplied by the standardized criteria, giving the following expression:

$$F = W_{Ci} \times \frac{\left(Ci - Ci^{min}\right)}{\left(Ci^{max} - Ci^{min}\right)} + W_{Ce} \times \frac{\left(Ce - Ce^{min}\right)}{\left(Ce^{max} - Ce^{min}\right)} + W_{DO} \times \frac{\left(DO^{max} - DO\right)}{\left(DO^{max} - DO^{min}\right)} \tag{3}$$

where *Ci*: capital cost; *Ce*: operating cost; *DO*: minimum value of the Dissolved Oxygen observed in the river. W_{Ci}, W_{Ce} and W_{DO}: weights.

The variables with superscripts in equation (3) correspond to the maximum and minimum values. The Ci^{min}, Ce^{min} and DO^{max} are obtained from the respective minimization and maximization functions. The other extreme values of these indicators are removed from the worst results obtained for those indicators in the other optimizations.

4 Case studies

The study used three different cases, corresponding to 3 test problems. These problems try to correspond to real-world problems, comprising similar characteristics (Figure 1). They were defined according to rules regarding shape and topography, location and size of population centers, the wastewater generation rate and location and maximum discharge at treatment plants. The implementation method for this can be found in Zeferino *et al* [22]). The three cases selected have different characteristics, in particular concerning the values of the ridges' orientations.

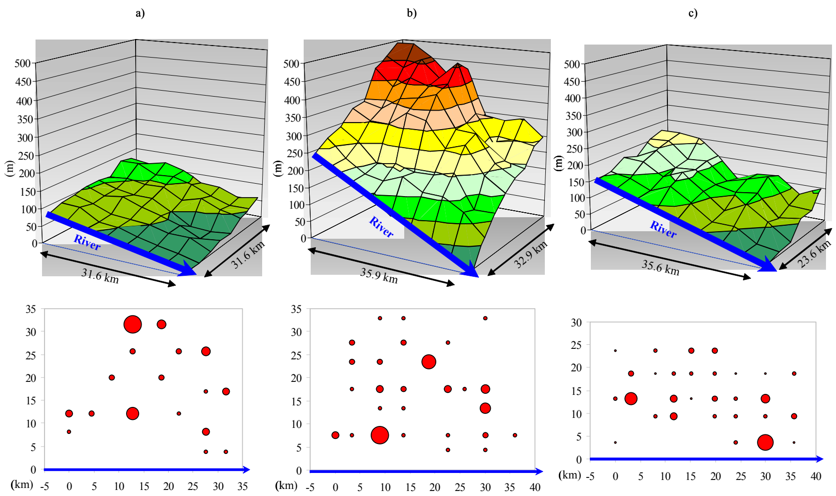

Figure 1: Shape, topography and location of the urban centers of the three regions used for this study.

The first phase of the implementation of the multiobjective approach was to determine the extreme values of the three criteria. This was done using three single objective functions: minimize *Ci*; minimize *Ce*; maximize *DO*. The SA algorithm requires the use of accurate parameters, essential for finding good quality solutions (Sousa *et al.* [16]). For the three cases presented, the four SA parameters (α: sets the initial acceptance rate for candidate solutions with value 10% smaller than the value of the incumbent solution, λ: sets the minimum number of candidate solutions that must be evaluated at each temperature, γ: sets the rate at which the temperature decreases, and σ: sets the maximum number of temperature decreases that may occur without an improvement of the best or the average solution value) were calibrated by the authors in previous work (Zeferino *et al* [22]). They are, for case a): $\alpha = 0.599$, $\lambda = 49$, $\gamma = 0.500$ and $\sigma = 13$; case b): $\alpha = 0.497$, $\lambda = 56$, $\gamma = 0.575$ and $\sigma = 12$; case c): $\alpha = 0.308$, $\lambda = 52$, $\gamma = 0.696$ and $\sigma = 12$. As SA is a random search algorithm, 10 different seeds were used for the pseudo-random generator for each of the three cases. The results obtained are given in Table 1, with *Ci* and *Ce* in M€ and *DO* in mg/l. The results for minimum *Ci* (Ci^{min}), minimum *Ce* (Ce^{min}) and maximum *DO* (DO^{max}) are obtained from each line of each case matrix. Note that all of these values match the diagonal of the matrix. This was expected, since the diagonal corresponds to the values achieved respectively in the minimization and maximization processes. The results for maximum *Ci* (Ci^{max}), maximum *Ce* (Ce^{max}) and minimum *DO* (DO^{min}) are obtained in the same way, and are given by the corresponding maximum or minimum values of each line. The process of defining the proper distribution of weights is not an objective of this work. As mentioned before, different combinations of weights were selected for this study. Once the results for these combinations are obtained, a small set of Pareto solutions is achieved, in order to relate the trade-off between the different criteria. The set of Pareto-optimal solutions makes it possible to see how the solutions change when given different weights. For this study 4 combinations of weights were chosen (Table 2).

Table 1: Results for the extremes.

a)	min Ci	min Ce	max DO
Ci	23,23	24,60	37,63
Ce	0,75	0,73	1,03
OD	6,088	6,108	6,175

b)	min Ci	min Ce	max DO
Ci	29,25	31,57	55,01
Ce	1,20	1,13	1,91
OD	5,800	5,849	5,939

c)	min Ci	min Ce	max DO
Ci	37,16	37,81	56,85
Ce	1,61	1,55	2,00
OD	5,861	5,860	5,923

Table 2: Combinations of weights used.

	W_{Ci}	W_{Ce}	W_{DO}
Combination 1	0,33(3)	0,33(3)	0,33(3)
Combination 2	0,60	0,20	0,20
Combination 3	0,20	0,60	0,20
Combination 4	0,20	0,20	0,60

The solutions generated can also be used to give a set of alternatives that would help with a complementary decision-making aid. This can be done using another multicriteria optimization, based on the selection of a solution from a limited number of alternatives (Vincke [19]). This posterior analysis is not within the scope of this study.

5 Multiobjective results

Once the extremes of each indicator were determined and the different combinations for the weights established, the multiobjective model was solved. This was done for the three cases presented, using 10 different seeds. The parameters used in each case were the same as those employed before in the evaluation of the extreme values. The results for each case are presented in Table 3. The tables on the left correspond to the three cases studied. Each contains the best values of the criteria for each combination. The tables on the right present the results for the respective standardized indicators ($\hat{C}i$, $\hat{C}e$ and $D\hat{O}$), where 0% corresponds to the best value of the criteria and 100% to the worst value of the criteria. The extreme values (0% and 100%) were obtained earlier in this work. The summation of these values multiplied by the weight of the respective combination gives the value of the function F that was minimized.

Table 3: Payoff results of the multiple objective problem. Left: Values of the criteria for the three cases; Right: Standardized criteria and F values.

a)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ci	26,078	23,670	23,790	26,896
Ce	0,777	0,744	0,739	0,792
DO	6,1697	6,1250	6,1250	6,1744

a)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ĉi	19,8%	3,0%	3,9%	25,4%
Ĉe	14,5%	3,6%	2,0%	19,6%
DÔ	6,60%	57,49%	57,49%	1,23%
F	0,136	0,140	0,134	0,098

b)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ci	33,265	33,236	33,290	34,956
Ce	1,185	1,186	1,184	1,211
DO	5,9249	5,9249	5,9249	5,9384

b)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ĉi	15,6%	15,5%	15,7%	22,1%
Ĉe	7,3%	7,4%	7,2%	10,7%
DÔ	10,29%	10,29%	10,29%	0,61%
F	0,111	0,128	0,095	0,069

c)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ci	40,052	40,052	40,402	40,482
Ce	1,576	1,576	1,573	1,590
DO	5,9210	5,9210	5,9210	5,9224

c)	Comb. 1	Comb. 2	Comb. 3	Comb. 4
Ĉi	14,7%	14,7%	16,5%	16,9%
Ĉe	5,8%	5,8%	5,0%	8,8%
DÔ	2,70%	2,70%	2,70%	0,47%
F	0,077	0,105	0,068	0,054

The analysis of the results in each case shows that, once again as expected, the minimum value of the normalized indicators appears in the combination that sets highest weight for the respective indicator.

In relation to the trade-offs between the criteria, the first two indicators, *Ci* and *Ce*, seem to be clearly incommensurable with *DO*. For all the cases studied, the best value of *DO* results in the worst solution for the other indicators. Relating to the indicators *Ci* and *Ce*, the only observation is that it was not possible to find a solution where both were minimized at the same time. Despite the trade-off between these indicators being only slight, this probably means that they are also incommensurable. Regarding the results for F, the minimum value obtained was always in combination 4, that is, when more weight is given to the *DO*. This indicates that it is easier to find solutions where the maximization of the *DO* is near the optimum, thus having suitable values for the other indicators at the same time.

The analysis of how the solutions physically change according to the different combinations of weights is also possible. Figure 2 gives some results of case b), showing the changes that occur in the solutions along with the increase of W_{DO}. The analysis of the three images in Figure 2 clearly shows how the solutions adapt as more weight is given to one criteria, in this case, the maximization of *DO*. In the top left figure, the W_{DO} is only 33.3(3)%, that is, the same as that given to *Ci* and *Ce*. In the top right figure, corresponding to a W_{DO} = 60%, the solution changes through setting one water treatment plant in the first node, in order to improve the *DO* in the river. However, since W_{Ci} = 20% and W_{Ce} = 20%, the solution still considers some aspects for minimizing costs. The figure at the bottom shows a solution where there is no concern with the cost, since it corresponds to the maximization of *DO*. This is equivalent to having a W_{DO} = 100%. As can be seen, the solution is quite unusual, given that it only concerns the wastewater flow that is discharged in each water treatment plant.

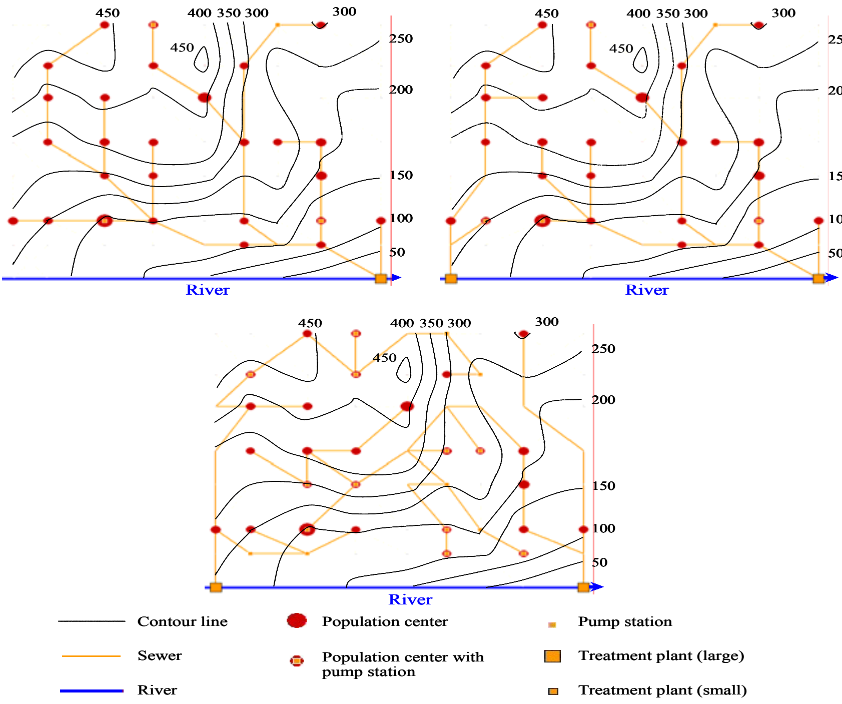

Figure 2: Top left: combination 1; top right: combination 4; bottom: DO maximization.

6 Conclusions

A multiobjective approach has been presented for the siting and sizing of the components of a regional wastewater system. A weighted summation method has been applied to find efficient solutions. The results obtained for three different case studies made it possible to analyse the solutions according to the importance given to each criterion. A set of alternatives was also generated, which helps to support decision-making.

In future work, this multiobjective approach might seek to find a large set of Pareto solutions, showing the best trade-offs between the criteria. More criteria can also be used, to give broader coverage of the objectives involved in regional wastewater system planning.

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References

[1] Balkema, A.J., Preisig, H.A., Otterpohl, R. & Lambert, F.J.D., Indicators for the sustainability assessment of wastewater treatment systems, Urban Water, 4, pp. 153–161, 2002.

[2] Bishop, A.B., Pugner, P.E., Grenney, W.J. & Narayanan, R., Goal Programming Model for Water Quality Planning, Journal of the Sanitary Engineering Division, 103 (2), pp. 293-306, 1977.

[3] Burn, B.D.H. & Yulianti, J.S., Waste-Load Allocation Using Genetic Algorithms, Journal of Water Resources Planning and Management, 127 (2), pp. 0121–0129, 2001.

[4] Burn, D.H. & Lence, B.J., Comparison of Optimization Formulations for Waste-Load Allocations, Journal of Environmental Engineering (ASCE) JOEEDU, 118 (4), pp. 597-612, 1992.

[5] Cardwell, H. & Ellis, H., Stochastic Dynamic Programming Models for Water Quality Management, Water Resources Research WRERAQ, 29 (4), pp. 803-813, 1993.

[6] Chang, N.-B., Chen, H.W., Shaw, D.G. & Yang, C.H., Water Pollution Control in River Basin by Interactive Fuzzy Interval Multiobjective Programming, Journal of Environmental Engineering, 123 (2), pp. 1208-1216, 1997

[7] Cunha, M.C., Pinheiro, L., Afonso, P. & Sousa, J., Decision-Aid Models for the Implementation of the Water Framework Directive, 4th International Conference on Decision Making in Urban and Civil Engineering (DMUCE), Porto (Portugal), 28-30 October, 2004.

[8] Ghosh, S. & Mujumdar, P.P., Risk Minimization in Water Quality Control Problems of a River System, Advances in Water Resources, 29, pp. 458-470, 2005

[9] Jia, Y. & Culver, T.B., Robust Optimization for Total Maximum Daily Load Allocations, Water Resources Research, 42 (2), W02412, 2006

[10] Katopodes, N.D. & Piasecki, M., Site and Size Optimization of Contaminant Sources in Surface Water Systems, Journal of Environmental Engineering, 122 (10), 917-923, 1996

[11] Lahdelma, R., Salminen, P. & Hokkanen, J., Using Multicriteria Methods in Environmental Planning and Management, Environmental Management, 26 (No. 6), pp. 595–605, 2000.

[12] Lee, C.-S. & Wen, C.-G., Application of Multiobjective Programming to Water Quality Management in a River Basin, Journal of Environmental Management, 47, pp. 11–26, 1996.

[13] Leighton, J.P. & Shoemaker, C.A., An integer programming analysis of the regionalization of large wastewater treatment and collection systems, Water Resources Research, 20 (6), pp. 671-681, 1984.

[14] Lohani, B.N. & Adulbhan, P., A Multiobjective Model for Regional Water Quality Management, Water Resources Bulletin, 15 (4), pp. 1028-1038, 1979.

[15] Loucks, D.P., Revelle, C.S. & Lynn, W.R., Linear Programming Models for Water Pollution Control, Management Science, 14 (4), pp. B166-B181, 1967.

[16] Sousa, J., Ribeiro, A., Cunha, M.C. & Antunes, A., An Optimization Approach to Wastewater Systems Planning at Regional Level, Journal of Hydriunformatics, 4 (2), pp. 15-123, 2002.

[17] Tung, Y.K., Multiple-Objective Stochastic Waste Load Allocation. Technical Completion Report to the U.S. Geological Survey (USGS G1459-07): 36 pp, 1988.

[18] Tyteca, D., Optimisation du dimensionnement et de la localisation des équipements de collecte et d'épuration d'eaux résiduaires. In Gestion de l'économie et de l'entreprise - L'approche quantitative, CORE (ed.), De Boeck. Université, Bruxelles, Coll. Ouvertures Économiques, Série Balises. chap.6: pp 427-453, 1988.

[19] Vincke, P., Gassner, M. & Roy, B., Multicriteria Decision-Aid. John Wiley & Sons, New York., 1992.

[20] Wang, C.G. & Jamieson, D.G., An Objective Approach to Regional Wastewater Treatment Planning, Water Resources Research, 38 (3), 2002

[21] Yandamuri., S.R. M., Srinivasan, K. & Bhallamudi, S.M., Multiobjective Optimal Waste Load Allocation Models for Rivers Using Nondominated Sorting Genetic Algorithm-II, Journal of Water Resources Planning and Management, 132 (3), pp. 133–143, 2006.

[22] Zeferino, J., Cunha, M.C. & Antunes, A., An Efficient Simulated Annealing Algorithm for Regional Wastewater Systems Planning, In 11th International Conference on Computing in Civil and Building Engineering, Montreal (Canada), 11-16 June, 2006.

Evaluation of the surface water quality in the Itapicuru river basin – State of Bahia, Brazil

S. S. P. Mestrinho[1], D. Reitemajer[2], D. C. Porciúncula[2] & R. Lyrio[2]
[1]*Superintendence of Researches and Graduate Studies, UCSAL, Brazil*
[2]*Associated Consultants, Brazil*

Abstract

The Itapicuru River basin is located in the northeast of the State of Bahia, Brazil. It is a large basin with variable conditions in terms of soil, climate, topography, natural vegetation, and economy. About 80% of the total area is subjected to severe drought periods. Poor cropping systems and lack of adequate systems for disposing wastewater from the urban areas determine the water quality along the basin. This proposal aims to carry out a study on the actual situation of the surface water resources, including diagnostic and prognostic actions. The study presents a spatial analysis of the surface water quality using hydro chemical data of the water and sediments from 30 stations along the river and the tributaries, including points in the reservoirs of the dam and estuary region. The main processes related with the water quality are: erosion, concentrations of salt and nutrients, punctual contamination of metals and bacteria. The spatial distribution of the parameters in the different points is illustrated in maps. Zones showing higher values of coliforms, nitrogen, phosphor, chloride, conductivity and nitrates are in the high and medium part of the river, where there is low rainfall, higher demographic density and expressive social-economic dynamics. Closer to the chromium mines the concentration of iron and chromium in the sediments is above the limits. The results are associated with the environmental quality in order to have a characterization of the hydro chemical and social-economic elements, in different scales. Also critical areas are defined in detailed studies to help implement corrective actions to minimize or eliminate the contamination problems. It is expected that the results and achievements of this project will be useful in order to support the development of state and federal programs on the sustainability of water resources in the region.
Keywords: water quality, environmental quality, Itapicuru river basin.

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1 Introduction

The Itapicuru river basin occupies a total area of 36,440 km^2, with an estimated population of 1,300,000 inhabitants. The environmental impact associated to the farming, mining, industrial, tourist activities and the disordered growth of the cities with inadequate infrastructure of sewers threaten the water quality. The basin is subdivided into four sectors with differentiated characteristics, as shown in table 1.

Table 1: Characteristics of the main sectors of the Itapicuru River Basin (SRH [1], PRODETAB [2]).

	Upper Sector (I)	**Middle Upper Sector (II)**	**Middle Lower Sector (III)**	**Lower Sector (IV)**
Annual Mean Rainfall	727.61 mm	500.29 mm	682.27mm	1,182.71 mm
Area	11,968.97 km²	10,106.57 km²	12,232.01 km²	2,131.45 km²
Hydrological Environment	High declivity terrain. The water from the rock fractures influences the permanent flow of the river. Many dams.	Crystalline rocks with discrete hydro potentiality. More saline waters. Rivers flow in the rainy period.	Ground that favors infiltration. Rivers are intermittent. The underground hydro potential is high.	Deeper ground. The greatest rainfall rates assures the permanent flow of the small and medium sized rivers
Physical Vulnerability	High and medium. Risk of erosion	Medium. High on some points.	Low. Medium and high on some points.	High. Risk of flooding.

The basin's volume of average annual precipitation is estimated to be 24,631.38 x 10^{6}m^3 and the total flow 1,269.96 x 10^{6}m^3. The superficial hydro potentiality is considered low (0.76 l/s/km^2). Around 80% of the area is in the conditions of semi-arid climate. The fluvial regimen in the sectors is a reflex of the regional variation of rainfall: from the middle lower sector of the river to the lower the flow is permanent; otherwise, at the upper and the middle upper sectors the river and its tributaries are intermittent. The state of hydro deficit has stimulated the construction of dams and the perforation of wells, to increase the availability of water in the period of greater demand.

Regionally, we don't see a control in the use of the ground and of the water and situations of aggression to the environment can be noticed, like: indiscriminant removal of the vegetation coverage; the absence of or deficient control of the activities of mining and of the launching of solid and liquid residues in the water courses; and the construction of dams without the correct management or prior study.

This work discusses the results of studies for the environmental diagnosis associated to the quality of the basin's superficial water, with the objective of guiding planning programs of its water resources and sustainable development. The research integrates the Investigation Project PRODETAB/EMBRAPA 055-01/01, under the auspices of the World Bank.

2 Methodological aspects

The Itapicuru River is formed mainly by the confluence of the Itapicuru, Itapicuru Açu and Itapicuru Mirim rivers, and the contribution of several tributaries. To characterize the quality of the water, was established a sample grid of 30 points for water collection and 20 points for sediments. The territorial extent of the basin is large, and although the number of points is limited, the sample contemplates the sections of the main course of the river and its tributaries (R1, R2,..., R23), including the reservoirs (B1, B2,..., B7). Two stages of fieldwork were carried out in distinct seasons: August 2005 (greater indices of rainfall) and January of 2006 (less rainfall). The standards recommended by APHA [3] were used as techniques for the samples collection and analyses. For each point the following variables were investigated: conductivity, chlorides, total nitrogen and phosphor, nitrates, thermo-tolerant coliforms, Trophic State Index – TSI introduced by Carlson [4], Water Quality Index – WQI National Sanitation Foundation - U.S.A., and the level of metal in the sediments.

In order to illustrate the information obtained, geoprocessing techniques were used to permit a spatial reading of the territory using maps in the regional scale (1:1,000,000). The final approach includes an analysis of the environmental quality of the basin, integrating other specifics information available in the scope of the Project PRODETAB/EMPRAPA.

3 Results and discussion

The basin of the river Itapicuru covers diverse climatic, geologic, geomorphologic environments and distinct ecosystems. The comprehension of the social-environmental dynamics, which influences the quality of the water resources, demands the integration of the variable information. As a methodological basis for the construction of the basin's diagnostics and prognostics associated to the quality of the superficial water, the physical

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vulnerability of the units of landscape were evaluated and integrated to the aspects of socioeconomic dynamics, including the concession of rights to its use. The results are discussed as follows.

3.1 Evaluation of the water quality

The largest concentration of metals found in the water was of manganese (0.06-13.42 mg/l), iron (>0.3 mg/l) and chromium in the points R20 (0.05 mg/l) and B3 (0.08 mg/l).

The hydro chemical parameters at each study point, fig. 1, show that, in general, the tributaries have water with less quality. Improper values of thermo-tolerant coliforms appear in the stations: R15 (5320 col/100mL), close to the sewers of the city of Queimadas; R18 (13600 col/100ml), downstream of the city of Jacobina; R22 (3200 col/100mL), a river tributary that receives effluents from Campo Formoso; and B7 (6300 col/100mL), at the Cariacá dam. The total nitrogen proportions are higher close to populated areas (>25,000 inhabitants), as in the cities of Tucano, Jacobina and Senhor do Bonfim.

The results suggest that the water quality reflects the natural and anthropogenic conditions. The main alterations can be related to the following processes:

- Siltation: the average depth varies in the stations of the river between periods 1 (0.8m) and 2 (0.4m) of the collection sample;
- Eutrophication: the water has an elevated concentration of total phosphor (0.07– 23.95 mg/l), the highest values (>1500 mg/l) are in the points closest to the cities of greater population of sector I (Senhor do Bonfim and Jacobina);

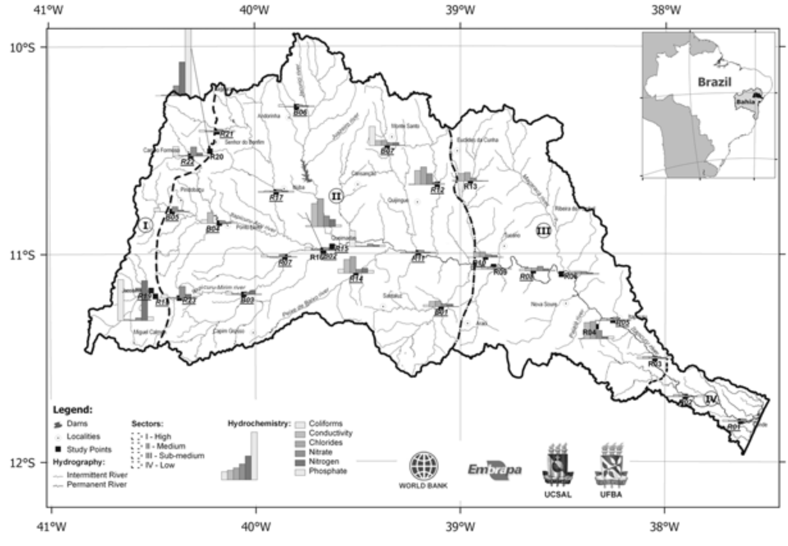

Figure 1: Maximum concentration of parameters in the sample points.

- Salinization: the points in the middle Itapicuru, where the precipitation is low, show greater values of conductivity (>600 μS/cm) and of chlorides (>300 mg/l);
- Chemical contamination: in the high Itapicuru, where there are more chrome mining activities, the concentration of metals in the sediments is higher.

3.2 Socio-environmental subsidies for the evaluation of the environmental quality

The integrated analysis of the physical environmental components of the territory (geology, terrain morphology, drainage density, soils, etc) resulted in the delimiting of homogeneous units of landscape, for which levels of physical vulnerability to the anthropogenic impacts were identified, based on the analysis of the intrinsic sensitivity and resilience of the systems.

The greater vulnerability zones (19% of the basin's area), fig. 2, are distributed preferably on the northern portion of the basin and on the areas of the coastal plain or estuary region. They are areas with an average precipitation of 1,100 mm and of high risk of erosion. In sector I are the springs of the main courses of water to the north and in sector IV the sensitive ecosystems.

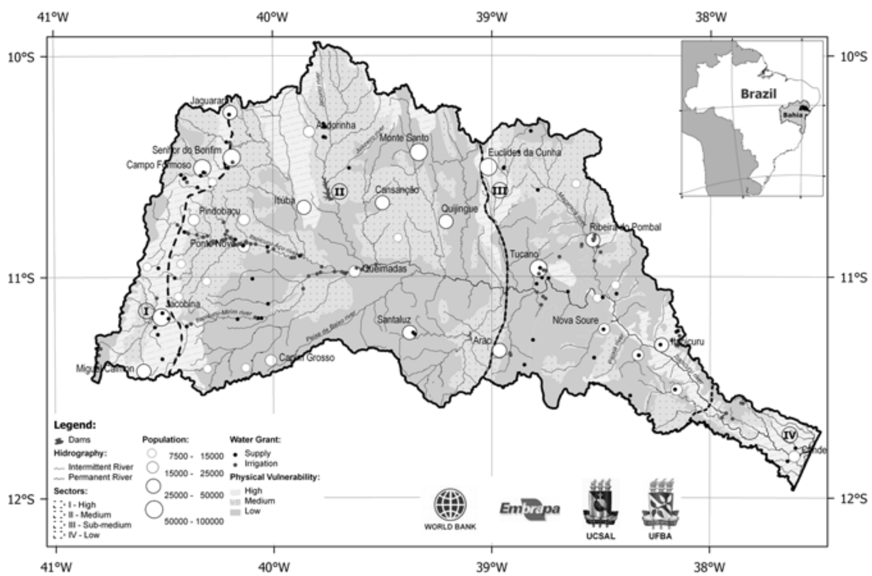

Figure 2: Map of the physical vulnerability associated to the population and water rights grant distribution in the Itapicuru basin.

The zones of medium and low vulnerability occupy, respectively, 35 and 46% of the basin's area. The areas of greater demographic density are the same as those of greater register in water rights grant or predominant use of the water resources (SRH [5]). Around 75% of the water rights grants are for irrigation and concentrated in the upper and in the middle lower sectors of the Itapicuru, respectively, sectors I and III.

Aiming at conjugating elements to mark the results of the environmental components was tried to understand the level of anthropogenic pressure associated to the economic activities of the basin. From this viewpoint, six homogeneous socio-economic units were delimited and mapped, fig. 3, identified from the analysis of socio-economic indicators, such as: urban population, GDP (Gross Domestic Product), farming activities, areas of mining and industry. The results indicate the low economic dynamism of the basin and confirm the scenery of the quality of superficial waters depicted in this study.

3.3 Environmental diagnosis and prognosis associated to the quality of the surface water

Comprehending the environmental quality of a certain territory is always a great challenge, for there are many involved variables and very often the methods used are not sufficient to produce a reading compatible to the scale and object analyzed. As a methodological procedure for the validation of the environmental quality of the Itapicuru basin, were used the indicators related to the environment, productive activities and water quality.

For water quality the indicators TSI and WQI - empirical indices aggregate values of various parameters in a sole numeric indicator. The TSI groups the results of total phosphor and chlorophyll *a*, and the WQI the following parameters: temperature, pH, dissolved oxygen, biochemical demand for oxygen – DBO, coliforms, nitrogen and total phosphor, total solids and turbidness. The metal concentration in the sediment for each point was evaluated for the three levels of standards of the National Oceanographic and Atmospheric Administration – NOAA (Buchman [6]):

TEL – Threshold Effects Levels – Concentration below which adverse effects seldom occur;

PEL – Probable Effects Levels – concentration above which is expected that the adverse effects occur; and

UET Upper Effects Threshold - concentration above which the adverse biological impacts would be expected for a certain bioindicator.

The results, fig. 3, show that in a good part of the points the TSI indicates the states of mesotrophy and eutrophy or, respectively, with intermediate or high productivity, associated to the areas agriculture and urban sewers. The hipertrophy state is identified in some points of sector I. According to WQI the water quality is good in most part of the basin, especially, in the middle and lower sectors. In sector I the values are of acceptable quality, with only point of improper quality, close to Senhor do Bonfim. The same happens for the TSI.

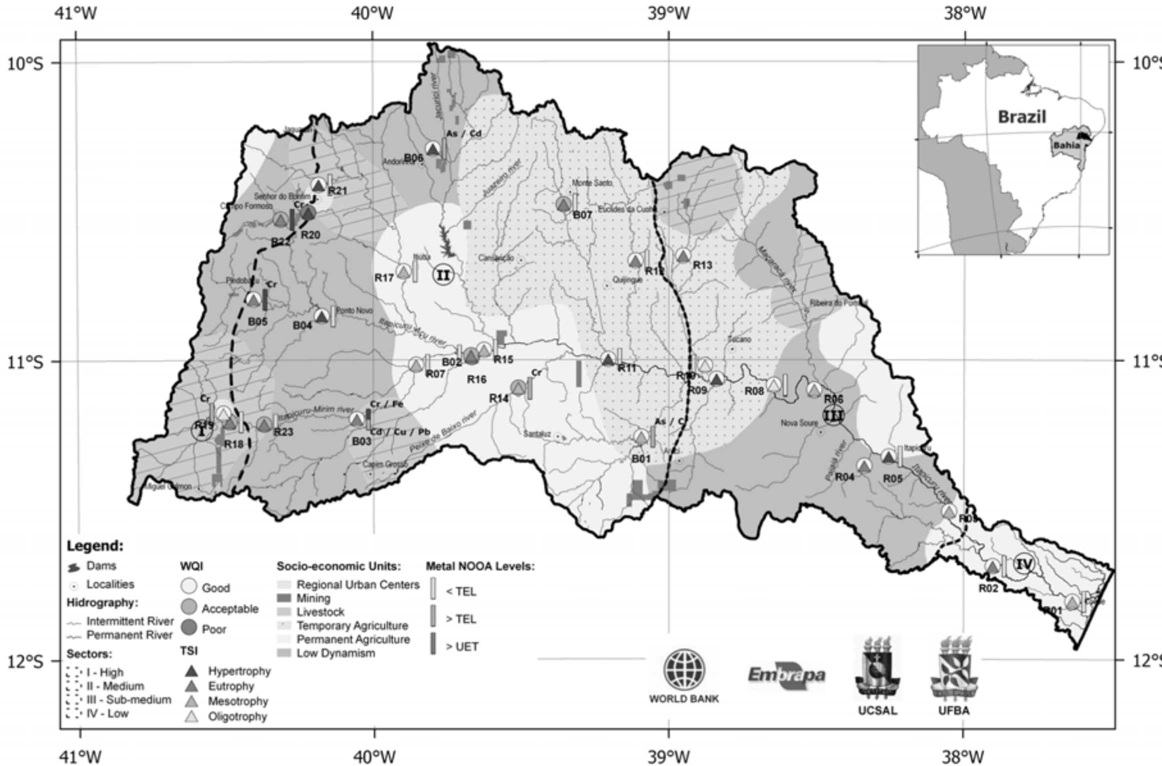

Figure 3: The zoning of the socioeconomic units on the Itapicuru basin, associated to the values of TSI and WQI in the water, and to the metals in the sediments of the sample points.

The unsuitability of the water is evident in areas of greater population, due to the sewers (residential, commercial, butchery, etc.). Mineral activity is also reflected in punctual results. In most part, the sediments bring metals with values below the TEL level, suggesting the good environmental quality of the basin. Chromium is over the UET level (95 ppm) is found in the dams B3 and B5 and on the point R22, areas with chrome mining activities.

The results lead to the diagnosis that the Itapicuru basin does have a satisfactory environmental quality, especially, related to low economic dynamism. The activities associated to agriculture and livestock have little effect on the GDP of the region and relative influence on the environmental impact on the water. The mineral activity gave dynamism to the economy and concentrated populations in the urban centers of average sizes in the past; today it doesn't promote the region's economic growth. The passive and active environmental impacts associated to the areas with mining activities are also reflected in the quality of the sediments.

4 Conclusions and recommendations

The results indicate that the quality of the basin is satisfactory. The productive activities have low polluting potential, and do not generate wealth, a factor that explains the unexpressive GDP per capita of the basin's population. The homogeneous socio-environmental units indicate the large areas where productive activities compatible with the environment's fragility can be implanted. Many areas are in zones of low economic dynamism and demographic density, indicating that there are no governmental actions to promote their sustainable development. The largest population concentrations are located in regional urban centers, given dynamism by the mineral industry that is in decadence. The economic stagnation, in most of the cities of the basin, contributes to soothe the impacts over the water resources.

The evaluation of the quality of the superficial water of the rivers, evaluated in this study shows satisfactory results, in terms of the quality indices defined by many legislation and international organisms. Then, despite the conditions imposed by the semiarid evidenced in the middle section, in general, the quality of the natural resources in the basin is good. However the economy is stagnated and the poverty rates are high. It's expected that the results and achievements of this study or the entire project will be useful to support the development of a management program on the sustainability of water resources, mainly, in order to increasing the economy in the Itapicuru basin.

Acknowledgements

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References

[1] BAHIA. Secretaria de Meio Ambiente e Recursos Hídricos (SEMARH). Superintendência de Recursos Hídricos. *Bacias Hidrográficas da Bahia*. Salvador, SRH, 2006. 2ª ed. 66 p.
[2] Projeto de Pesquisa para o Apoio ao Desenvolvimento de Tecnologia Agropecuária para o Brasil (PRODETAB). *Contribution to an integrated plan for the use and conservation of water resources in the Itapicuru river basin, State of Bahia, Brazil.* Projeto 055-01/01 EMPRAPA /FAPEX/UCSAL/UFBA. 2007.
[3] American Public Health Association (APHA-AWWA-WPCF). *Standards methods for the examination of water and wastewater.* 19th ed. Washington. DC: 1995. 1268p.
[4] Carlson, R.E., *A trophic state index for lakes*. Limnology & Oceanography, (22), pp. 361-369. 1977.
[5] BAHIA. Secretaria de Recursos Hídricos, Saneamento e Habitação. Superintendência de Recursos Hídricos. *Plano Diretor da Bacia do Itapicuru*. Salvador, SRH, 1995. 243. http://hidricos.mg.gov.br.
[6] Buchman, M.F. *NOAA Screening Quick Reference Tables.* NOAA HAZMAT Report 99-1, Seattle WA, Coastal Protection and Restoration Division, National Oceanic and Atmospheric Administration, 1999. 12p.

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An early warning monitoring system for quality control in a water distribution network

R. Guercio & E. Di Ruzza
Department of Hydraulic, Transportation and Roads,
Faculty of Engineering, University of Rome "La Sapienza", Italy

Abstract

The goal of an early warning water monitoring system (EWWMS) for quality control in a water distribution network is to identify low probability/high impact contamination events in source water or distribution systems. It should be able to detect not only intentional contamination, but also contaminants introduced accidentally or natural occurrences. Firstly, to ensure the full protection of drinking water, a technology- based EWWMS should be one component of the program. While laboratory technology exists to measure a wide range of contaminants, it's not the same for on-line and real time technology. A number of research projects are investigating rapid and on-line monitoring technologies, including biosensors and biochips, fiber optics and microelectronics. In most cases the performances of these systems has not been fully characterized. In this paper, a submergible UV/VIS spectrometer has been extensively tested in laboratory experiments, with the aim to check its limits of sensitivity, its ability to identify and quantify specific contaminants and its rate of false positive/negative.
Keywords: early warning water monitoring system; drinking water distribution system control; spectroscopy; on-line monitoring, tracer test, contamination.

1 Introduction

The goal of an EWWMS is to identify reliably low probability/high impact contamination events in source water or distribution systems. Monitoring water quality in real time allows an effective local response that minimizes the adverse impacts that may result from the event [1, 14]. Other desirable features of an EWWMS include: affordable cost, low skill and training, coverage of all

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potential threats, ability to identify source, enough sensitivity to quality changes at regulatory levels, minimal false positive or negative responses, robustness, reproducible and verifiable results, availability of remote operation. Analytical chemical methods are usually target oriented; in other words, these methods can detect only a specific compound or a range of compounds having similar properties. Furthermore, analytical-chemical identification does not give information about bioavailability and possible toxic effects, especially from mixtures of compounds. On the other hand, changes in the behaviour or properties of on-line biological early warning systems may indicate the sudden occurrence of a pollutant not detected in conventional, analytical warning systems. [4–6]

The EWWMS here described was developed and field applied to ensure water quality protection for one of the inlet points of the Acquedotto del Ruzzo water distribution system, which mainly supplies drinking water to Teramo city (about 500.000 people) in Abruzzo region, Italy. The objective of this monitoring system is to check accidental spills or contaminations coming from temporary on-working activities carried on in one of the two car tunnels placed just above the water source in the Gran Sasso Mountain, and to detect potential accidental spills coming from the National Institute of Nuclear Physics (INFN), also placed into the Gran Sasso Mountain, that is one of the largest world underground research laboratories for Nuclear Physics and cosmic research.

Owing to the relevance for supply uses of the water flowing around the highway galleries and due to recent and limited time spills of contaminant from INFN laboratory, which were not detected by the current conventional monitoring system, a new advanced real time EWWM system was designed and installed in some critical points of the water supply pipe, in order to ensure the best possible level of security and to quickly detect possible water contaminations.

2 Methods

The new EWWMS was designed and developed by Systea Srl (Italy) to be completely modular, expandable and very flexible in its use, in order to be easily adapted and installed in different monitoring sites during the experimentation phase and the operative survey activity.

The basic layout of the EWWMS is composed by:

- a 220 Vac / 12 Vcc external power supply, mounted in a IP-65 wall mounting box;
- a local control unit made by a wall mounting IP65 plastic box, which contains the industrial PC and the I/O devices, the 12 Vcc power supply with back-up batteries and any auxiliary hardware device;
- a multiparametric probe (YSI 6820, YSI Environmental Inc., USA) to measure standard physical-chemical parameters in water (temperature, pH, conductivity, turbidity) which can be placed, according to the specific site requirement, directly in the water source or working off-line ;

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- an in-situ UV spectrometer (Spectro::lyzer, s::can Messtechnik GmbH, Austria) mounted in a special horizontal flow-through cell, to detect organic compounds;
- a water flow sensor, to be chosen according to the different installation site and application specification (Argonaut YSI/Sontek, (USA), electromagnetic flow sensors, in-tube mounting type ultrasonic sensors);
- a 500 ml, 8 bottles automatic sampler, specifically designed in a IP-65 wall mounting layout, for the collection of water samples in presence of threshold alarms detected by the on-line measuring devices.

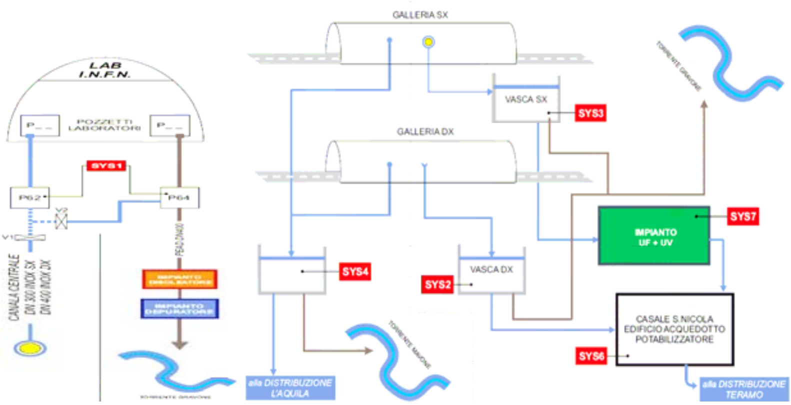

Figure 1: On- line monitoring system and sensors locations.

Depending on the specific installation site, each monitoring station is connected with an external Web server through the use of dedicated TCP-IP communication lines, automatically connected to the Internet through analogical or digital modems, private DSL line coupled with an ISDN external line, using the actual cheaper and very flexible routing technologies.

Each monitoring station is also connected to an industrial GSM device to send to operator's and technician's mobile phones Short Messages, informing them of any alarms and parameter's thresholds overlaps, which will cause a reaction on the system with automatic closure of hydraulic valves mounted on the tubing, to quickly redirect the water to be discharged in spite of being introduced in the potable water distribution system.

The tested submersible UV/Vis spectrometer measures absorbance of ultraviolet light from 200 to 400 nm. The instrument is built as a compact submersible sensor enabling measurements of UV/Vis spectra; The results of the measurements are recorded and displayed in real-time, with a single measurement typically taking 45 seconds.

The instrument is a 2-beam, 256 pixel, UV/Vis spectrophotometer, with a Xenon lamp as a light source. By recording the light absorption in a sample between 200–400nm, the probe is capable of determining more than simply a fingerprint spectrum; besides this fingerprint, the values for a number of specific parameters are calculated from the data contained in the fingerprint, using algorithms provided by S::can [8, 9].

Figure 2: Installation of EWWS monitoring station.

3 Results

3.1 Laboratory tests

The parameter deducted from the fingerprint, using the software provided by Scan, are: Turbidity, TOC, DOC, Nitrate, SAK254.

We could besides monitor specific compounds, once their fingerprint is known and recorded on the probe; for this purpose we decided to use it to monitor also Benzene and Tri-methyl- benzene.

The first one because it could show pollution contamination from the high way road inside the tunnel. The second one because of possible accidental spills coming from the underground laboratory.

The calibration curves were determined using local calibration provided by the software. A global calibration is also available but for particular water composition, like drinking water, the local one is more accurate. The measurements were performed with both drinking and distilled water to appreciate the effects of the water matrix on the measurements. We performed cross sensitivity test between nitrate and benzene and between nitrates and Tri-methyl-benzene to study how they interact with each other. For calibration, samples with known concentration were measured and we determined the accuracy of the instruments, the higher and lower detection limits. Figures 3, 4, 5, show the local calibration curves determined for the all the parameters of the probe.

The nitrate could be detected in drinking water down to 0,1 ppm NO3 –Neq. Below this concentration, the deviation from background spectra became too small for detection. The upper level of detection was 11 ppm, even if the calibration curve results linear until 6 ppm; over that limit the correlation became polynomial.

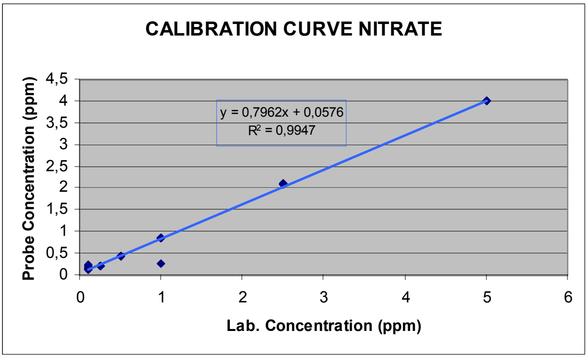

Figure 3: Nitrates calibration curve.

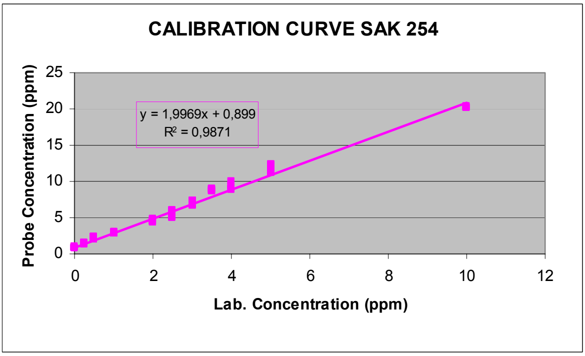

Figure 4: SAK254 calibration curve.

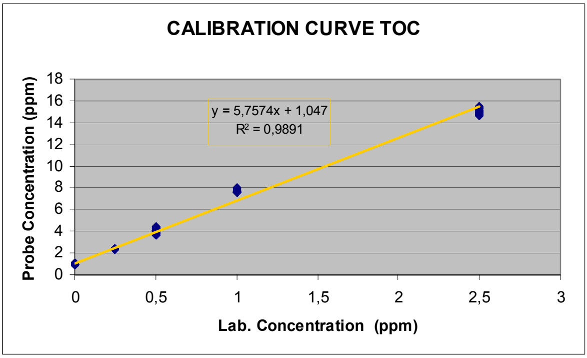

Figure 5: TOC calibration curve.

The TOC could be detected down to 0,1 ppm and the higher level of detection was 3 ppm. This shows that the probe results in high sensitivity on organic compound.

In particular, for the benzene, the lower limit of detection is 1 ppm and the higher limit is 500 ppm; the water matrix doesn't affect the measures.

Regarding the Tri-methyl-benzene, it is extremely volatility in water, doesn't allow to correlate the absorption vs. concentration with high precision, but the probe is able to detect the presence of the substance also for low concentration.

3.2 Application

3.2.1 Laboratory tests

The UV/VIS spectrolyser was also tested in-situ, simulating a pollution release.

Since we are talking about drinking water, we decided to use as tracer an inert substance, like ascorbic acid (Vitamin C).

Moreover Vitamin C was identified as reliable absorbance indicator for the UV detection probe used in the EWWMS because it can simulate the presence of organic contaminant in the water.

As Figure 7 shows, ascorbic acid absorbs light in the range between 200 and 250 nm; that is in a typical absorbance range of many organic compounds.

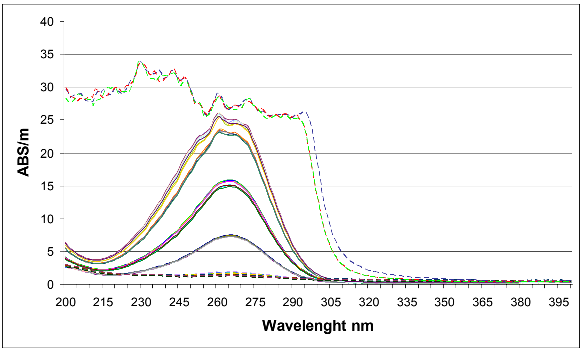

Figure 6: Typical UV spectra for ascorbic acid.

We did a proper calibration of each probes at different acid concentrations, in order to check the sensitivity of the system for different level of detection, and to compare the concentration of Vitamin C to the values given by the probes, and to establish the reproducibility of the measures. Concentrations of ascorbic acid in the range 0,1-10 ppm were tested.

The fingerprint of Vitamin C showed a clearly visible peak also for high concentrations, but in this case the value of the TOC parameter reached the saturation.

The value of SAK 254 and TOC increased linearly with the increasing of Vitamin C concentration.

Also the nitrate concentration recorded by the probe was influenced; but the relation between the two values was not linear .

Figures 8 and 9 show the correlation between Vitamin C, TOC and SAK 254.

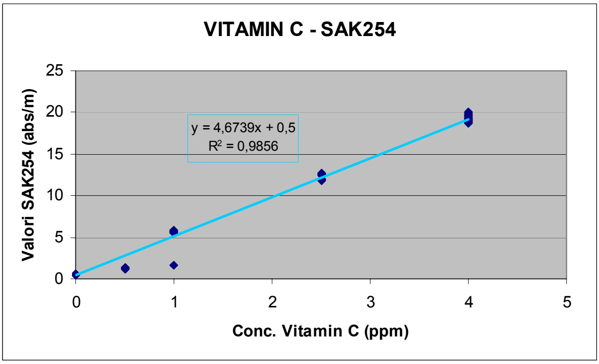

Figure 7: Correlation between Vitamin C and SAK254.

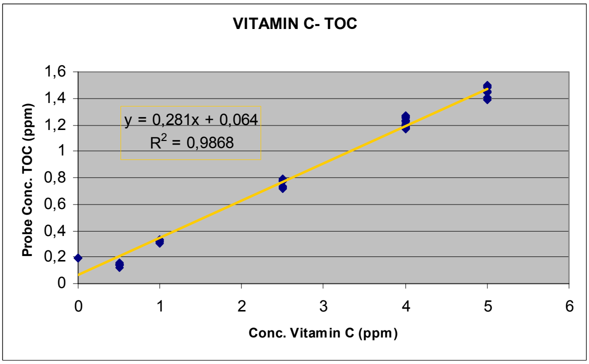

Figure 8: Correlation between Vitamin C and TOC.

After a long period of time to establish that the signal was stable and that the system could record clearly the difference in the water matrix, we spiked Vitamin C with a pre-set concentration.

Table 1 shows all the data necessary for the test.

During the test, the spiked concentration of ascorbic acid was kept at 0,5 mg/l.

Figures 10 and 11 show the data recorded by the system during the test on the SYS1 station.

Data were registered every 30 seconds for SYS 1 and every 15 minutes for SYS 2. As Figures 12 and 13 show, data recorded in SYS 2 made visible a lower concentration than those in SYS1 because a dilution occurred in the thank before the SYS2 station. A shift in time of about 15 minute was also registered, due to the different retention time of the two systems.

Table 1: Tracer test data.

Spiked time	1 h
Vitamin C concentration	0,5 mg/l
Network Flow	130 l/s
Volume of solution	300 l
Concentration in the tank	780 mg/l
Vitamin C flow	5 l/min

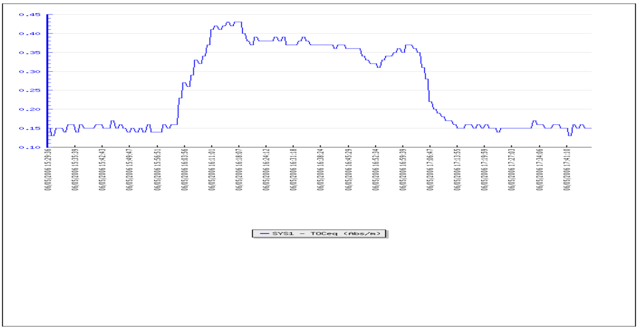

Figure 9: SAK254 data recorded by the system during the tracer test on SYS1.

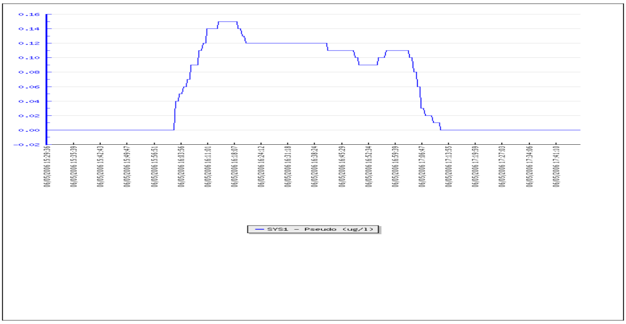

Figure 10: TOC data recorded by the system during the tracer test on SYS1.

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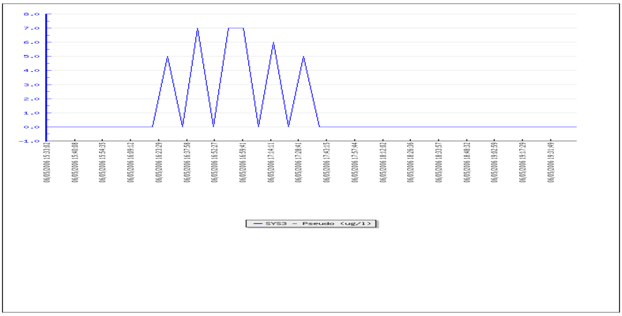

Figure 11: SAK254 data recorded by the system during tracer test on SYS2.

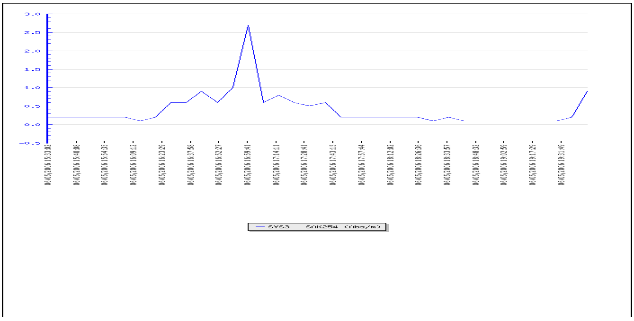

Figure 12: TOC data recorded by the system during tracer test on SYS2.

4 Conclusion

In order to develop a new early Warning System for contamination detection, a submergible UV/VIS spectrometer has been extensively tested in laboratory experiments, with the aim to check its sensitivity, its ability to identify and quantify specific contaminants and its rate of false positive/negative.

Laboratory and in situ tests have shown that the spectrolyser is easy to use and it provides robust results. It has high sensitivity and reproducibility especially for drinking water; it can also detect and monitoring the natural fluctuations in water composition. Moreover the probe is able to detect specific compounds and to quantify their concentration.

Even if the compound is unknown, the probe is able to detect him in drinking water, as not belonging to the natural fluctuation of the matrix. All the tests showed that the detection limits for all the parameters were very low. The spectral information for most compounds will not be enough for quantity identification, but parameters like TOC, DOC, SAK254 and NITRATE can be used to detect water contamination with unknown compounds.

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The entire system is able to detect an event of contamination, quickly and with high precision, and it has almost all the characteristics that an EWWS should have.

References

[1] Brosnan T.M. (1999). *Early warning monitoring to detect hazardous events in water supplies*. An ILSI Risk Science Institute Workshop Report.

[2] Gunatilaka A. and Dreher J (1996). *Use of early warning systems as a tool for surface and ground water quality monitoring*. Proc. IAWQ and IWSA Symp. on Metropolitan areas and Rivers, Rome. TS1 – River quality surveying and monitoring methods **2**, 200-211.

[3] Ulitzur S. Lahav T. and Ulitzur N. (2002). *A Novel and Sensitive Test for Rapid Determination of Water Toxicity*. Environmental Toxicology Journal **17**, 291-296.

[4] Clark, R. M., N. Adam, V. Atluri, M. Halem, E. Vowinkel, P. C. Tao, L. Cummings, and E. Ibrahim (2004). *Developing an early warning system for drinking water security and safety*. Pages 8.01-8.19 in L.W. Mays (ed.) Water supply systems security. New York, NY: McGraw-Hill Companies.

[5] Grayman, W. M., R. A. Deninger, and R. M. Males (2001). *Design of early warning and predictive source water monitoring systems.* Denver, CO: AWWA Research Foundation.

[6] Grayman, W. M., R. A. Deninger, and R. M. Clark (2002). *Vulnerability of water supply to terrorist activities*. CE News 14:34-38.

[7] Grayman, W. M., R. A. Deninger, R. M. Males, and R. W. Gullick (2004). Source water early warning systems. *Pages 11.01-11.33 in L.W. Mays (ed.) Water supply systems security*. New York, NY: McGraw-Hill and Companies.

[8] Langergraber, G., Weingartner, A., Fleischmann, N. (2004): *Time-resolved delta spectrometry: A method to define alarm parameters from spectral data*. Water Science & Technology 50(11), 13-20.

[9] Langergraber, G., Fleischmann, N. and Hofstaedter, F. (2003). *A multivariate calibration procedure for UV/VIS spectrometric quantification of organic matter and nitrate in wastewater*. Wat. Sci. Tech., 47(2).

[10] Lee, J. Y., and R. A. Deininger (1992). *Optimal locations of monitoring stations in water distribution systems*. Journal of Environmental Engineers. 118(1): 4-16.

[11] Ostfeld, A, Kessler, A, Goldberg, I, *A contaminant detection system for early warning in water distribution networks*, (2004). Engineering Optimization, Vol. 36, No. 5, 525–538

[12] Jafrul Hasan, Stanley States, and Rolf Deininger, *Safeguarding The Security Of Public Water Supplies Using Early Warning Systems: A Brief Review,* (2004) Journal of Contemporary Water Research and Education, Issue 129, Pages 27-33.

Three applications of dams in Nepal, Malaysia, and Turkey

R. Morimoto
Toulouse Business School, CERMAS and Department of Strategy, Toulouse, France

Abstract

This paper empirically applies cost benefit analysis to much debated hydropower projects in Malaysia, Nepal and Turkey. The study selects an interesting mixture of cases, as main characteristics of each dam, geographical locations of each dam, and the development stage of each country differ. The study brings together all the major issues attached to each hydropower project and estimates the quantitative impacts of these controversial dams. The cost benefit analysis model in this study takes into account of premature decommissioning of dams and the correlation between the parameters of generation capacity, total construction cost and construction period. The mean cumulative net present value at the 100th year of the analysis with the 5% discount rate for Sharada-Babai dam in Nepal shows a positive figure, whereas the mean cumulative net present value after 100 years for both Bakun dam in Malaysia and Ilisu dam in Turkey are negative. The mean value of the cumulative net present value for Sharada-Babai becomes negative when the pure rate of time preference is larger than 6%; for Bakun and Ilisu, converge to zero as pure rate of time preference becomes larger. The sensitivity analysis shows the dominant positive impact of the generation capacity parameter on net present value for Bakun; and the parameter expressing initial expected increase in economic output for Sharada-Babai and Ilisu.
Keywords: Bakun, cost benefit analysis, hydro power development, Ilisu, net present value, sensitivity analysis, Sharada-Babai.

1 Introduction

The serious negative effects of large dams have been widely discussed, and the roles of large dams have been questioned, over the past two decades (WCD [1]). However, there are still numerous plans to build massive dams worldwide,

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especially in developing countries. Are these dams really worthwhile to build? Will they contribute to the sustainable development process? This paper tries to answer the question based on an empirical application of probabilistic CBA (cost benefit analysis) to much debated hydroelectric projects in Nepal, Malaysia and Turkey. The study selects an interesting mixture of cases, as main characteristics of each dam, geographical locations of each dam, and the development stage of each country differ. The study brings together all the major issues attached to each hydropower project and estimates the quantitative impacts of these controversial dams. The CBA model in this study is further developed from the previous version in Morimoto and Hope [2, 3]), in order to take into account the correlation between the generation capacity, the total construction cost and the construction period.

This paper consists of the following four sections. The next section describes the characteristics of three hydropower projects. The third section explains the methodology and lists the variables used in the model. The fourth section presents the findings from the study including sensitivity analysis and the variation of cumulative Net Present Value (NPV) against the pure rate of time preference. The last section summarizes and concludes the study.

2 Malaysia: Bakun Hydro Project

Together with rapid economic growth, electricity consumption in Malaysia grew at 5.6% between 2000 and 2005, according to APEC Energy demand and Supply Outlook 2006. The electricity demand in Malaysia is mainly met by three major sources; gas (47%), hydro (10%), and coal (43%) in 2005, according to Tenaga National Berhad (TNB). The government intends to balance this ratio and tries to increase hydropower generation, which has led to the controversial Bakun Hydroelectric Project construction plan. The Bakun dam on Balui River in Sarawak state on Borneo Island will be South East Asia's largest dam with a generation capacity of 2.4GW, and is supposed to cater for the rising industrial demand for electricity in Malaysia. Although this development of a remote part of Malaysia might improve the regional economy, through increased employment and infrastructure development, such as access roads and airports, there are also many costs associated with it. It will flood 70,000 hectares of rainforests and abundant timber sources, equivalent to the size of Singapore, and fertile agricultural lands. The construction costs for this gigantic project will be huge, despite the fact that there would be a possibility of insufficient demand to absorb its enormous generation of electricity. Another great concern of the project is the displacement of 10,000 indigenous people. The entire ecosystem of the area also might change; especially large negative effects on downstream fishery are anticipated. The area is already suffering from siltation due to timber extraction, which is highly likely to restrict power generation capacity in the near future. Poor working conditions at the dam site are also reported. The project has been delayed due to serious environmental, social and financial problems. The project was originally scheduled to complete in 2003, however was revised to 2007.

3 Nepal: Sharada-Babai Hydro Project

The electricity demand in Nepal is estimated to grow at 7.5% annually until 2020 (Asian Development Bank (ADB) [4]). Lack of electricity may restrict economic growth, especially in the industrial and service sectors – the fastest growing economic sectors in Nepal (Pokharel [5]). The electrification rate in Nepal is still very low – only about one third of the population has access to electricity, and the rate is much lower for rural population (ADB [4]). The country's major sources of energy for rural population are fuelwood and kerosene (Mahapatra [6]). The government in Nepal plans to provide electricity to 55% of the population by 2007, which requires rapid expansion of electrification in the country (ADB [4]). Nepal has the great potential for hydropower and its theoretical generation capacity is 83 GW according to the Nepal Electricity Authority (NEA). Currently, there are many planned hydropower projects in Nepal in order to utilize the resources. The proposed Sharada-Babai hydropower project in the Mid Western Region of Nepal has an installed capacity of 93MW, and requires a 5-year construction period. The project is located between the tropical and sub-tropical climatic regions of the country. The land of the reservoir area is very fertile, whose main crops are paddy and wheat. More than 300 families will be relocated from the core project area. The major impact of the project will be the submergence of rich forest and agricultural land. The impoundment and reduced flow downstream will change the hydrology of the basin, which is likely to affect the aquatic environment of the river. The holy pond and the Kali Vhagwati (a temple of religious and cultural importance) will be inundated due to the reservoir formation. Infrastructure relocation such as a school, a post office, and suspension bridges are also required.

4 Turkey: Ilisu Hydro Project

The electricity consumption in Turkey has been growing by the rate of 6% per year as a result of rapid urbanization and industrialization (Turkey's Energy Strategy by Ministry of Foreign Affairs [7]). The electricity demand in Turkey is met by oil (38%), coal (27%), gas (23%), and hydro and renewable (12%), according to Turkish Ministry of Foreign Affairs [7]. The Ilisu dam is presently the largest hydro project in Turkey with a generation capacity of 1,200 MW. The dam will be located on the Tigris River in South-East Anatolia, 65 km upstream of the Syrian and Iraqi border. The project plan was stopped in 2002 as it is extremely controversial for a variety of political, social, environmental, economic, and archeological reasons. However, the project resurfaced in 2005. The project will submerge approximately 52 villages and 15 small towns and affect between 25,000 and 78,000 people mainly ethnic Kurds. (http://www.rivernet.org/turquie/ilisu.htm) The people living in the project area are mainly farmers who grow cereals, vegetables, fruits, cotton, tobacco as well as herbs and a small number of shepherds. Although there is no real commercial fishing in the River Tigris, farmers fish for local consumption. People moving to cities will face sharp changes in their daily activities and traditional ways of life

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which often cause significant problems of adaptation to the urban environment, family tensions, psychological stress and social disruption. Hasankeyf, the main town to be inundated is the only town in Anatolia which has survived since the middle ages without destruction, and is also known as an important pilgrimage center. The town's history dates back at least 2,700 years. Being a rich treasure of Assyrian, Christian, Abassidian-Islamic and Osmanian history in Turkey, Hasankeyf was awarded complete archeological protection by the Turkish department of culture in 1978, and 22 monuments have been entered on the Turkish Cultural Inventory List in 1981. The lack of funds and time means that it will be difficult to relocate the treasures that will be inundated. Moreover, even after successful relocation, they may not create the same aesthetic impact in a new site. Hasankeyf is also a tourist destination and a holiday resort, so inundation of this town is likely to affect tourism in the area. The area to be flooded by the reservoir includes 7,353 hectares of good agricultural land, 4,820 hectares of medium-low quality land, and 15,675 hectares of land not suitable for growing crops. (http://www.rivernet.org/turquie/ilisu.htm) Since, the amount of trees and shrubs in the area to be flooded is relatively small, the carbon dioxide release from the reservoir is assumed to be small compared to the other reservoirs of the same size in dense forest (IEG [8]). Solid waste and wastewater of major cities are being dumped into the Tigris River without any treatment. The Ilisu reservoir will vastly reduce the auto-purification capacity of the Tigris. This is then highly likely to cause water quality degradation and possibly to affect the downstream fishery negatively.

5 Methodology

The CBA model used in this study is further developed from the previous version in Morimoto and Hope ([2] and [3]), in order to take into account the correlation between the generation capacity, the total construction cost and the construction period. Table 1 lists the variables used in the CBA model: the core variables included in all the case studies are PG (power generation), CP (clean power), EG (economic growth), CC (construction cost), OM (operation and maintenance cost), RE (resettlement cost), IN (losses due to inundation of land), AC (accident cost). The project specific variables are FI (impacts on downstream fishery) for Bakun; IF (infrastructure cost) for Sharada-Babai; and LT (tourism loss) and AS (archaeological loss) for Ilisu. A brief summary of the equations used in the model, and all the parameters in the model and their values are listed in Morimoto and Hope [9]. Dixon *et al.* [10] argue that the appropriate time horizon should be long enough to encompass the useful life of the proposed project, therefore the project life of 100 years is selected in this study.

Some inputs in the CBA model seem to be clearly correlated to each other. If the generation capacity were to be reduced, the total construction cost and the construction period would also be reduced accordingly. Hence, we assume that a minimum value for one of these variables implies a minimum value for the others. The same assumption is made for most likely and maximum values. The correlation coefficients that this implies for Bakun, Sharada-Babai and Ilisu project are all above 0.7. There is a wide range in the data for GC (generation capacity) in the Bakun case, as the project size might be drastically reduced due to the protests. The minimum, most likely and max values are 0.5, 0.7 and 2.4 GW respectively. Therefore the consideration of this input dependency would be particularly important for Bakun.

Table 1: Variables used in the CBA model.

Variable (core variables in **bold**)	**Description**
EG (economic growth)	The forgone economic costs for electricity not served, which will occur during the time when an alternative power generation technology is not available.
PG (power generation)	Calculated by multiplying the quantity of electricity generated by the price of electricity. A possible reduction in the quantity of electricity generated due to sedimentation, and changes in electricity prices, are also included in the equation.
CP (clean power)	The environmental benefit of avoiding damage from air pollution when generating an equivalent amount of power by thermal power. This benefit will occur during the time when an alternative power generation technology is available.
IN (losses by land inundation)	Financial values of forests and agricultural lands to be inundated
AS (archaeological loss)*	Approximated by the values of cultural antiquities being sold on the market.
CC (construction cost)	Construction costs of the power station and transmission facilities.
FI (impacts on fishery)	Decline in revenues from fishery due to dam construction.
LT (tourism loss)	Losses in tourism revenue as a result of the dam construction
OM (operation & maintenance)	O&M costs for running the hydropower station.
RE (resettlement cost)	Compensation to individuals and development costs for new houses and infrastructure. Theoretically, estimations of what resettled people are willing to accept are preferred as better estimates. However, those data are difficult to obtain and their results are not very reliable, especially in developing countries.
IF (infrastructure relocation cost)	Costs required to relocate infrastructures to be inundated
AC (accident cost)	During construction, O&M & special circumstances (technical failure, terrorism attack or earthquake). Calculated by multiplying the estimated number of deaths and injuries due to the project by the economic value of deaths and injuries. Assumes that accident risks are not internalized in wages.

*How archaeology is perceived is different from place to place and generation to generation (Carver [11]). Although there are many studies on how to place values on archaeological sites, there is no simple conclusion for this argument (Carman *et al.* [12]; Carver [11]; Darvil *et al.* [13]; Lipe [14]; Schaafsma [15]).

The data are given as ranges, collected mainly from the existing project reports including Environmental Impact Assessment (EIA) reports and energy reports. Supplementary data are based on an extensive collation of information from past studies on similar topics, newspapers, Internet and other relevant sources. The most appropriate data for each parameter are selected using my best knowledge on the energy policy of the country, their current situation and future direction of power generation. Best efforts are made to find as accurate and representative data as possible. Visiting the project site and interviewing government officials, energy experts, policymakers, researchers, academics, environmentalists and locals during our fieldtrip in Malaysia, Nepal and Turkey have greatly helped such decision making process. Some of these data may not be very accurate or precise. Many of the values are tentative and open to criticism. However, this is inevitable, as many variables are not readily quantifiable and some data have a limited availability because of the project complexity or simply they do not exist. Repeated runs of the model obtain a probability distribution of possible outcomes, which is a more defensible procedure than just using single values for inputs that are in reality not well known.

6 Results

Table 2 lists the impact of each variable on the NPV for each project; EG (economic growth), PG (power generation), and CC (construction cost) are significant in all the projects; IN (inundation loss) for Bakun and Sharada-Babai; and AS (archaeological loss) for Ilisu. The impact of CC (construction cost) is huge as much as EG (economic growth), but not as prolonged. The sharp decay of the positive impacts of EG (economic growth) and PG (power generation) can be explained as a result of serious sedimentation problems. In comparison, AS (archaeological loss) and LT (tourism loss) are long lasting negative impacts, and are also not recoverable once the dam is constructed. The sensitivity analysis shows the dominant positive impact of GC (generation capacity) on NPV for Bakun, and EO (initial expected increase in economic output) for both Sharada-Babai and Ilisu.

For Bakun, the 5th percentile, mean and the 95th percentile of cumulative NPV with a premature decommissioning option at a 5% discount rate at t = 100 years are \$ -9.6, -2.8, and 7.0 billion respectively (without decommissioning they are \$ –9.9, -2.9, and 7.0 billion respectively; showing that the option of premature decommissioning gives a slight improvement). The mean, the 5th and the 95th percentiles of the cumulative NPV are all initially strongly negative due to the large construction cost. The 95th percentile recovers quickly after the construction is complete since the benefits of increased economic growth soon outweigh all the costs. Its peak is at t = 40 years, as after this date the revenue is insufficient to cover the total annual costs. The 5th percentile and the mean values both remain negative throughout the life of the project. The mean shows a slight recovery from year 10 until year 30 followed by a permanent downward movement. The 5th percentile shows a similar pattern to the mean, though it already starts declining at t = 20 years due to the large anticipated costs such as inundation and fishery losses.

For Sharada-Babai, the 5th percentile, mean and the 95th percentile of the NPV at the 5% discount rate at t = 100 years with premature decommissioning option are \$ -0.009, 0.14, and 0.34 billion respectively. The results without the premature decommissioning option are exactly the same since the recoverable costs such as AC (accident costs during operation and maintenance period) and OM (operation & maintenance cost) are small for this specific project. The range of cumulative NPV over time shows that the huge initial capital costs are gradually outweighed by the anticipated large benefits from power generation and increased economic growth.

Table 2: Mean cumulative present values for the variables at t = 100 years in US$ billion.

		Bakun	Sharada-Babai	Ilisu
PVEG	**Economic growth**	**+13**	**+0.18**	**+13.8**
PVPG	**Power generation**	**+3.6**	**+0.21**	**+3.3**
PVCP	Clean power	+0.9	+0.03	+0.5
PVIN	**Inundation loss**	**-10.5**	**-0.09**	-0.2
PVAS	**Archaeological loss**	-	-	**-9.5**
PVCC	**Construction cost**	**-4.5**	**-0.14**	**-4.9**
PVFI	Fishery loss	-4.4	-	-0.9
PVLT	Tourism loss	-	-	-3.5
PVOM	O & M cost	-0.4	-0.03	-0.5
PVRE	Resettlement	-0.1	-0.003	-1.0
PVIF	Infrastructure relocation	-	-0.02	-
PVAC	Accident cost	-0.03	-0.0004	-0.04

Note: Significant variables are expressed in bold letters.

For Ilisu, The 95th percentile is positive whereas the mean and the 5th percentile are negative. The mean, the 5th percentile and the 95th percentile of the cumulative NPV at the 5% discount rate with the premature decommissioning option at t = 100 years are \$ –9.9, -3.5, and 4 billion respectively. The benefits never outweigh the costs throughout the period for the 5th percentile and the mean, while the 95th percentile turns to be positive soon after the completion of construction. The 95th percentile starts declining gradually after its peak at t = 40 years. The NPV are all negative at the beginning because of large construction and resettlement costs. The NPV increases rapidly once electricity starts to be generated, however the mean and the 5th percentile remain negative as a result of huge archaeological losses from the destruction of this important area in Turkey which has numerous unique archaeological sites as well as tourist and pilgrim attractions. The result also shows that the consideration of premature decommissioning would not improve the result, since those recoverable costs after premature decommissioning such as operation and maintenance cost, accidents costs during operation, and land inundation loss for this project are relatively small.

7 Conclusion

This study has examined the possible outcome of three highly controversial dam projects in Malaysia, Nepal and Turkey. Characteristics of each dam differ in terms of size, geographical locations, and the level of economics development. The result for the Bakun dam in Malaysia shows expected huge losses due to the inundation of the dense rainforest and the fertile agricultural lands, and the significant negative impacts on downstream fishery. The benefit of 2.4GW electricity supply and facilitated economic growth might outweigh these massive losses. However, this is unlikely to occur during any feasible planning horizon, as can be seen from the 5th percentile and the mean of the cumulative NPV being negative even after 100 years. The result obtained for the Sharada-Babai dam in Nepal presents more positive outcome. There are large benefits from the increased economic growth due to an increased power supply as the country is presently seriously short of electricity, large revenues from electricity sales, and the benefits of clean hydropower, in exchange for a large construction cost and the unrecoverable loss of submerging fertile agricultural land as well as forest. This study confirms that the improvement of hydropower development in Nepal seems to be crucial given the careful consideration of the consequences to the community and the environment. The main finding for the Turkish Ilisu dam is the enormous impact of the loss of unique archaeological sites, some of which date back at least 2,700 years. The losses are not only the value of these cultural heritages and artefacts to be submerged, but also the significant number of tourists and pilgrims visiting the sites. Both the 5^{th} percentile and the mean of the cumulative NPV depict that the proposed project benefits will not outweigh the large costs of the project. This result coincides with the even that the project was postponed in 2002, due to significant social and environmental concerns.

The overall results show that large scale dams generally tend to create large social and environmental impacts. The Nepalese dam case illustrates that smaller scale dams seem to be more sustainable than gigantic scale dams. Furthermore, huge costs are involved in each project discussed above, therefore an increase in electricity prices might not be avoidable. However, this may not be the case for the Sharada-Babai Project, as the benefit is likely to outweigh the cost soon after the construction. The beauty of the model developed here is its flexibility: developers could choose from an array of variables that most closely fit the constraints and conditions of their individual case. Some of the available variables are either included or excluded in order to serve the differences in each project. These case studies with diverse individual concerns in this study has demonstrated that the generalized model can be a highly practical tool, with the great advantage of its simplicity and a capability to cater for almost any kind of hydropower project assessment under uncertainty. Moreover, the application of this type of probabilistic CBA model is not limited to assess only hydropower projects, but it is also possible to extend its ability to assess other engineering or infrastructure projects, such as new aircraft, water pipes or tunnels (for example Morimoto and Hope [16]).

The limitations of the model are such that some impacts could be much larger than the impacts included in the above model, though may not be quantifiable. For example, the scale of the resettlement for the Bakun dam is enormous and the resettlement impact is therefore expected to be huge, though this impact seems to be not fully reflected in our results. This suggests that current estimates of the resettlement cost might be too low, and that alternative approaches such as 'willingness to pay' or 'willingness to accept' might better estimate the true resettlement cost. Furthermore, some evaluations in the model might be over-simplified. There would be other possible impacts for each project not considered; some of these could be major. For example, increased malaria infection, health problem caused by reduced water quality, negative impacts on biodiversity in the area. Displaced people are further impoverished economically, and may suffer cultural decline, high rates of sickness, malnutrition, deaths, and great psychological stress. Some of the dam sites, such as Bakun, are a home of many unique species of fauna and flora, which may disappear due to the project's impact. The Bakun dam site is located in dense forest; therefore the CO_2 release from the reservoir might not be negligible. These impacts are omitted from the current analysis due to lack of data and the difficulty of quantifications. Including every single impact is not possible, therefore the variables entered into the model are prioritized in each project assessment in this research. They could, however, be included in the future analysis, if relevant, with sufficient background information.

References

[1] WCD, Dams and Development: A new framework for decision-making, Earthscan, London and Sterling, 2000

[2] Morimoto R and Hope C, 'The CBA model for the Three Gorges Project in China', Impact Assessment and Project Appraisal Journal, **22**(3), 205-220, 2004

[3] Morimoto R and Hope C 'An extended CBA model of hydro projects in Sri Lanka', International journal of global energy issues: special issue on energy and renewable energy with economic development in developing countries, **21** (1/2), 47-64, 2004

[4] ADB, Technical Assistant Report for Nepal, Manila, 2004

[5] Pokharel S 'Hydropower for Energy in Nepal', Mountain research and Development, 21 (1), 2001

[6] Mahapatra R, 'Power the people', Appropriate Technology, Jan-Mar, 2001

[7] Ministry of Foreign Affairs, Energy Strategy, Ministry of Foreign Affairs Turkey, 2006

[8] Ilisu Engineering Group (IEG) Ilisu Dam and HEPP Environmental Impact Assessment Report April, 2001

www.witpress.com, ISSN 1743-3541 (on-line)

[9] Morimoto R and Hope C, An empirical application of probabilistic CBA: three case studies on dams in Malaysia, Nepal and Turkey, JBS WP 19, http://www.jbs.cam.ac.uk/research/working_papers/2002/wp0219.pdf 2002

[10] Dixon, J.A. Scura, LF. Carpenter, RA. and Sherman, PB, 2nd ed, Economic Analysis of Environmental Impacts, The Asian Development Bank and the World Bank, London, 1994

[11] Carver. M, 'On archaeological value' Antiquity **70** 45-56CTGPC (1995) Environmental Impact Statement for the Yangtze Three Gorges Project (A Brief Edition) Science Press, 1996

[12] Carman. J, Garnegie, GD, and Wolnizer. PW, 'Is archeological valuation an accounting matter?' Antiquity **73** 143-8, 1999

[13] Dervil, T., Saunders, A., and Startin, B. 'A question of national importance: approaches to the evaluation of ancient monuments for the monuments protection programme in England' Antiquity **61** 393-408, 1987

[14] Lipe, WD, 'Value and meaning in cultural resources' in Cleere. HF (1984) (ed.) Approaches to the Archaeological Heritages, Cambridge University Press 1-11, 1984

[15] Schaafsma. CF 'Significant unit proven otherwise: problems versus representative samples' in Cleere (1984), 1989

[16] Morimoto R and Hope C 'Making the case for developing a silent aircraft' Transport Policy 12(2) 165-174, 2005

www.witpress.com, ISSN 1743-3541 (on-line)

Synthetic hydrology and climate change scenarios to improve multi-purpose complex water resource systems management. The Lake Ontario – St Lawrence River Study of the International Canada and US Joint Commission

L. Fagherazzi[1], D. Fay[2] & J. Salas[3]
[1]*Plan pluriannuel et systèmes hydriques, Hydro-Québec, Canada*
[2]*Great Lakes – St. Lawrence Regulation Office, Cornwall, Canada*
[3]*Department of Civil Engineering, Colorado State University, USA*

Abstract

The hydrological data used in the design, planning and operational studies of water resource schemes are often limited to historical records, which are usually short, incomplete, sparsely distributed in space, and poorly synchronized. Moreover, a particular sequence of flow observations rarely reoccurs in identical form in a future period. If important characteristics of the historical series such as annual sequences of low and high flows are poorly modeled, design and/or planning shortcomings may then result, because their effects were underestimated. To better grasp drought and flood variability, the Lake Ontario – St. Lawrence River Study used both historical and synthetic hydrological scenarios. A set of stochastic hydrological Net Basin Supplies series for the Great Lakes, and local inflows of the St Lawrence River's major tributaries upstream from Trois-Rivieres sequences was developed, based on the statistical properties of the historical series. The set constitutes a large number of potential hydrological scenarios that could occur due to the observed natural variation in climate. The simulated series were then used, along with the observed 1900-2000 data, to design and evaluate the adequacy of newly proposed Lake Ontario multiobjective management strategies. In addition, four hydrologic scenarios incorporating climate change fields on seven climate variables were developed for the Great Lakes and Ottawa River. Scenarios from General Circulation Model (GCM) output changes, covering the range of climate change variability for the future 30-year period 2040-2069, were used to test the robustness of the management strategies under possible future climates. This paper summarizes the special hydrological characteristics of the Great Lakes – St. Lawrence River System, the major modeling hypothesis and retained strategies used to prepare a stochastic generation model, and the preparation of climate change scenarios for a major tributary, the Ottawa River basin. It also illustrates their use in evaluating Lake Ontario multiobjective management strategies.

Keywords: stochastic generation model, shifting mean and ARMA process, climate change scenarios, Lake Ontario regulation plans.

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1 Introduction

The Great Lakes – St Lawrence River system is a complex lake-river system, having large amounts of over-year storage, characterized by particular spatial and temporal properties of inflows series. Figure 1 presents the basins and sub-basins of the system, covering an area of 1021000 km². The orange trace outlines the Great Lakes system, the green the Ottawa River system and the red the Lower St Lawrence River tributaries that were considered in the study.

As explained in [4, 6, 7], since October 1963 Lake Ontario outflows have been regulated according to the rules of Plan1958-D (International St. Lawrence River Board of Control. 1963), which was developed and tested using the historical 1860-1954 Lake Ontario total water supplies. Since regulation began, more extreme dry and wet supply scenarios have been observed on the lower lakes, resulting in Lake Ontario water levels and flows outside the range of the existing International Joint Commission criteria.

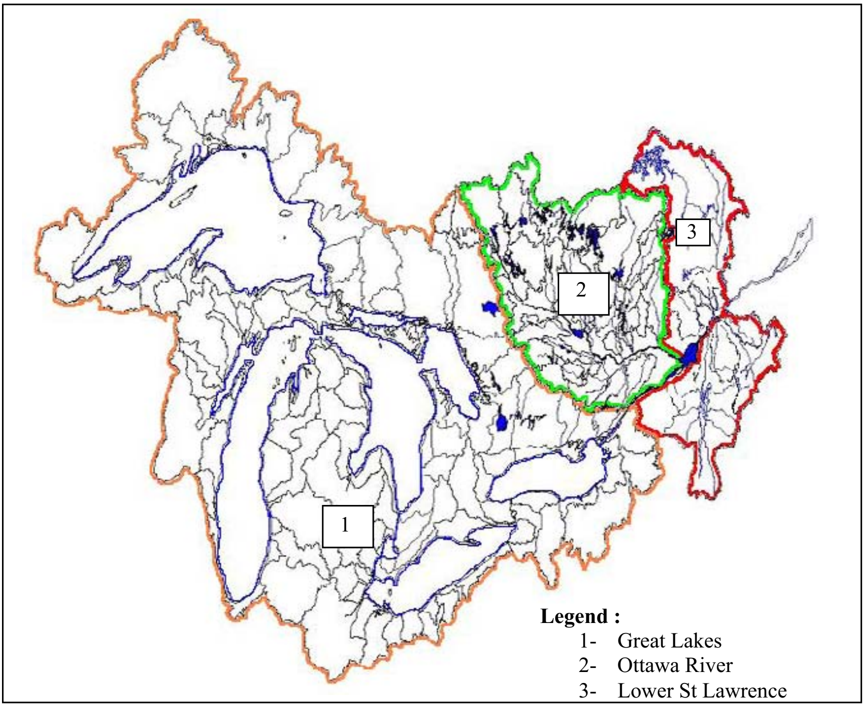

Figure 1: Chart of the Great Lakes – St. Lawrence River tributaries.

To improve the Lake Ontario multiobjective management strategies and to better understand the variability and range of plan performance, an innovative approach was used by the Lake Ontario-St Lawrence River Study (LOSLRS) of the International Joint Commission. As explained in [2], a joint effort involving the participation of an international group of experts has been undertaken to characterize the statistical properties of the hydrological system, formulate an adequate stochastic model, and simulate a sample set of 50000 years of synthetic Net Basin Supplies (NBS) for the Great Lakes and local inflows of the St Lawrence River's major tributaries upstream from Trois-Rivieres. Of particular interest to the planners was the reproduction of the temporal variability at each site (high and low flows), at each time step (annual, quarter-monthly), and the spatial characteristics (cross-correlation structure) between the Great Lakes and the Ottawa River System, the most important St Lawrence River tributary located immediately downstream.

To consider the impacts of climate change on Lake Ontario total supplies and the Ottawa River outflows, four hydrological scenarios exhibiting a wide range of plausible GCM climate changes on several climate variables, were prepared for the LOSLRS study region [2, 4, 8]. Adjusted historical meteorology (Method of Deltas) were then used to simulate four hydrologic sequences reflecting the climate change impacts over the Great Lakes and St Lawrence River tributaries, used to test the robustness of newly proposed Lake Ontario management strategies.

The present document is organized in four sections, as follows:

1. Inflow Data Analysis, presenting special characteristics of Great Lakes NBS and St Lawrence tributaries' flows at annual level.
2. Stochastic Model Formulation, including selection criteria (temporal and spatial characteristics of the inflows to be considered explicitly in the modelling approach), modelling strategies (annual, annual-monthly), and selected model results, comparing the statistical characteristics of historical and synthetic NBS series at annual level.
3. Climate Change Hydrological Scenarios, describing preparation of hydrological sequences for the Ottawa River basin.
4. Simulation of synthetic and climate change hydrological series using Lake Ontario's "Plan 1958-D with deviations" and analysis of resulting levels statistics.

2 Special characteristics of the hydrological data

2.1 Great Lakes annual inflows characteristics

Water inflows for each lake are presented as Net Basin Supplies (NBS), representing the net result of runoff from the tributaries and aquifers draining into the lake, precipitation directly over the lake, and evaporation from the lake [3, 5]. The analysed hydrological set starts in January 1900 and finishes in December 2000.

The stochastic spatial and temporal characteristics of the Great Lakes NBS are very complex. Figure 2 shows the time plot of the four Great Lakes annual NBS values along with their corresponding mean. Lakes Erie and Ontario time plots, at the bottom, suggest the presence of local non-stationarities in their annual NBS. Dry and wet sequences seem to have occurred simultaneously on both lakes, like the dry spells of the 1930s and mid-1960s, and the wet sequences of 1927-1931 and 1970s and 1980s. Very similar behaviour was found in the precipitation series (not shown).

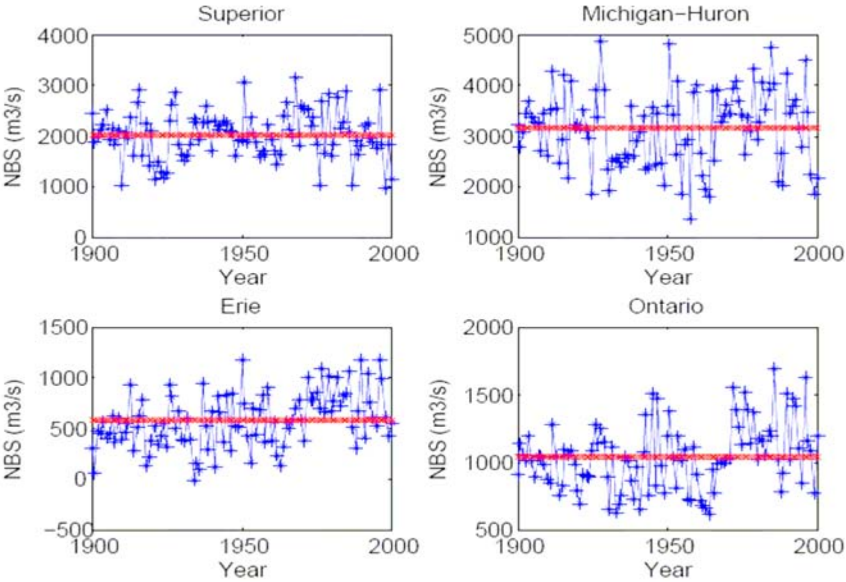

Figure 2: Time plots for the Great Lakes.

These local non-stationarities are definitively not present in the annual NBS of Lake Superior. Annual data of Lake Michigan-Huron, on the upper right part, seems to present also the dry spells of the 1930s and 1960s, and the wet sequence of the 1970s, but not as intensely as the Lower Lakes. For lakes Superior and Michigan-Huron, the lag-1 correlation coefficients are low and non significant at a 5% level of significance, and the general shape of the autocorrelation function shows a smooth decay to lag-5 [3]. On the contrary, the lag-1 to lag-4 coefficients for Lake Ontario and the lag-1 to lag-5 coefficients for lake Erie are still low but significant at the 5% level of significance. Their serial autocorrelation plot shows a slow decay for many lags.

Many techniques were applied to the Great Lakes data series (NBS and precipitation) for detecting and testing the statistical significance of these special temporal characteristics, like Lee and Heguinian Bayesian statistics, wavelets analysis, and two-state multivariate Hidden Markov procedures [3]. The following can be concluded:

- It seems that there are no significant changes in the mean of annual NBS series for Lake Superior.

- Annual NBS series for Lakes Michigan-Huron, Erie and Ontario seem to have exhibited multiple shifts in the mean. The most important change is positive, occurs around 1970 on Lakes Erie and Ontario and spans many years. Moreover, annual serial correlations for the Lower Lakes are low but significant for many lags.
- Annual cross-correlations decrease with the distance between lakes. Lake Michigan-Huron, Erie and Ontario cross-correlations are significant, varying between 0.5 and 0.7.

2.2 Ottawa River and other Lower St Lawrence River tributaries spatial characteristics

The Ottawa River system was divided into 28 sub-basins. The hydrological series were parsed to form a concomitant period starting in January 1969 and ending in December 2000, with 32 years of recorded data. Most of the sites show no significant annual serial correlation. However, three sub-basins located in the southern portion show local non-stationarities, characterised by significant serial correlation for several lags, high cross-correlation with Lake Ontario NBS, and somewhat lower correlation with Lake Erie NBS.

Four tributaries to the lower St Lawrence River, other than the Ottawa River, were selected to represent the hydrological characteristics of the intermediate basin between Cornwall and Trois-Rivieres. Their annual lag-1 serial correlations are non significant.

3 Stochastic model formulation

To evaluate the adequacy of Lake Ontario management strategies on the overall GLSLR system, it is necessary to generate concurrent NBS and flows at all 36 sites. Because the number of years of available time series is short, and the number of sites is too high to estimate parsimoniously the parameters of the stochastic generation models using all data at once, it was necessary to reduce the number of sites. The alternative of aggregating spatially the available data was chosen, in order to create homogeneous groups according some representative variables. Consequently, the whole GLSLR system was grouped into 10 regions: the four Great Lakes, five Ottawa regions and the Lower St Lawrence River tributaries region.

The system's special stochastic characteristics oriented the choice to a multivariate contemporaneous mix of Shifting Mean and ARMA process, for annual modeling of series with shifts in the mean, and regular stationary ARMA processes [3, 10–12]. The annual model characterizing the 10 regions was coupled with a disaggregation model for the annual to monthly disaggregation of flows in time and space (36 sites). To reduce the size of the disaggregation model, many strategies of decoupling the problem in stages were tested to minimize the number of parameters and maximise the explicitly preserved statistics. Finally, a *temporal-spatial strategy* using the Stedinger, Pei and Cohn disaggregation procedure for the temporal disaggregation and the Mejia and

Rouselle model for the spatial disaggregation was selected. This choice enabled use of all data. The Stedinger temporal disaggregation model, being contemporaneous, produces the best results with the different sample lengths.

3.1 Routed series

A generated set of 10000 years of NBS series for Lakes Superior, Michigan Huron and Erie using the selected modeling strategy was routed with the Coordinated Great Lakes Regulation and Routing Model [5], along with initial lake levels, and series of channel ice roughness data and diversions. The final output is the routed monthly or quarter-monthly series of outflows and levels for Lake Erie.

Lake Erie simulated outflows and Lake Ontario NBS were then combined and routed using the Lake Ontario pre-project outlet conditions [1]. This set of Lake Ontario outflows and levels values constitutes a reference basis for comparison of the simulated outflows using historical NBS series and routed outflows and levels from the generated NBS series.

Figures 3 and 4 show Empirical Frequency Plots of 100 samples of 100 years of the routed annual average outflows calculated from routed generated and historical data. Figure 3 illustrates an enhanced view of the upper tail of Lake Erie's average annual outflows. The red curve illustrates the sorted observed annual outflows while the blue curves show routed synthetic outflows series. Lake Erie results summarize the characteristics of the Upper Lakes. The graph shows a good agreement with the historical series and a lot of variability of the samples around the observed series. Figure 4 shows the upper tail of routed annual average outflows at Lake Ontario, calculated from quarter-monthly data. The results are presented in tens of m^3/s (*10 m^3/s). Although all frequencies were reproduced, there is less uniformity in the dispersion in the high tail.

Once the statistical characteristics of the generated series were assessed, four 101-year long Lake Ontario net total supply (Lake Erie outflow plus NBS) series were extracted to develop and test newly proposed Lake Ontario management strategies under "different than the observed water supply conditions". Those sequences exhibit the lowest and highest 5-year moving average supply, the largest range from wet and dry supplies, the longest Lake Ontario drought, and a sequence closer to the average but with a different annual pattern than the observed one [5, 7]. The full 50000-year stochastic series were used to assess the variability and range of the management strategies performance. Some descriptive statistics using "Plan 1958-D with deviations" will be presented in the last section.

4 Climate change scenarios

To consider the possible impacts of climate change on inflows to the different major basins of the Lake Ontario – St Lawrence River system, and resulting water levels and outflows, four hydrological scenarios were prepared with meteorological outputs from individual General Circulation Model GCM

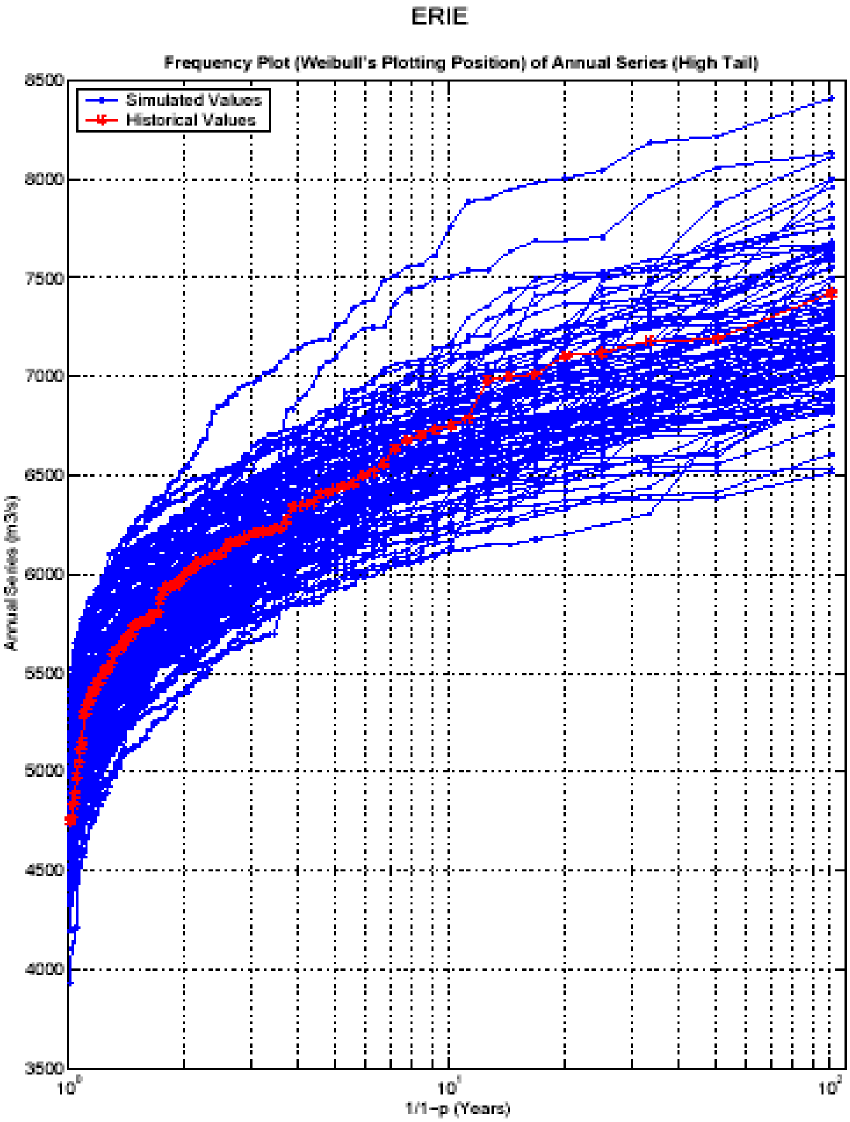

Figure 3: Empirical frequency plots of 100 samples of 100 years simulated annual average outflows vs historical outflows – Lake Erie (calculated from monthly values).

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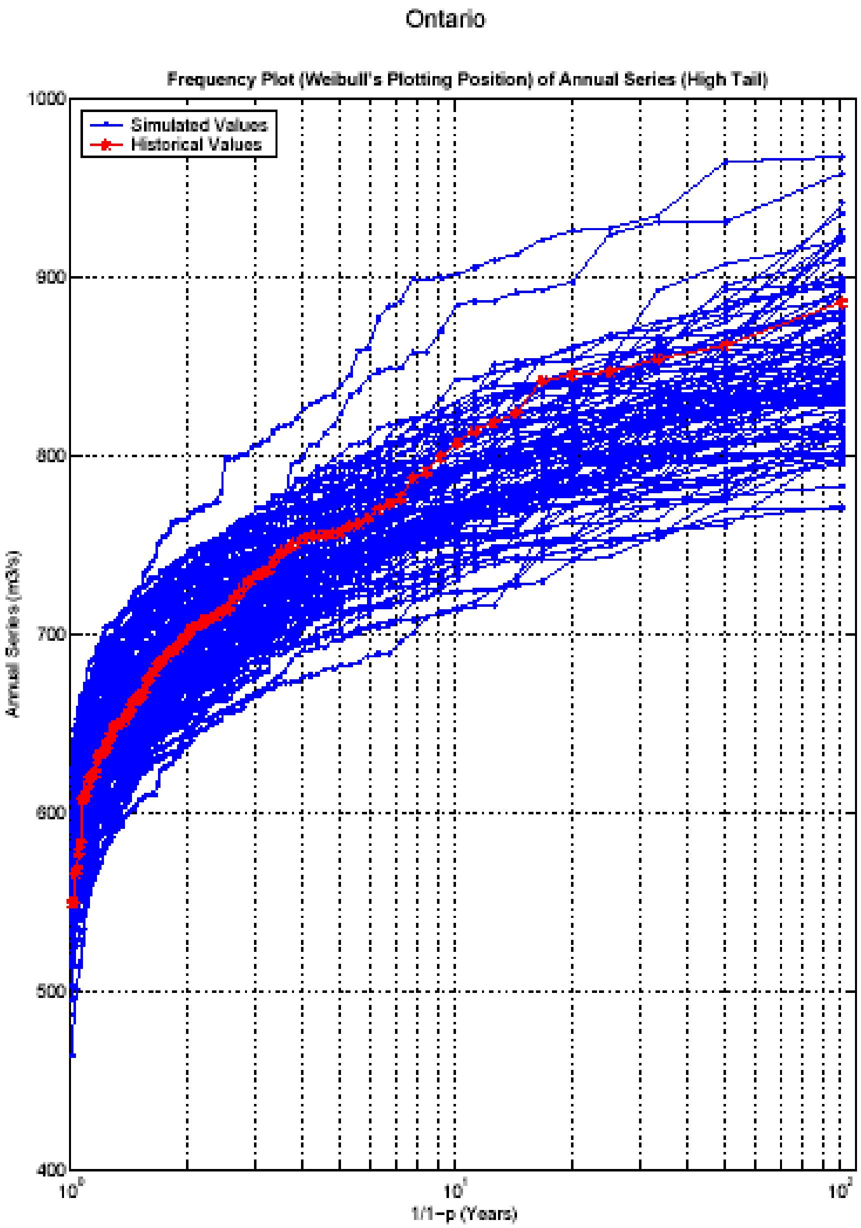

Figure 4: Empirical frequency plots of 100 samples of 100 years simulated annual average outflows vs historical outflows – Lake Ontario ((calculated from quarter-monthly values).

simulation results. The Hydrology and Hydraulics Technical Working Group of the LOSLR Study selected four climate scenarios exhibiting 1) most warming and wettest, 2) most warming and driest, 3) least warming and wettest, and 4) least warming and driest conditions, with the goal of "boxing the uncertainty" [8]. Downscaling of the GCM scenarios was limited to interpolation of the GCM grids.

The Great Lakes Environmental Research Laboratory GLERL (Ann Arbor, US) acquired the identified GCM climate change scenarios and supplied results to hydrologic modellers: monthly GCM output changes between the baseline period and the future 30-year period (2040-2069). GLERL adjusted historical meteorology data for the Great Lakes basin (spanning the 1950-1999 period) with the GCM climate changes, and used hydrological runoff and lake model of the Great Lakes to simulate NBS scenarios for the base case, that is local inflows series resulting from observed climatology, and each climate change scenario [2, 5]. The Great Lakes NBS series for the base case and each of the four changed climate scenarios were then routed with the Coordinated Great Lakes Regulation and Routing model to prepare five sets of Lake Erie outflows and net total supply series for Lake Ontario. Hydro Quebec and the Quebec Ministry of Environment, from Canada, prepared the base case and four climate change sets of intermediate flows to each of the 40 Ottawa River designated sub-basins. The observed meteorological data were obtained for the 1962-1990 period. The five sets of sub-basin inflows were then routed to the river mouth, at Carillon, using the MENVIQ routing model and adequate management rules.

The following color map, Figure 5, shows the spatial variation of the mean annual runoff for the most warming and driest climate scenario. The map shows mean annual values by surface unit, for resulting inflows of the overall Great Lakes and Ottawa River System. In spite of different hydrological and routing models used for the Great Lakes and Ottawa River sub-basins, the color maps shows overall harmonious behaviour of the two systems. The transition at the frontier of the two systems is very smooth. Reference [2] illustrates Lake Ontario net total supply annual and quarter-monthly characteristics for the base case and the climate change scenarios. The following section describes more detailed results of the climate change analysis on the Ottawa River system.

4.1 The Ottawa River system

The analysis of the climate variables shows that the resulting annual mean total precipitation (liquid rain plus snow) for the four climate change scenarios were higher than the observed record. Because the annual maximum and minimum temperatures also increase, the evapotranspiration is higher for all scenarios. The net impact on inflows is generally an increase in the Ottawa River system. For one scenario (warm/dry), the annual mean corresponding inflow is lower than the base case.

Monthly histograms of differences between the climate change scenarios and the base case show increasing rainfall in winter months for all scenarios, lower snowfall amounts in January, February, March and April for the one of the GCM scenarios, and an overall increase of total precipitation.

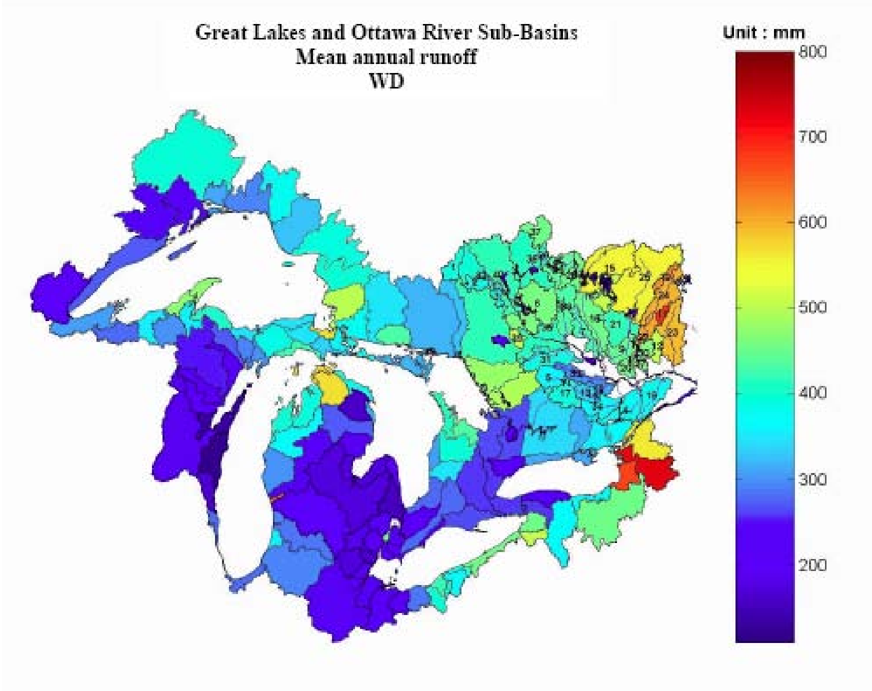

Figure 5: Great Lakes and Ottawa River spatial variation of the mean annual runoff for the most warming and driest climate scenario.

The routing of the Climate Change hydrological scenarios showed that MENVIQ Routing Model management rules based on the historical flows were not able to respect the actual storage volume and reservoir water level requirements. The drift to early spring floods and more flow peaks during the winter months that characterize the climate change inflows hydrographs produce higher discharges in late winter months than the outflows set according to the existing management rules, which specify that reservoir levels should be emptied in preparation for the spring flood. Consequently, the reservoir storage could not be adequately filled with the spring floods, and the expected reservoir water levels during the summer season were hard to reach.

Changes to the reservoir management rules were necessary, but it was impossible to anticipate the management outflow and level constraints required to prepare an optimal set of management rules for each climate change scenario. Finally, it was decided to develop a general adjustment to existing management rules for all scenarios, respecting the actual set of level and outflow constraints of the system. It was established that the spring flood shifts by about -30 days on average, for the four CC scenarios and all sub-basins. The management rules were consequently modified to better respect the operational constraints by shifting constraints dates by 30 days.

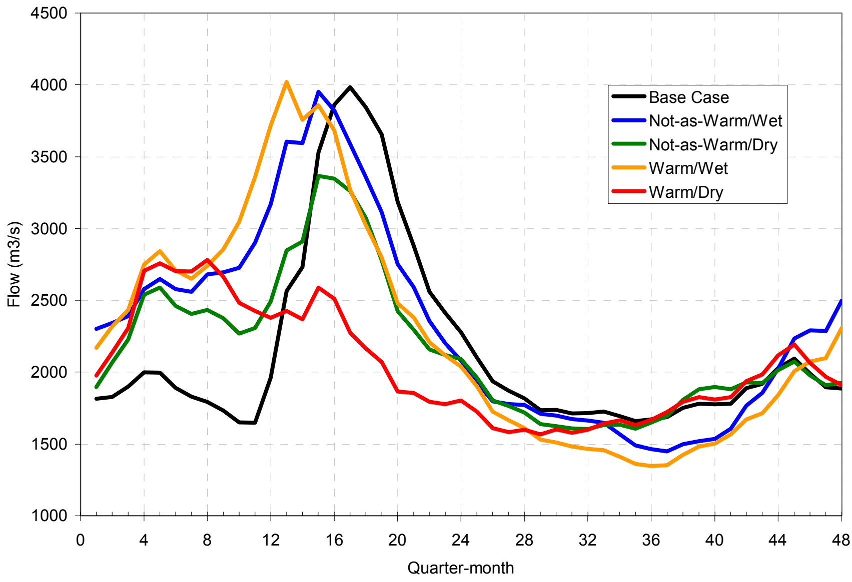

Figure 6: Comparison of quarter-monthly outflows Averages at Carillon, the outlet of the Ottawa River system.

Figure 6 shows the routed quarter-monthly mean outflow hydrographs for the base case and each of the four CC scenarios at Carillon. The simulated hydrographs show a similar increase of the average mean outflow during January and February for all four scenarios, a drift to earlier spring floods (resulting from higher mean total precipitation and annual minimum temperature increase, producing increasing rainfall in winter months), but major differences between the CC scenarios during March and April.

5 Lake Ontario management strategies

The LOSLR Study has introduced a new planning development vision combining scientific work, constraints and preferences of the affected interests, and public input in an interactive analytical framework that resulted in the formulation and performance analysis of many possible management strategies or plans for Lake Ontario. The performance of each candidate Lake Ontario regulation plan was simulated with synthetic and climate change hydrological series and the different sets of outflows and levels that resulted were extensively analysed with specialised models of ecosystem response, shoreline erosion dynamics, and economics that described the trade off between benefits and losses associated with recreational boating, hydropower and commercial navigation. The modelling effort provided a better understanding of the performance of the regulation plans, as well as gains in knowledge of the effects of varying water levels on shoreline erosion, wetland plant communities and other elements of the ecosystem. For more detail, see [7].

Table 1: Lake Ontario quarter-monthly mean level statistics for the historic NBS case and selected stochastic centuries simulated with Plan 1958 D with deviations.

	Historic case	**C1**	**C2**	**C3**	**C4**
Average (m)	74.74	74.59	74.78	74.76	74.49
Maximum (m)	75.68	75.72	76.16	75.68	75.48
Minimum (m)	73.78	73.04	73.52	74.06	73.26

For the past several decades the outflows of Lake Ontario have been regulated by a Board of Control according to the rules set out in Plan 1958 D as well as their own discretionary judgement. A plan called "Plan 1958 D with deviations" was developed to simulate this present regulation regime with the different supply sequences described in the preceding sections. This plan was then used as the basis to compare the outcomes of the new candidate regulation plans. Table 1 from [5] summarizes some descriptive statistics of Lake Ontario simulated quarter-monthly levels using "Plan 1958 D with deviations" and the four selected 101-year long Lake Ontario net total supply series described in subsection 3.1 (C1, lowest 5-year moving average Lake Ontario supply; C2, highest 5-year moving average, which also had the largest range from wet to dry supplies; C3, a sequence with a similar range and average of supplies as the historical; and C4, the longest Lake Ontario drought). The simulation results for the longest synthetic Lake Ontario drought (C4), and the lowest 5-year moving average Lake Ontario supply (C1), show the lowest levels. Table 2 shows climate change scenarios statistics. The simulated levels with the Not as Warm and Wet scenario are close to the base case, while the Warm and Dry scenario simulated levels are the lowest.

Table 2: Lake Ontario quarter-monthly mean level statistics for the climate change scenarios simulated with Plan 1958 D with deviations.

	Base Case 1962-1990	**Warm and Dry**	**Not as Warm and Dry**	**Warm and Wet**	**Not as Warm and Wet**
Average (m)	74.84	74.37	74.59	74.52	74.76
Maximum (m)	75.53	75.06	75.24	75.39	75.54
Minimum (m)	74.22	72.71	73.35	73.01	74.08

6 Conclusion

The LOSLR Study not only introduced a new planning development vision, but also a sophisticated hydrological analysis using synthetic flow samples and climate change affected series. The objective was to develop robust Lake Ontario management strategies that perform adequately under a wide range of conditions.

The formulation of a complete hydrological modeling strategy using adequate models was one of the major challenges in the study. The team of international experts that was organised developed an innovative approach to capture the variation in hydrological conditions that any regulation plan selected for Lake Ontario regulation may have to deal with in the coming decades. The major achievements in terms of innovation are as follows.

- The use all available tools to assess the observed changes in the mean of Great Lakes precipitation and NBS series;
- The application of the Contemporaneous mix of Shifting Mean SM model and CARMA to the Great Lakes – St Lawrence System coupled with multi-stage disaggregation strategies, for generating NBS and synthetic flows series for the overall system.
- The coordinated use of many hydrological rainfall-runoff models, lake models, routing models with appropriate management rules, to prepare climate change hydrological NBS and flows series.
- The development of effective statistical and graphical frameworks to compare different modeling strategies, and visualize temporal and spatial results in an organized way.

The work of the LOSLR Study was accomplished using an open, collaborative process with the participation from multi-disciplinary scientists, engineers, economists and non-expert members of the public that are concerned about Lake Ontario outflow regulation. The outflow regulation options presented in the final report to the International Joint Commission all offer an improvement in the overall economic and environmental benefits of Lake Ontario outflow regulation but differ in the distribution of those benefits to the different interests. The wide array of plausible hydrological conditions used to develop and evaluate these plans adds to the confidence that the decision-makers have that their selected plan will perform as expected.

Acknowledgements

The work described in this paper would not have been completed without the participation, support and guidance of many specialists. In particular, deep gratitude is extended to Dr. Oli Sveinsson, for his tireless support with the stochastic modeling software. It has been a privilege and opportunity to discuss stochastic models and modeling strategies with such insight. Also, acknowledgements go to Hydro-Québec colleagues R. Guay, D. Sparks, D. Tremblay and M. Latraverse for their collaboration and contributions. Heartfelt thanks go to T. McAuley, A. Carpentier, T. Sassi and Dr. Y. Fan.

References

[1] Caldwell R., and Fay D., *Lake Ontario Pre-project Outlet Hydraulic Relationship*, Environment Canada, Cornwall, Ontario, 2002.

[2] Croley, Thomas E. II, *Great Lakes Climate Change Hydrologic Impact Assessment IJC Lake Ontario – St. Lawrence River Regulation Study*. NOAA Technical Memorandum GLERL-126 NOAA, Great Lakes Environmental Research Laboratory, Ann Arbor MI 48105, September 2003.

[3] Fagherazzi L., Guay R., Sparks D., Salas J. and Sveinsson O., *Stochastic Modeling and Simulation of the Great Lakes – St. Lawrence River system.* Report prepared for the Lake Ontario-St. Lawrence River study of the International Joint Commission, Ottawa and Washington, 2005.

[4] Fagherazzi L., Guay R., Sassi T., Climate *Change Analysis of the Ottawa River system. Report prepared for the Lake Ontario-St. Lawrence River study of the International Joint Commission*, Ottawa and Washington, 2005.

[5] Fay D. and Fan Y., *Hydrologic scenarios for the evaluation of Lake Ontario outflow Regulation Plans*, paper presented to the CWRA Conference, Toronto, 2006.

[6] Fan Y. and Fay D., *Final report on the development of empirical relationships to estimate water levels of the St Lawrence River from Montreal to Trois Rivieres*, Report prepared for the Lake Ontario-St. Lawrence River Study of the International Joint Commission, Ottawa and Washington, 2002.

[7] International Joint Commission, *Options for managing Lake Ontario and St. Lawrence River Water Levels and flows*, Final Report by the International Lake Ontario – St. Lawrence River Study Board, March 2006.

[8] Mortsch, L., Alden M., Klaassen J., *Development of Climate Change Scenarios for Impact and Adaptation Studies in the Great Lakes – St. Lawrence Basin*. A Report prepared for the Hydrologic and Hydraulic Modeling Technical Working Group, International Lake Ontario - St. Lawrence River Study Board, International Joint Commission, Toronto, ON. (Adaptation and Impacts Research Group, Meteorological Service of Canada), 2005.

[9] Ripple Effects, Public Information Advisory group (PIAG) of the International Lake Ontario – St Lawrence River Study Board, July 2001

[10] Salas, J.D., Saada, N., Chung, C.H., Lane, W.L. and Frevert, D.K., *Stochastic Analysis, Modeling and Simulation (SAMS) Version 2000 - User's Manual,* Colorado State University, Water Resources Hydrologic and Environmental Sciences, Technical Report Number 10, Engineering and Research Center, Colorado State University, Fort Collins, Colorado. 2000.

[11] Sveinsson O., Salas J., *Stochastic Modeling And Simulation Of The Great Lakes NBS Based On Univariate And Multivariate Shifting Mean* – Dept.

of Civil Engineering, Colorado State University, Fort Collins, January 2002.

[12] Sveinsson, O., and Salas, J.D., *Multivariate Shifting Mean Plus Persistence Model for Simulating the Great Lakes Net Basin Supplies*, Submitted to a Journal.

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Section 2
Hydrological modelling

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Commercial water auditing in Stella

S. J. Mecca[1] & S. D. Mecca[2]
[1]*Department of Engineering-Physics-Systems, Providence College, Providence, USA*
[2]*WaterWise Technologies, Jamestown, RI, USA*

Abstract

Stella has been successfully used to model water supply systems [1] including the representation of residential demand [2] in such systems (Stella is a product of ISEE Systems, Hanover, NH USA.) This paper documents the use of this modeling tool in simulating demand from several appliances common to commercial water users and applying this technique to commercial water auditing. The application of the model to the audits of six commercial organizations is presented. The model is being used for water management/conservation studies but also has potential for representing commercial water demand in more comprehensive supply system models.
Keywords: water audit, commercial water auditing, commercial water demand, water management, water conservation.

1 Motivation

The management of water resources has required extensive modelling with reliable data for both validation and decision making. Information regarding end-use consumption in residential, commercial, industrial and governmental sectors is essential in comprehensive water resource models. Yet, reliability has often meant metering, which, on a distributed basis, can be expensive and time consuming. In the residential sector, guidance on water demand can often be gleaned and scaled from large studies such as the AWWA REUW study, which spanned several areas and districts around the United States [1]. This data has been successfully scaled to a small municipal system in Jamestown, Rhode Island in the US by normalizing computed demand to billing audit data for this community [2]. The effort was motivated by a need for representing residential end-use of water in a comprehensive dynamic-stochastic water management

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model of the system [3]. A segment-representation of the model is shown in Figure 1. It had been economically unfeasible to acquire such end-use data through a distributed monitoring system in this small island community. Nevertheless, the normalized end-use data successfully represented the residential consumption in the model. The commercial sector is more challenging in that the uses of water in this sector are varied depending on the nature of the various commercial enterprises comprising the sector. For example, a commercial boating and marine operation will have a different mix of uses than will a dentist's office or a printing press or a restaurant. Nevertheless, there is a more common set of appliance-related uses, such as toilets and urinals, faucets and showers. Demand from these applications is expected to be dynamic and will depend on factors such as the number and efficiency of appliances, number of employees, patrons, etc.

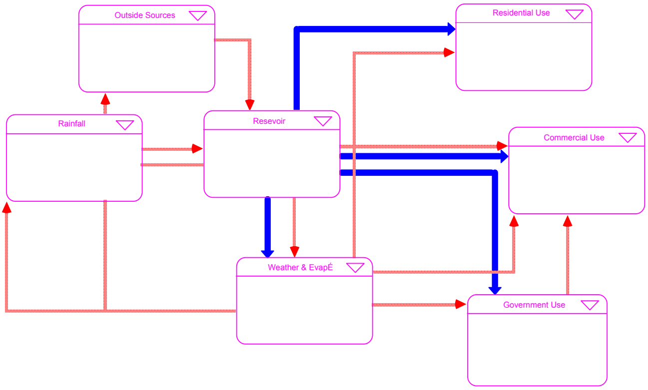

Figure 1: The major segments of a dynamic-stochastic model of a municipal water system [3].

Hence, one motivation for the present study is to address the representation of commercial end-use demand with a view towards upgrading that sector in the aforementioned dynamic-stochastic model. The impetus for one of the authors (SJM) was the (eventual) temporal and functional aggregation of water use in the commercial consumption segment of the model, a task that has been undertaken in the residential sector. See references [2] and [4] and references therein.

The motivation of the other author (SDM) was very practical. As part of an organization (WaterWise Technologies) in the business of providing water management consultation services to industrial, commercial and governmental organizations, he does extensive water auditing prior to developing a strategy of water management opportunities for clients and prospective clients. These audits have multiple dimensions and often involve reconciling billing data with calculated end-use patterns. Rather than providing long term averages, the audits are frequently expected to display dynamic patterns of water use. The effort involved in developing individual commercial audits as well as the need for including temporal analyses was the motivation to consider developing a dynamic commercial water audit that could be tailored to fit most if not all commercial users.

The model, depicted in segment form in Figure 1 and reported in earlier studies [2, 3], is written in Stella. This tool, which is widely used in system dynamics, was an appropriate choice for the present study especially since it might ultimately support the commercial demand segment of the aforementioned dynamic-stochastic municipal water resources model.

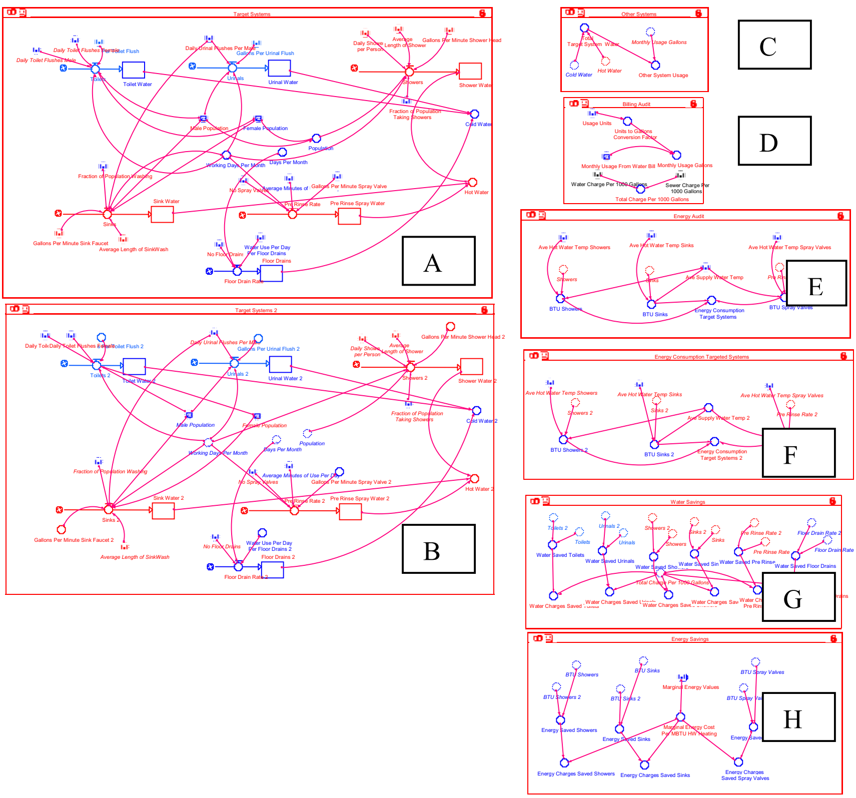

Figure 2: Overall structure of the audit model. See text.

2 Structure of the audit model

The commercial audit requires consideration of a common set of end-uses in this sector. It was decided to include use for six appliances somewhat universal in commercial enterprises: toilets, urinals, sink faucets, active drain-traps, showers and pre-rinse kitchen spray valves. While there are other common end-uses, these elements were chosen to reflect a mix of typical water conservation technologies. However, the model has the capacity to readily add other 'common' end-use applications. The tool also has the ability to represent non-common uses, i.e. applications that are more or less unique to a particular enterprise e.g. a cooling water application.

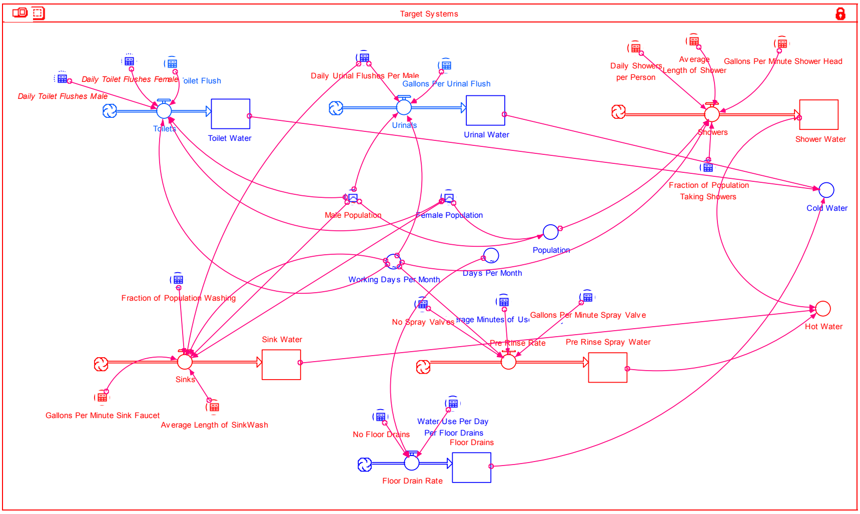

Figure 3: Graphic structure of the targeted systems segment of the model in Stella.

The audit structure is shown in Figure 2. The aforementioned common end-uses are treated in segments A and B, which are labelled as 'Targeted Systems', A being used to describe the existing situation and B a proposed situation using conservation technologies, labelled as Targeted Systems P. This allows the audit to be used for exploring the potential of such technologies in targeted systems. Segment D considers water use from billing audit data and is labelled, 'Billing Audit', while section C, labelled as 'Other Systems', calculates aggregate water use from processes other than the common or targeted ones by subtracting this use from the total use found in the billing audit . (As noted earlier, this use can be disaggregated by adding custom modules for unique applications, though this is not the focus of the present paper.) Some water use, especially hot water, has corresponding energy use and this is calculated in segments E and F for Targeted Systems and Proposed Systems, Targeted Systems P. Finally, segments G and H compute water and energy savings respectively. Segments A and B have exactly the same structure and this is shown in Figure 3. The six targeted – common commercial uses are shown together with the factors affecting water use rates for these appliances. The parameters – male, female and total-population, days per month and working days per month – impact most of the six common uses. The parameters used in the model are, for the most part, self-documenting, i.e. parameter names are chosen with their commonly understood definition. These may be described functionally or graphically, the latter being the most common way. For example, the male population as a function of time of year can be input as a table, which is then represented as a graph in the model. These features make Stella a rather user friendly tool.

The ability to create a separate user interface makes the process even more approachable for the casual user while simultaneously isolating the user from the model layer of the audit. This interface is shown in Figure 4. The multiple pages of the interface level allow for the entry of most of the parameters in the model. Appropriate limits are set to trap user typographical errors. The full interface screen gives basic outputs of the model in both graphic and tabular form.

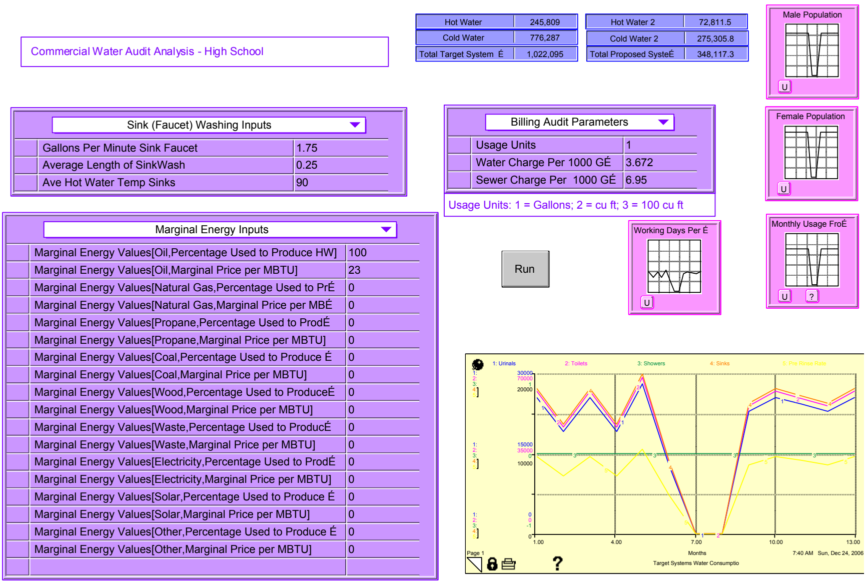

Figure 4: User interface for the audit model in Stella.

3 Applications

Six commercial enterprises were selected and water audits for common uses were carried out. The organizations and their computed consumption for common uses are summarized in Table 1.

The selected organizations focus on different functions and serve different populations. Within a few percent, total consumption, determined via a utility billing audit, was accounted for in the computed consumption for the YMCA, Audubon and Marina operations. The Marina operation, which uses much more water for its entire operation than what is shown in Table 1, had separate metering for the functions analyzed in the present study. The college dormitory was not separately metered and the high school computed consumption was 71% of its total consumption; other non-common water use functions are served there. The motel likewise used water for other purposes besides that used by clients in their rooms. The audit does not account for laundry and some staff kitchen uses.

Table 1: Summary of selected organizations.

Organization	Description	Computed Annual Consumption for Common Uses (gallons)
YMCA	Entire single structure athletic & administrative complex	1,911,153
Audubon	Education center	132,058
College Dorm	Two-building dormitory complex	1,083,186
High School	Public secondary school	1,022,095
Marina	Segment of a full marina complex	76,145
Motel	12-unit motel complex	316,077

It is tempting to hypothesize that common-use water consumption in the commercial sector should be proportional to the average population served per day especially since the uses chosen for the common-use set in this study are appliances used by or for individuals. The scatter plot shown in Figure 5 gives the computed annual consumption as a function of average population served for the six cases included in this study.

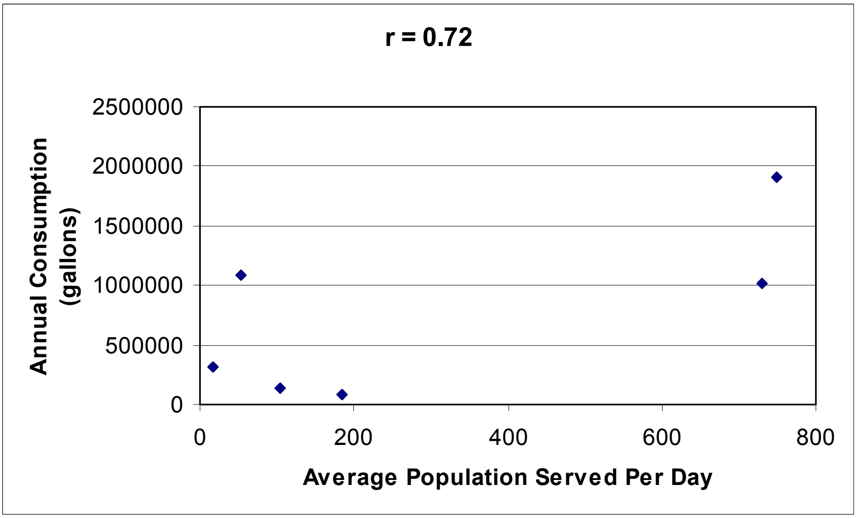

Figure 5: Computed annual consumption as a function of average population served for the six cases included in this study.

The correlation coefficient, r, is 0.72; assuming unrelated variables, the probability for $r \geq 0.72$ is about 10% for this set. It is clear that reasonable estimation of commercial consumption, even consumption from appliances related to individuals, requires more than a simple over-riding parameter such as population served. Hence, the model presented herein includes parameters related to the appliances used, the frequencies and durations of use and differences based on gender and time. For each case, the relevant input factors were either directly measured or inferred from interview and observation. Monthly simulations were run for each case. Results for the common uses for each organization are shown in the graphs of Figure 6.

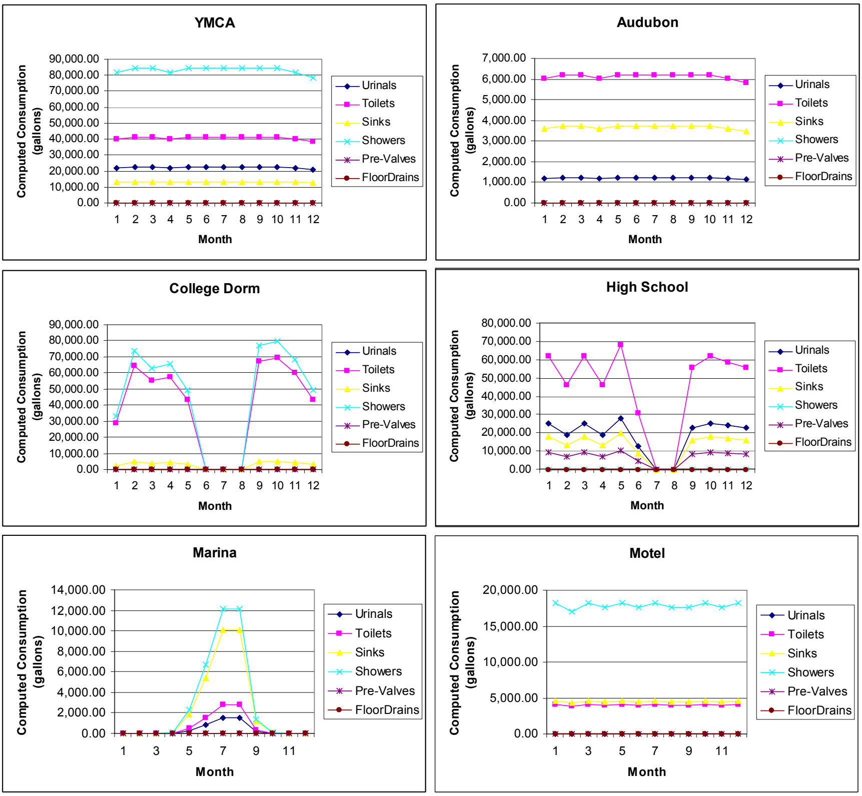

Figure 6: Computed use for targeted appliances in the 6 cases analyzed with the commercial water audit used in this study.

Several things are apparent in these results. Some operations have rather constant monthly water use; others show seasonal variations that depend on the schedule, e.g. the college dormitory and the public high school, which are generally closed during the summer months and the marina, which has its major operation during the summer months. The amount of water use for each appliance varies by enterprise both in order and relative intensity.

The audit model reported in this study is capable of computing more than present water use for targeted systems. Among the other computations and analyses are:

- Water use computed assuming implementation of available water conserving devices.

- Estimation of 'other water uses' by subtracting computed values for common appliances from consumption determined from a billing audit.
- Estimation of energy used to produce hot water for common appliances where applicable.
- Estimation of energy to be saved from implementing the aforementioned water conserving devices.

Results for these additional computations are outside the scope of this paper. Nevertheless, it is interesting to note the projected water use for similar operations using generally available current technologies for water conservation for the six cases used in this study. These are summarized in Table 2 below.

Table 2: Computed present and projected consumption with annual water savings.

Organization	Present Computed Annual Consumption for Common Uses (gallons)	Projected Computed Annual Consumption for Common Uses with Water Conserving Devices (gallons)	Savings (gallons) Percentage savings (%)
YMCA	1,911,153	800,925	1,110,228 (58%)
Audubon	132,058	44,028	88,030 (67%)
College Dormitory	1,083,186	358,566	724,620 (67%)
High School	1,022,095	348,117	673,978 (66%)
Marina	76,145	22,251	53,894 (71%)
Motel	316,077	117,717	198,360 (63%)

The savings projected in Table 2 are significant as is the investment potential. Simple paybacks for both water and energy in order from shortest to longest are as follows: YMCA - 17 months, Motel - 26 months, High School - 35 months, Marina - 40 months, and Audubon - 76 months. The College Dormitory Complex had two buildings one of which had a 12 month simple payback and the other a 40 month payback on investment. Payback periods generally depend on the user to fixture ratio (for toilets and urinals) and the pattern of showerhead use; the dorm with the low payback period had an average user to fixture ratio much higher than the one with the longer payback time. In addition, much of its water use came from showerheads which have the benefits of low replacement cost and significant energy savings. The energy savings impact on the total conservation opportunity is a factor that is well documented by deMonsabert and Liner [5].

4 Conclusions

A dynamic model written in Stella and used to compute water use from six common appliances has been developed and applied to six commercial enterprises. The model was originally motivated by a need to augment the commercial water demand sector of a previously reported municipal water system model [3] and by a separate need to represent water use to commercial clients who are considering several generally available water management strategies in connection with their operations. For the six cases studied, there is sufficient variability of demand in both time and relative intensity even for common-appliance uses to suggest the value of a dynamic model that disaggregates such uses enterprise by enterprise. While further analyses are required before the present audit can be used in a summative approach to representing overall commercial demand in the aforementioned municipal water system model, it has provided some insights into that problem. It has met the second need for representing water consumption and water management strategies for individual commercial water users.

Acknowledgements

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References

[1] Mayer, P.W., DeOreo, W.B., Opitz, E.M., Klefer, J.C., Dziegielewski, B., Davis, W.Y., Nelson, J.O., Residential End Uses of Water, v-xxxvii, 1999 American Water Works Association Research Foundation.
[2] Mecca, S.J. & Dotto, S. Normalizing residential water demand to water consumption, *Water Resources Management III*, edited by M. De Conceicao Cunha, University of Coimbra, Portugal and C.A. Brebbia, Ecology and the Environment Vol. 80, UK, 2005.
[3] Mecca, S.J. & LaMontagne, R. Dynamic-Stochastic Modeling of a Municipal Water Supply System in Stella, *Water Resources Management II*, edited by C.A. Brebbia, Progress in Water Resources Vol. 8, UK, 2003.
[4] Guercio, R., Magini, R. & Pallavicini, I., Temporal and spatial aggregation in modeling residential water demand, *Water Resources Management II*, edited by C.A. Brebbia, Progress in Water Resources Vol. 8, UK, 200.
[5] S. deMonsabert and B. Liner, Watergy: A water and energy conservation model for federal facilities, CONSERV'96, Orlando, Florida, January 6, 1996. http://www1.eere.energy.gov/femp/pdfs/watergy_manual.pdf

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Hydrologic modeling support for sustainable water resources management

R. G. Quimpo
University of Pittsburgh, Pittsburgh, PA, USA

Abstract

Prior to the implementation of an infrastructure project, an environmental impact assessment is necessary to secure the acceptance of stakeholders and the approval of various regulating agencies. Since the project has not yet been constructed, the projected impacts are based on surveys and modeling studies that need to be verified during and after construction. In projects which impact the water resources of a region, planners and designers use hydrologic models to predict future conditions on water quantity and quality. This paper will focus on the hydrologic modeling, instrumentation and monitoring requirements using as case study the design and construction of a stretch of the Interstate system of highways in the United States.

The highway segment traverses agricultural, forest and game lands resulting in the alteration of landscapes and changes in watershed delineations. In some sections of the project, the highway cross-section includes an infiltration gallery constructed under the roadway which permits groundwater flow from headwater areas to bypass the construction corridor. The behavior of the altered watersheds needs to be examined to determine if they conform reasonably with projected performance. Aspects of the study related to runoff modeling and prediction, storm water sediment and erosion practices, wetland and stream restoration were investigated. The logistics of instrumentation, monitoring, data-acquisition and analyses are discussed. The findings will form the basis for developing guidelines for use by the implementing agency in future projects.
Keywords: hydrologic models, hydrologic monitoring, erosion and sedimentation, environmental impact, infiltration gallery, stream restoration, watersheds, wetlands, BMP, watersheds.

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1 Introduction

In the last quarter century, the tenet of environmental responsibility has been embraced almost universally. Even less developed countries whose primary concern is the improvement of the population's standard of living have implemented programs to protect the environment. In countries with proactive population, non-governmental groups exert political pressure on government institutions to focus attention on potential risks to the environment when economic development projects are proposed. As a result, regulatory agencies have been set up to monitor the planning, design and implementation of projects.

Formally, environmental impact assessment (EIA) is a planning tool that is now regarded as an integral component of sound decision making. Its purposes include the support of the goals of environmental protection and sustainable development; the integration of environmental protection and economic decisions and the assessment of plans to mitigate any adverse impacts; and providing for the involvement of the public and government agencies in the review of proposed activities in the project. In the United States, EIA is pervasive and has filtered down from the federal and state to municipal levels of government.

To mitigate the conflict between projected economic benefits and adverse environmental impacts, the concept of sustainability has evolved. The Bruntland Report [1] defines sustainable development as that which "meets the needs of the present generation without compromising the ability of future generations to meet their own needs." This definition implies limitations imposed by the state of technology and social organization on the environment ability to meet present and future needs.

2 Need for modeling support – a case study

In assessing water quality, the assessment tools require that hydrologic variables be quantified. While general hydrologic models are useful for the qualitative analysis of overall project impacts, if it is desired to investigate specific mitigation strategies, it is necessary to develop a site-specific model of hydrologic interactions that result from the project. This paper examines the needs for quantitative hydrologic modeling and how they were addressed in assessing the environmental impact of highway construction. The case study derives from the use of new technology in the design and construction of a segment of Interstate-99 in Pennsylvania in the United States. The project was enabled by federal legislation and is intended to improve the transport capacity of the existing highway routes and facilitate product mobility in the region. The route of the highway passes through headwater regions of forests and farms and straddles protected game lands. The construction project is managed by the Pennsylvania Department of Transportation (PennDOT).

The highway was designed with the assistance of consulting engineers following design guidelines developed by PennDOT. In addition to the consultants, PennDOT has a complement of planners, engineers, environmentalists and social scientists who as a matter of policy use the available technology in all the projects that the agency undertakes, subject to cost and environmental constraints. Environmental protection legislation regulates many of the planning procedures. State and federal agencies have developed guidelines regarding the design of highways and their appurtenant structures in order to minimize the impacts of such projects.

Pursuant to EIA guidelines, PennDOT sought the opinion and advice of all the stakeholders. To keep all stakeholders fully informed, a policy of strict transparency was adopted to dispel any fears and apprehensions that constituencies typically experience whenever changes in their environment are proposed. Often conflicts arise between the designers and the constructors on one side and regulating agencies, who are often supported by public interest groups and local community movements, on the other side.

The design of the highway incorporated many technologies that the designers believe will improve the project's effectiveness in minimizing adverse environmental impacts. In the design and construction, both source and treatment Best Management Practices (BMPs) for erosion and sedimentation control were adopted. These include installation of silt fences during construction, detention and sedimentation ponds to capture sediment and runoff from the construction site, wetland mitigation and stream restoration activities. Also included is a new feature in the design of the highway cross-section – the incorporation of an infiltration gallery constructed underneath the roadway. The gallery was designed to direct groundwater from upstream of the highway to the area downstream and bypassing the area that is disturbed by construction.

Plans for the project went through federal and state regulatory scrutiny. Public hearings and town meetings were conducted to provide opportunities for the public and all stakeholders to voice their concerns. Although no specific objections to the design were raised, a consensus on the overall impacts of the project on the receiving waters and wetlands downstream was difficult to achieve. Eventually, a compromise was arranged to resolve the remaining issues. PennDOT agreed to have an independent group conduct a verification of the effectiveness of the technology used in the design. PennDOT contracted with the University of Pittsburgh to carry out a testing and monitoring program to investigate the environmental impact of the project.

In examining the effectiveness of the design, hydrologic modeling support is necessary. We will focus on two technologies. These are the use of detention and sedimentation ponds and the incorporation of the infiltration gallery constructed underneath the roadway.

3 Hydrologic setting

The drainage of hill slopes is characterized by small gullies and creeks which run downstream from the ridge to the valley floor. The gullies generally have steep slopes and would drain relatively narrow strips of area until the gullies reach a flatter topography when they combine to form creeks which become tributaries of the main drainage channel. The main channel would run almost parallel the ridge line in the direction of the downstream gradient. A highway, which runs parallel to the ridge, would intercept the steep-sloped channels and form creeks which drain at the bottom of the valley approximately parallel to the ridge line along a downstream gradient.

In this setting highway construction significantly alters the watershed. As a result of the project, many small watersheds were subtended by the highway. These small watersheds would cover only several tens or a few hundred acres. Since there would be many of these sub-watersheds throughout the length of the construction project, only two watersheds were investigated. The locations of the two test watersheds are shown in Figure 1. Figure 2 shows a schematic representation of Watershed 1 in greater detail. For the purpose of modeling, the watershed consists of seven subareas; a clean watershed upstream of the highway which channels runoff under the roadway to the outlet; five sub-watersheds representing the pavement and another sub-watershed downslope of the roadway. Except for the first sub-watershed, the other six watersheds carry dirty water. The roadway sub-catchments and the dirty sub-watershed drain into two sedimentation ponds. The pond outlets then discharge into a channel to join the clean water from the upstream sub-watershed and the accumulated flows are transported to the watershed outlet.

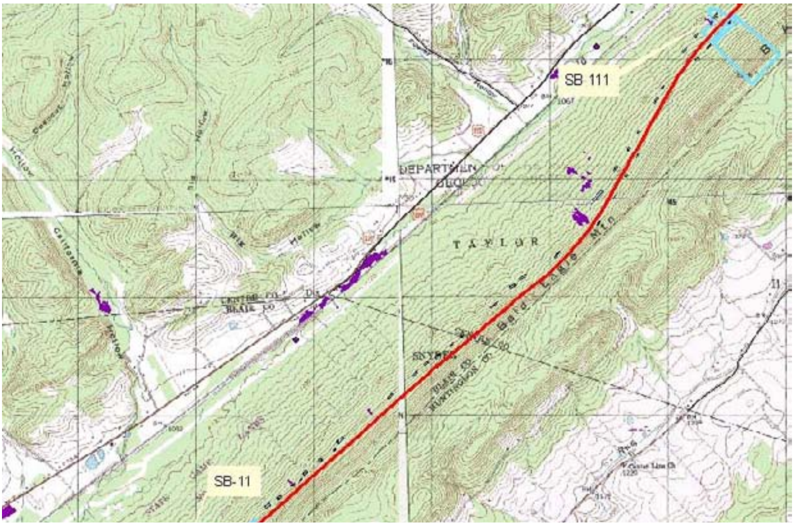

Figure 1: Location of construction relative to receiving stream.

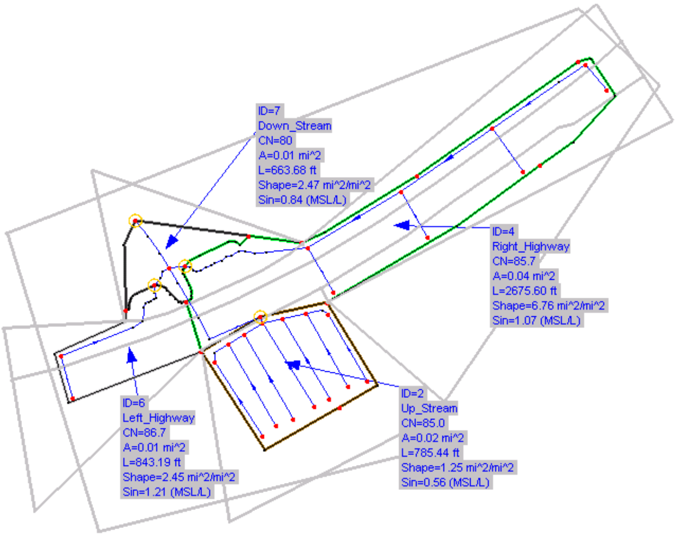

Figure 2: Schematic of watershed model.

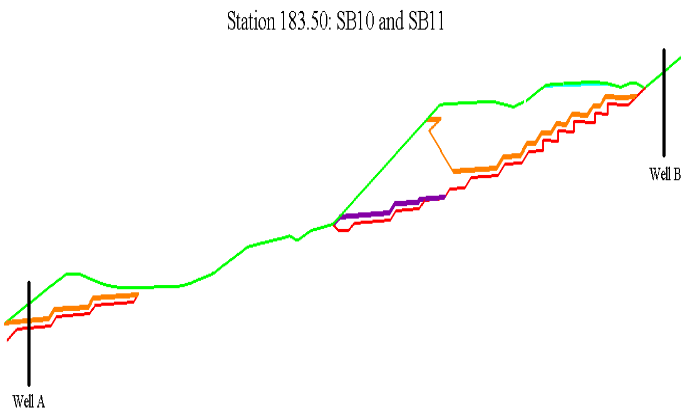

Figure 3: Location of monitoring wells upstream and downstream of highway.

Hydrologic elements are used to represent each of the sub-watersheds, the channels and the ponds. Runoff contributions are routed through these hydrologic components until they reach the watershed outlet where a Venturi flume is installed to continuously monitor the outflow. The characteristics of these elements and routing characteristics were determined from the construction drawings. Measurements were made to determine the water level changes at the storage ponds located within the test watersheds. This required the installation of water stage recorders to track the time variation of pond water levels. A Parshall

Venturi flume was installed at the outlet of each watershed to measure runoff. Each of the flumes is provided with an automatic water level recorder which measures the depth of water at the throat of the Venturi flume. For tracking groundwater, deep monitoring wells were installed upstream and downstream of the roadway in each of the test watersheds in order to measure the fluctuations in groundwater levels. This is shown in Figure 3. Rainfall data were obtained from the precipitation gage nearby. Runoff data are transcribed from downloaded readout from water level recorders. Water levels are converted to discharge using the discharge rating curve provided by the manufacturer.

4 Analysis

The hydrograph prediction algorithm takes the precipitation input record, and runs it through the model in order to calculate the resulting runoff. The model takes into account abstractions that may be attributed to losses due to infiltration, soil moisture accretion, depression storage, basin recharge and evapotranspiration. For short-term events evapotranspiration losses are negligible. After correcting for base flow, the direct runoff is first checked if it equals the estimated rainfall excess to meet mass-balance requirements. The numerical criteria for model acceptability are that the calculated peak flow and the time to peak must not differ from the measured values by a significant amount. As a rule, a difference of 15% or less was deemed acceptable.

To explain the process, this report will discuss the data analysis and calibration procedure for the test watershed 1, identified in Figure 1. Because the watershed area is small (46.1 acres), it was assumed that the storm deposited a uniform depth on the whole watershed over its duration. As shown in Figure 2, the watershed was subdivided into seven sub-watersheds so that runoff contributions can be more accurately delineated. For each of the seven sub-watersheds, the rainfall excess was calculated using the Soil Conservation Services method of relating the water retention characteristics to the nature of the surface, soil type and land use. For impervious surfaces such as the highway pavement, curve numbers in the range of 80–98 would be appropriate since most of the precipitation on impervious surfaces will be converted to runoff. Subtracting the abstraction from the precipitation resulted in excess rainfall amounts at each sub-watershed. These are then processed by routing them through the model components until the final outlet hydrograph is obtained. Processing included routing the runoff from dirty areas to two storm detention basins which also serve as sedimentation ponds.

In addition to a visual comparison, the numerical criteria for goodness-of-fit consisted of comparing the times to peak and the peak discharge as calculated by the model with the measured quantities. A difference of 15% was adopted as acceptable. After the model was calibrated, it was applied to several more storms.

Based on the results of the tests, we determined that the computer code for the hydrologic model that was developed to predict runoff has performed well.

Hydrograph predictions for the significant storms that occurred during the testing period compared reasonably well with actual measurements.

5 Effectiveness of the infiltration gallery

The infiltration gallery is a new feature in the design. In this scheme, “dirty water” infiltrating from disturbed area would pass through an infiltration zone and be laterally transported downstream without mixing with “clean groundwater” coming from undisturbed areas upstream of the highway. The schematic of this is shown in Figure 4.

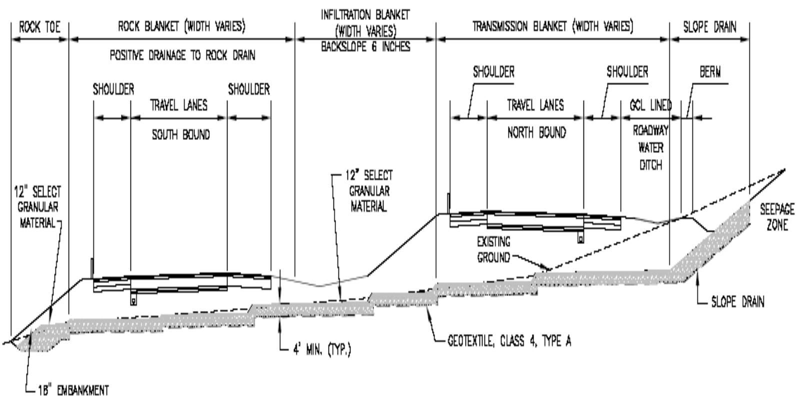

Figure 4: Schematic of infiltration gallery.

The infiltration gallery was intended to catch infiltrated dirty water (water from the disturbed portion of the watershed) and transport it laterally thus preventing it from percolating into the deep groundwater zone. The infiltration gallery, being constructed of fill material, represents a stratum of lower hydraulic conductivity than the natural subsurface zone. According to groundwater theory, the horizontal permeability of layered media is always greater than its vertical conductivity. Hence, in addition to the filtering effect, the gallery will indeed transport infiltrated dirty water laterally downstream where it will be captured in the storage pond. It is possible that some of this dirty water could percolate deeper and mix with the clean groundwater from above the highway. One method to examine the effectiveness of the infiltration gallery is to compare the quality of water in the pond with that of the water in the deep well downstream.

This comparison was carried out by sampling water from the ponds and the deep monitoring well nearby. The samples were analyzed to determine the concentrations of several minerals and pollutants at both locations. The water quality in the deep well which would represent deep groundwater flow is significantly better than the water quality of water in the pond which represents filtered dirty water. It was determined that the pH in the deep well is not significantly different from in the pond. As expected the conductivity in the pond

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was found to be almost twice as that in the well. Concentrations of iron and manganese between the ponds and the well are comparable. Those of magnesium and calcium are significantly higher in the ponds than in the well.

Table 1: Comparison of water quality in ponds and well.

	pH	Cond. (mg/l)	Mg (mg/l)	Ca (mg/l)	Mn (mg/l)	Iron (mg/l)
Pond 10	6.7	503	24	69	0.06	0.03
Pond 11	6.0	715	29	110	0.08	0.03
Well B	6.0	314	15	47	0.07	0.03

In order to carry out the verification of effectiveness, a monitoring program which documents the above practices as construction proceeds was set up. This includes periodic visits to the sites of BMPs and recording changes in erosion prevention structures due to construction traffic, vegetation growth and extraordinary runoff episodes, documenting them with photographs for later analysis to determine their effectiveness recording effectiveness.

6 Instrumentation

Modeling support for impact assessment requires not only a site-specific formulation of the hydrologic interactions. It also requires instrumenting the watershed to verify the model performance. At a minimum, one must provide for the installation and maintenance of equipment used for observing variables at various points of the watershed. For the test watersheds, deep monitoring wells and shallow groundwater stage recorders are required. Storage ponds must be equipped with stage recorders to track storage fluctuations. Flow recorders at the watershed outlet and precipitation and other hydro-meteorological variable must be tracked.

All measurement gages must be capable of continuously recording and storing data. This enables downloading information at regular intervals and reduces the frequency of field visits. All the instruments used in this study were equipped with automatic recorders. Data downloads were made approximately every month.

The flow metering device needed at the watershed outlet depends on the size of the watershed. The device should cover the range of discharges that can be expected. For small watersheds, as in the present case, a Parshall flume was selected. This must be provided with a metering device that can continuously record the discharge to capture the hydrograph from a storm.

For monitoring the deep groundwater zone, at least two wells must be installed – one to capture the groundwater levels above and another below the impacted area. Shallow groundwater may be tracked using commercially available water stage recorders. Water level fluctuation in storage ponds must be tracked to carry out hydrograph routing.

It is critical that precipitation data for calibrating and testing the model be accurate and reliable. The possibility of record interruption due of a power cut off should also be anticipated. A back-up power system is recommended.

For impact analysis, it is required to compare conditions before and after construction. This implies that monitoring must commence before actual construction begins. Monitoring instruments must therefore be installed as soon as the highway alignment has been determined and test watersheds can be delineated. Finally, periodic checks must be made to determine if instruments are functioning as intended

7 Conclusions

In order to complete an Environmental Impact Analysis for a construction project, one must have a hydrologic model to obtain numerical measures of water quantity and quality. The development of a model to predict runoff is therefore an indispensable requirement. In formulating the model one must take into account the altered watershed delineation and instrument the watershed to capture changes in the environment engendered by the construction project. In addition to the mathematical development, the field verification of the hydrological model requires installation of data acquisition hardware supported by software that permits continuous monitoring of the variables. These must be installed as early as possible in order to capture changes in hydrologic quantities as a result of the project.

Acknowledgements

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References

[1] Bruntland, G. (ed.), *Our common future: The World Commission on Environment and Development*, Oxford, Oxford University Press, 1987.
[2] Carson, R., *Silent Spring*, Houghton Mifflin, New York, 1962
[3] Meadows, D. et al., *Limits to Growth*, Universe Books, New York, 1972.

Hydrological analysis of the water flow through the Excellence Development Zone (Magdalena, Colombia)

N. Molinares, J. Manga, A. Sisa & J. Arrieta
Water Technologies Research Group. Universidad del Norte, Colombia

Abstract

The Excellence Development Zone is located on the eastern shore of the Magdalena River, and belonged to the Natural Reserve of the Salamanca Park in the Santa Marta's Big Marsh Ramsar site. This zone was split in order to allow the socioeconomic development in Palermo (Sitionuevo, Magdalena). Due to the lack of certain knowledge of the flow behaviour in this zone, and to make an appropriate distribution of the land for different kind of activities, a hydrological study of the EDZ was carried out. According to the data collected, it was determined that the water enters the zone mainly through the Clarin Nuevo drain, across the south portion of the EDZ near to Palermo, meeting the water coming from the sea and the Santa Marta's Big marsh, and going back underground parallel to the Clarin Viejo drain to the Cantagallo marsh. Following that trends and after making a global environmental analysis, it was recommended to improve the structure of the Clarin Nuevo drain in order to settle most of the particulate material in the water from the Magdalena, avoiding public health and environmental problems.
Keywords: Excellence Development Zone, sustainable development, water resource management.

1 Introduction

Sustainable development has turned into a key topic for socioeconomic growth in all countries around the world. With an appropriate harnessing of natural resources, especially in developing countries which have a significant environmental offer, equilibrium between economic activities, society expansion, and ecosystem protection can be reached.

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One of the most important issues to be carefully analyzed when the sustainability of certain development strategy is discussed is how extent the availability or the maintenance of the natural resources would be committed with the new activities and land uses. The water resources can be quickly damaged by a wide spectrum of economical activities if it is not well know the flow direction through the zone. It may happen, e.g. that groundwater gets contaminated causing, therefore, public health problems and the long-term deterioration of ecosystems.

In this research, a hydrological analysis in the Excellence Development Zone was made, to determine the convenience of permit industrial and commercial activities into it. This analysis was taken as a criterion in a Global Environmental Analysis carried out by the research group and local environmental authorities.

2 Results and discussions

2.1 The Excellence Development Zone (EDZ)

The Excellence Development Zone (EDZ) is a portion of land located on the east shire of the Magdalena River, from the Clarin Viejo drain to the mouth of the river. The zone belonged to the Natural Reserve of Salamanca Park in the Santa Marta's Big Marsh Ramsar site. This zone was split from the reserve in order to allow the socioeconomic development of citizens in Palermo (Sitionuevo, Magdalena), and also impel industrial and commercial activities on this shore of the river.

Figure 1: Location of the EDZ.

The EDZ has an approximate length of 22 km and includes the split zone with 100 m into the river and 500 m into the park. According to the available cartography, the EDZ has a width of 1350 m in its southern side, near to Palermo. This widths decrease to 900 m between the Clarin Viejo and Torno drains, then increase again up to 1200 m until the Limon drain, and after it, maintains its width in 600 m. See Figure 1.

There are not enough previous studies about this zone. The most relevant one was made by the Colombian Environment, Housing, and Land Development Ministry (MAVDT) which contains the criteria and rules for sustainability evaluation and land ordering in the Ramsar site. However, it is still a lack of basic information.

From another point of view, antropic influence could affect the natural reserve and the Ramsar site transporting pollutants depending on the real direction of the water flow through this zone. Moreover, the environmental problems of the Magdalena River, related to organic matter, nutrients, pathogens and sediments could cause health disturbances and disease on people living there.

Based upon there was no a certain knowledge of the flow behaviour, and to make an appropriate distribution of the land for different kind of activities, it was carried out the hydrological study of the EDZ.

2.2 Hydrological study

The Santa Marta's Big Marsh lagoon complex is conformed by more than 20 lagoons with different settling and salinity characteristics, connected by drains and channels, fed mainly by the Magdalena River, the Santa Marta's Sierra Nevada, and the Caribbean Sea. In this complex, the whole system of drains and channels has variable flow-rate magnitudes and directions (Rivera and Caicedo [1]).

As a part of this complex, the hydrological study for the EDZ has to take into account the behaviour of its four (4) main water providers: the Caribbean Sea, The Magdalena River, groundwater flows, and superficial flows. The topographical profile and vegetation distribution were also considered.

2.2.1 Topography

Topographical information plays an important role in the determination of the superficial flows direction because the land slope fixes the flow gradient. The estimation of flow gradients, therefore, was based upon topographical information due there was no previous measurements of flow-rates in drains and channels of the EDZ. The profile is shown in Figure 2.

According to the data, land has a slight slope from the Ramsar site to the river, but on the shore a small terrace was found. It was also noted that the elevation tend to get lower from south to north.

This information implies that gradients try to impel superficial flows towards to the river mouth, passing through a big portion of the EDZ.

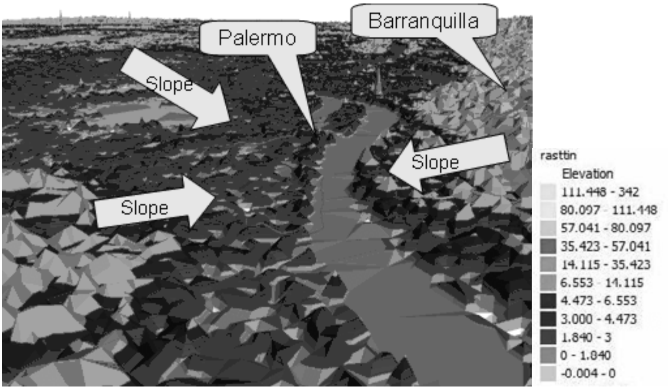

Figure 2: Digital Elevation Model for the EDZ (vertical scale factor = 30).

2.2.2 Caribbean Sea

The Caribbean Sea has tide waves with a mean amplitude of 0.35 m and maximum amplitude of 0.60 m, decreasing to the east side due to the presence of the Sierra Nevada.

The main connection between the Caribbean Sea and the lagoon complex is made through the coastal bar gap in a continuous way, and by soil infiltrations of seawater at the coast.

The combined action of northeast winds and tidal forces modifies the levels of water bodies, and, thus, the water exchange capability in the channels connecting them.

2.2.3 Magdalena River

The Magdalena River is the most important water source for the Natural Reserve of Salamanca Park, through the river shore and the Clarin Nuevo drain. In order to analyze its behaviour, the levels of the river at Calamar station (90 km up from Palermo) were taken. The observed behaviour is seasonal, reaching the highest levels between June and December, and the lowest between February and April. The mean variability of the 50% leave curve shows that levels range 2.8 and 7.3 m. along the year.

With levels, the flow-rates at Calamar were estimated. The best fit was obtained with the exponential regression, eqn (1),

$$Q_C = 1586.5 \cdot e^{0.2865 \cdot L} \qquad (1)$$

where Q_C = river flow-rate at Calamar ($m^3 \bullet s^{-1}$), and L = river leave level (m). According eqn (1), the mean annual river flow-rate at Calamar is 7742 $m^3 \bullet s^{-1}$. However, this flow-rate cannot be used directly in the analysis because the station is located before the Dike Channel derivation.

Deeb Sossa S en C [2] established a simple relationship to obtain the flow-rates downstream the Dike Channel, eqn. 2,

$$Q = 105 + 0.9105 \cdot Q_C \qquad (2)$$

where Q = river flow-rate downstream the Calamar station (m^3•s^{-1}). By applying the eqn (2), the mean annual river flow-rate at EDZ is 7154 m^3•s^{-1}.

Another relevant issue to be reviewed is Magdalena River water quality. The data analyzed from sampling showed that the principal source of contamination in the EDZ is domestic wastewaters and the transport of some particulate material and NPK fertilizer residues. The highest values identified corresponded to nutrients (N and P), and coliforms. Nutrients can cause eutrophication, and coliforms generate disease on human settlings. Whichever the case, the water quality of river could threaten the public health and the natural reserve if the inflow coming from gets increased.

2.2.4 Groundwater flows.

Based upon the soil characteristics, and taking into account that most of the water exchange in the lagoon complex is made by means of infiltration; groundwater flows could have an important effect on the hydrodynamic stability due possible alterations on phreatic levels which also would change the soil salinity.

In order to estimate the current groundwater conditions, 50 holes were made in EDZ, and in 14 of these phreatic level was measured. Figure 3. The aim of the perforations was to generate a gradient plan that indicates the flow direction near the Cantagallo Marsh.

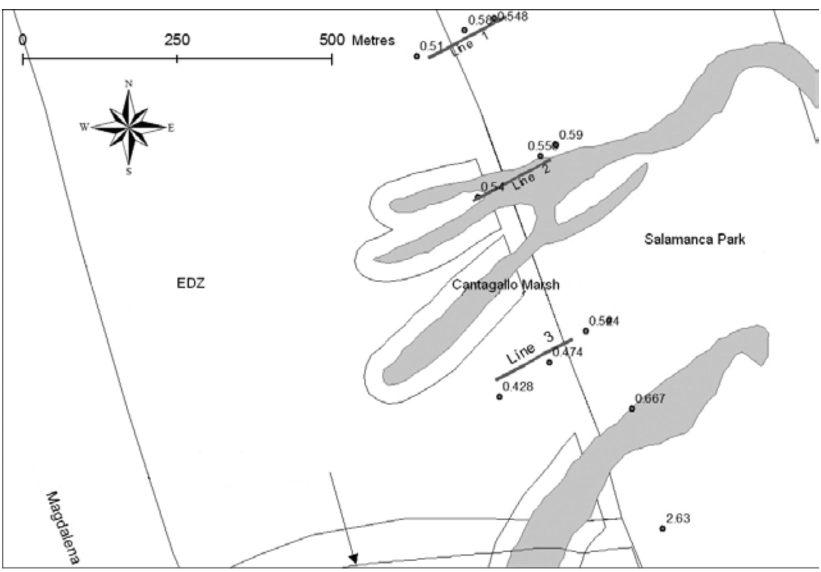

Figure 3: Phreatic level measurement stations for the EDZ.

From the data collected, phreatic surface was calculated, and three sections located at lines 1, 2, and 3 (Figure 3) are shown in Figure 4.

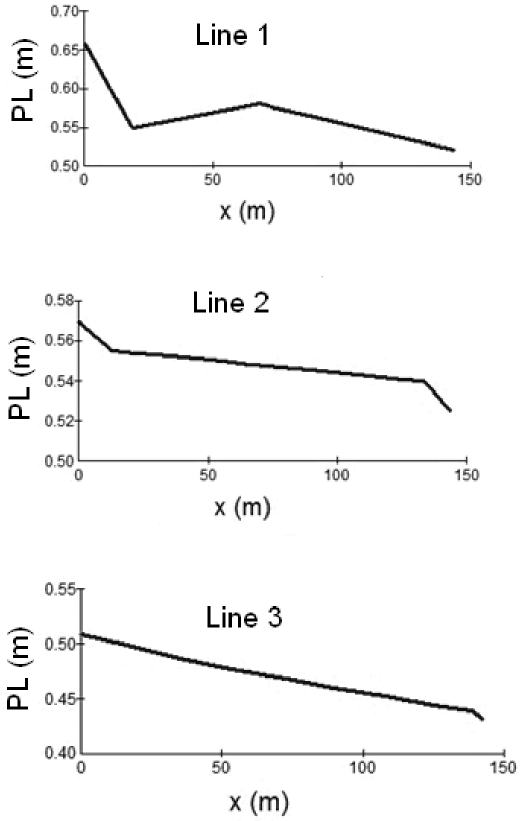

Figure 4: Measured phreatic level (PL) near the Cantagallo Marsh at three different locations. Abscises are measured along the line from east to west.

It can be noted in Figure 4 that the phreatic level descends as the distance from the river is decreased. This fact confirms the assumptions made on the basis of the topographical data.

2.2.5 Superficial flows

Concerning superficial flows, there was identified two different trends were recognized. At the north of the EDZ, the water enters from the Magdalena River through El Torno and Limon drains; whereas the south portion of EDZ has a little more complex flow structure.

At the south of the Clarin Viejo drain, the water enters to the Salamanca Park from the Magdalena River passing through the Clarin Nuevo drain. Then water goes from the reserve to the EDZ by infiltration and some little superficial drains and creeks. No flow from the river was identified due to the terrace on the shore.

This flow configuration limits a close zone between the Clarin Nuevo and Clarin Viejo drains. Nevertheless, erosion processes could create a new entrance to this portion of the EDZ if the shore is not reinforced. The flow direction is shown in Figure 5.

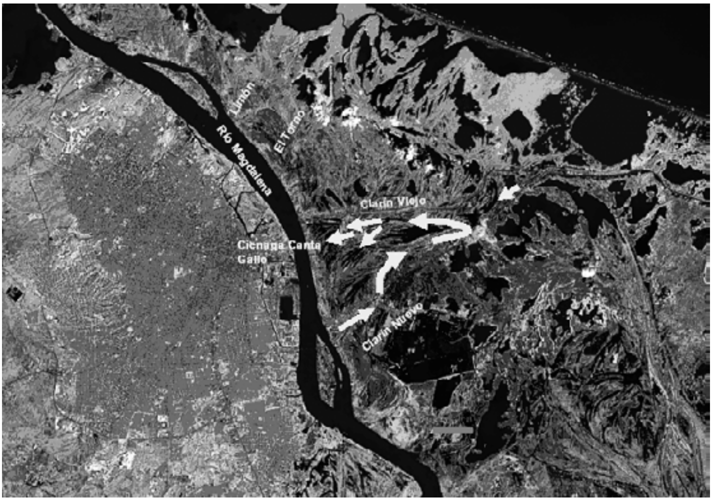

Figure 5: Flow trend through the southern EDZ portion.

3 Conclusions

According to the hydrological analysis made for the EDZ, the development of economical activities in the EDZ does not represent a risk for the Natural Reserve of Salamanca Park. The water was found to flow mainly from the reserve to the river slightly deviated to north. Water enters to the reserve through the Clarin Nuevo drain at the south, and through the Torno and Limon drains at the north.

It is recommended to improve the structure of the Clarin Nuevo drain in order to settle most of the particulate material in the water from the Magdalena River, avoiding, therefore, health and environmental problems.

References

[1] Rivera, M. and Caicedo, D. *Informative Chart of Ramsar Wetlands – Estuarine Magdalena River's Delta/Santa Marta's Big Marsh System.* Colombian Environment Ministry. Colombia, 1998.

[2] Deeb Sossa S en C. *Recovery Plan for the Santa Marta's Big Marsh Lagoon Complex, Hydraulic design.* Final Inform. DNP, CORPAMAG. Colombia, 1993.

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Decomposition of organic materials and its effect on transport of nitrogen, phosphorous and water in unsaturated soil

I. S. Al-Salamah[1], I. N. Nassar[2], A. I. Turki[3] & F. M. Alromian[3]
[1]*Department of Civil Engineering, College of Engineering, Qassim University, Saudi Arabia*
[2]*Faculty of Agriculture-Damanhoar, Alexandria University, Egypt*
[3]*Faculty of Agriculture and Veterinary, Qassim University, Saudi Arabia*

Abstract

A simultaneous transport of water, phosphorous, and nitrogen and organic matter decomposition were studied experimentally and numerically. Greenhouse experiments were achieved for collecting data of organic carbon, nitrogen, phosphorous and water contents. A numerical model (LEACHM) was used for simulating these data. Simulated results were compared with experimental data. The experiments were conducted in one-end open PVC columns (0.056-m ID and 0.30-m high) using unsaturated sandy soil under the atmospheric condition of Buridah, Al-Qassim, KSA. Municipal solid wastes (fermented partially) were mixed with the sandy soil. A 0.05-m mixed soil layer was located at six different depths (6 treatments) within the soil columns. Two replicates were assigned for each treatment. The soil or the mixed layer had initial water content of 0.0802 m^3/m^3. The initial carbon in the soil and mixed layer were 0.117 and 0.451%, respectively. The experiment lasted two months with application distilled water periodically at the open end of soil columns. Both predicted and measured final water showed nonlinear distributions. The model overestimated slightly the water content in comparison to the observed data. The stored water in a soil column decreased as the depth of mixed layer increased. The predicted and observed concentrations of organic carbon, NH_4^+, NO_3^-, and available phosphorous behaved similarly. The beak concentrations of the aforementioned variables occurred at the depth of the mixed layer. The organic matter decomposition was limited because the soil water content was low and the duration of experiment was short. The results of the study could encourage using the LEACH as a tool for organic matter management strategy and monitoring the fate and transport of plant nutrient as N and P in soil.
Keywords: LEACHM, organic carbon, decomposition, water movement, modeling, nitrogen, phosphorous.

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1 Introduction

Soil organic matter improves root growth, uptake of minerals and aids the plant in the physiological activities. It increases mobilization of nutrients both major and minor from the soil to plant roots. It also produces growth promoting substances and higher nitrogen fixation by bacteria. It is essential in the formation of soil aggregates and soil structure which has a direct bearing on soil aeration. Acids released, dissolve insoluble phosphates and make it available. Decomposition of organic matter is variously referred to as oxidation, metabolism, degradation and mineralization. Organic matter is first oxidized by molecular oxygen, and the products (or metabolites) of the reaction are carbon dioxide and recycled nutrients Froelich et al. [1] and Drever [2]. For example, the aerobic respiration of organic matter yields 106 moles of carbon dioxide (CO_2), 16 moles of ammonia (NH_3) and one mole of orthophosphate (PO_4) (reaction 1). If sufficient dissolved oxygen is present, bacteria oxidize the NH_4 produced in reaction 1 to nitrite (NO_2) and then to nitrate (NO_3) in a process called nitrification. The decomposition of organic matter may take several months to several years to complete. The study of soil organic matter decomposition is important for assessing the problems of organic matter depletion in soil, long term soil fertility and sustained productivity of soil. A variety of soil parameters based models have been used by several researchers to represent short and long-term changes in single state variable model form to represent the decline of organic matter in cultivated soils. In the original CENTRUY model Parton et al. [3], the soil organic matter (SOM) consists of three fractions. They constitute (i) an active fraction (active SOM) of soil C consisting of live microbes and microbial products, along with soil organic matter (SOM) with a short turnover time (1-5 years); (ii) a pool of C (slow SOM) that is physically protected and/or in chemical forms with more biological resistance to decomposition, with an intermediate turnover time (20-40 years), and (iii) a fraction that is protected with the longest turnover time (200-1500 years). According to CENTRUY model, the rate of decomposition, k, of soil organic matter depends on soil types, soil texture (silt + clay content), rainfall, soil temperature, potential evapotranspiration, initial soil organic carbon content and the maximum decomposition rate parameter. The k value for active pools ranges from 0.29 yr^{-1} van Veen and Paul [4] to 0.66 yr^{-1} Parton et al. [5], Jenkinson [6]. Intermediate pools k values range from 0.02 yr^{-1} Jenkinson [6] to 0.04 yr^{-1} Parton et al. [5] to 0.003 yr^{-1} van Veen and Paul [4].

In the LEACHM model Hutson and Wagenet [7], it is assumed three organic pools: plant residue, manure and soil humus, each of which is defined in terms of C:N:P ratios. Inorganic N pools include urea, nitrate and ammonium. There are two inorganic P pools: a labile pool which is always in local equilibrium with sorbed and solution phases, and a bound pool which is in kinetically-controlled equilibrium with solution P. P sorption is described by Freundlich isotherms. Mineralization rate constants refer to transformation of organic carbon, which is split into three transformation pathways: into soil humus, into biomass, which remains an integral part of organic pool, and into carbon dioxide. The relative sizes of these pathways are defined by the synthesis efficiency factor and the humification fraction. The efficiency factor is the fraction of C mineralized that is converted to humus and biomass rather than CO_2, while the humification factor defines the relative amounts of humus and biomass produced. LEACHM uses adaptations of the concepts and equations described by Johnsson et al. [8]. C:N ration for biomass and humus determine N released or consumed during mineralization. Thus C:N ratios of residue and manure decrease during mineralization; if insufficient N is available to satisfy the C:N ratio of the products, the rate slows. Inorganic N is subject to nitrification and denitrification. Denitrification rates increase with increasing water content and insufficient organic C needs to be present to support denitrification. Although organic pools are regarded as insoluble in LEACHM, this only means that they are not subject to transport. Conceptually, organic C can provide a C substrate for denitrification. Amonium volatilization is a complex, pH-dependent process. No attempt is made to simulate the various processes which contribute to and influence NH_3 volatilization. For simplicity, NH_3 volatilization is described assuming first-order kinetics, and is a function of a rate constant (which is not adjust for temperature or water content) and the concentration of NH_4 in the upper soil segment. LEACHM simulates flows between these pools in each soil segment as well as on the soil surface. C and N cycling is based on the procedures described by Johnsson et al. [8] but with additional pools and pathways. The inorganic P model was based on concepts described by Shaviv and Shachar [9]. In Shaviv and Shachar's model the strongly bound P pool was considered to be a precipitated form of P having a very low solubility; in the LEACHM this pool can be considered to be a precipitate or to follow a sorption isotherm. The labile pool is always in local equilibrium, but sorption to or desorption from the bound pool is kinetic, where the bound pool as well as the labile pool are described by Freundlich isotherms.

The objective of this study was to develop an integrated methodology for execution of LEACHM model developed by Hutson and Wagenet [7] for assessment soil organic matter decomposition and release of nitrogen and phosphorous nutrients. In addition, modeling water, N, and P movement in unsaturated soil is considered. In this model, it is considered only "active SOM" fraction of soil organic matter with a short time. In soil system, a continuing loss of soil humus carbon (decomposition loss) taken place by microbial oxidization.

2 Materials and methods

2.1 Greenhouse experiments

An experiment was achieved in Al-Qassim region, Saudi Arabia. Al-Qassim lies between 40 to 45 E longitude and 25 N to 30 N latitude approximately. The climate of the region is dry. The average rainfall of the area ranges from 100 to 200 millimeters. The ambient air temperature (T_{air}) of Saudi Arabia varies greatly from season to season and from region to region. T_{air} in the summer is about 43-48°C during the daytime and 32-36°C during the night time. The microclimate is such that soil water evaporation readily occurs. Soil materials were sampled from a surface layer (0.0-0.3 m depth) from the Agriculture and Veterinary Collage farm, Al-Qassim University. The soil materials were obtained by excavation, then air-dried and ground to pass a 2-mm sieve. The soil was composed of sand materials. A soil material batch was wetted with distilled water to obtain an initial volumetric water content of 0.0802 m^3/m^3. Another batch of soil was mixed with municipal solid waste (partially fermented) at 1% then wetted with an initial water content of 0.0802 m^3/m^3. Each soil batch was covered and stored at 20°C for 4 days. PVC cylinders (0.056-m ID and 0.3-m high) were closed at the bottom ends using epoxy-sealed PVC lids. Six treatments were obtained based upon the layer depth of mixed soil layer within the soil column. The treatments are shown in Table 1. The thickness of mixed layer was 0.05 m. Two soil columns replicates were packed at bulk densities of approximately 1514 kg m^{-3} for each treatment. The soil columns were buried vertically within a bare soil field with exposing the open upper end to the natural atmosphere of Al-Qassim region. Distilled water was poured at the open of soil column at different time as shown in Table 2. The soil temperatures at both ends of soil column were recorded. The soil columns were sectioned into a 0.05-m increment. The soil of each increment was divided into two parts: a part for water content and organic carbon and a part for ammonium, nitrate, and phosphorous determinations. The soil water contents were determined gravimetrically in the increments. Available P was extracted using the sodium bicarbonate (Olsen et al. [10]) then was determined by the chlorostannous phosphomolybdic acid method (Jackson [11]). The organic matter was estimated using the Walkely and Black method (Nelson and Sommers [12]). The nitrogen in the soil samples was determined by distillation (Page et al. [13]).

2.2 Theoretical analysis

LEACHM model devolved by Hutson and Wagenet [7] was used to describe water and chemicals transfer, and organic matter decomposition in a soil under nonisothermal condition. A brief outline of LEACHM is presented below for completeness, with additional details found elsewhere (Hutson and Wagenet [7]).

2.2.1 Water flow

The soil water flow equation for transient vertical flow derived from Darcy's law and the continuity equation, is:

Table 1: The depth of municipal waste with the soil column.

Treatment number	Depth of the mixed layer, m
1	0.0 0.05
2	0.05-0.10
3	0.10-0.15
4	0.15-0.20
5	0.20-0.25
6	0.25-0.30

Table 2: Amount and time of water application (m/column).

Date of application	Amount of applied water, m	Date of application	Amount of applied water, m
26/7/2003	0.01016	2/9	0.00203
29/7	0.02032	7/9	0.00203
6/8	0.01016	10/9	0.00203
10/8	0.01016	13/9	0.00203
26/8	0.00203	15/9	0.00203
30/8	0.00203	20/9	0.00203

$$\frac{\partial \theta}{\partial t} = \frac{\partial \psi}{\partial t} C_w = -\frac{\partial q}{\partial z} - U(z,t) \tag{1}$$

where θ is water content (L^3 L^{-3}), q is water flux density (L/T), defined as q=-K(Ψ)(δH/δz), H is hydraulic head (L), defined as H=Ψ-z, Ψ is soil water matric potential and z is depth), K is hydraulic conductivity (L/T), t is time (T), C_w is differential water capacity (L^{-1}), and U is a sink term representing water lost per unit time by transpiration (T^{-1}). The later term is neglected under the present study because there is no plant. The water transfer properties (Ψ -θ, and hydraulic conductivity, K) were described in detail by Campbell [14].

2.2.2 Decomposition of organic C, N and P

The rate of loss of soil organic matter (SOM) is based on the instantaneous decay constant, k, which expresses the proportion of the pool that turns over per unit time. We simulated soil organic decompostion for two months using the LEACHM model. The simulated results for total soil C were compared to the measured values from the greenhous experiment. First-order mineralization rate constants determine the overall decomposition rate of the humus pools.
Decomposition follows first-order kinetics:

$$\frac{dC_i}{dt} = -\mu_{mi} C_i \tag{2}$$

where C_i represents the concentration of humus, μ_{mi} is first-order rate constants, and t is the time.

Organic N and P associated with the decomposing C pool is released. C:N and C:P ratios defined for humus how much of the N and P released by mineralization are assigned to the humus pools. Dentrification process is neglected in the present work because the soil was unsaturated.

2.2.3 Flow of nitrogen and phosphorus

The movement of miscible solute (i.e., N and P elements) through a soil is assumed in LEACHM to be accomplished by chemical diffusion in the liquid phase in response to an aqueous concentration gradient and convection of the solute as the result of movement of water flow in which the solute is dissolved. That is:

$$J_t = J_d + J_c \tag{3}$$

where J_t is total solute flux (M/L^2 T), J_d and J_c are diffusion and convection fluxes in the liquid phase, respectively.

The diffusion flux in a soil solution can be obtained from:

$$J_d = -D_p(\theta)\frac{\partial C_l}{\partial z} \tag{4}$$

where $D_p(\theta) = D_o\, \varepsilon \exp(b^*\theta)$, is the diffusion coefficient of solute in porous media and ε and b are empirical constants Olsen and Kemper [10]. Values of ε ranged from 0.05 to 0.01 and b=10. C_l is the concentration in the liquid phase (M/L^2). D_o is the diffusion coefficient of solute in a free-water system (L^2/T). The diffusion coefficient was described in details Nassar and Horton [15] and Lyman et al. [16].

The convective flux of a solute is usually represented as:

$$J_c = -\theta D_m(q)\frac{\partial C_l}{\partial z} + qC_l \tag{5}$$

where $D_m(q) = \lambda|v|$, is the mechanical dispersion coefficient that describes mixing between large and small pores, $v=q/\theta$, is the average pore velocity and λ is the dispersivity (L) and its value ranged between 0.5 Δz to $2\Delta z$, and Δz is node spacing (L).

The total steady state solute flux in the liquid phase in a porous media, J_t can be described as:

$$J_t = -\theta D_m(q)\left(\frac{\partial C_l}{\partial z}\right) - D_p(\theta)\frac{\partial C_l}{\partial z} + qC_l \tag{6}$$

The nonsteady-state equation for the solute transport can be written as:

$$\frac{\partial C_t}{\partial t} = -\frac{\partial J_t}{\partial z} \pm \Phi \tag{7}$$

where $C_t = \theta C_l + \rho_b C_s$, is the total solute concentration in liquid and sorbed phases (M/L^3), ρ_b is the bulk density of soil (M/L^3), C_s is the concentration of solute in the sorbed phase (M/M) (i,e. phosphorous or ammonium) and Φ is sources or sinks of solute (M/L^3 T). Φ is considered negligible for N compounds for the present study because we did not add mineral fertilizer.

LEACHM model can reflect sorption using curvilinear Freundlich isotherms. This is required in order to describe P sorption realistically. The Freundlich isotherm is defined as

$$C_s = k_f C_L^{n_f} \tag{8}$$

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C_s is sorbed concentration (mg kg-1), C_L (mg L^{-1}) is solution concentration, and k_f ($mg^{1-nf} kg^{-1} L^{nf}$) and n_f are constants.

In the case of ammonium, the sorbed ammonium concentration can be described as

$$C_s = K_d C_l, \tag{9}$$

K_d is a partition coefficient (L/M).

2.3 Initial and boundary condition

The initial conditions associated with Eqs. (1) nd (7) are given by

$$\theta(z,0)=\theta_i,\ C_t(z,0)=C_i \tag{10}$$

The upper boundary conditions for water and solute are given in terms of net mass fluxes by

$$q(0,t) = \zeta E \quad \text{(under the evaporation conduction)} \tag{11}$$

$$J_t(0,t) = 0.0 \quad \text{(for a non-volatile compound)} \tag{12}$$

or

$$q(0,t) = \inf \quad \text{(under ponding condition)} \tag{13}$$

$$J_t(0,t) = 0.0 \quad \text{(no added N or P compounds in the applied water)}$$

where ζ is the evaporation pan coefficient, E is the potential evaporation rate (LT^{-1}), inf (LT^{-1}) is the infiltration rate .

The lower boundary condition for water and solute (N and P solutes) are:

$$q\ (l,t) = 0.0 \tag{14}$$

$$j_t(l,t) = 0.0 \tag{15}$$

2.4 Model and results evaluation

Model ability to predict a variable should be evaluated. Visual comparison of simulated and observed data provide a quick and often comprehensive mean of assessing the accuracy of model prediction. However, quantitative evaluation of the model is recommended. In the present work, the mean error (ME) and root mean square error (RMSE) are used as criteria for evaluating the model Milly [17]. The ME values can be estimated as:

$$ME = \frac{1}{N}\sum_{i=1}^{N}(V_p - V_m) \tag{16}$$

and RMSE can be estimated as:

$$RMSE = \left(\frac{1}{N}\sum_{i=1}^{N}(V_p - V_m)^2\right)^{1/2} \qquad (17)$$

V_m is the observed value of a variable (θ, or C_l), V_p is the predicted value of a variable, and N is number of observations for the variable.

2.5 Soil parameters and characterization

The retention curve data (Ψ vs. θ) and the saturated hydraulic conductivity were measured for the sandy soil used in the present study. The relation of Ψ vs. θ was fitted to the Campbell's function [14]. The coefficients of the function, Ψ_e and b, are shown in Table 3. The saturated hydraulic conductivity, Ks, of the soil was measured in a laboratory soil column using a constant head method Klute and Dirksen [18]. The unsaturated hydraulic conductivity, $K(\Psi)$, was estimated from knowledge of K_s and the retention curve data Campbell [14]. For more details, the reader is referred to Al-Salamah [19].

Table 3: Input characterization data for LEACHM model.

Parameters	Values
ρ (bulk density) (kg m^{-3})	1514
θ_i (initial water content) (m^{-3} m^{-3})	0.0802
θ_s (saturated water content) (m^{-3} m^{-3})	0.4286
K_s (m s^{-1})	$2.25x10^{-5}$
Clay, (%)	1.9
Sand (%)	96.3
Silt (%)	1.8
$CaCO_3$ (%)	3.72
Ci (P), ppm	13.7 (17.0)*
Ci (NO_3), ppm	20.04 (37.22)*
Ci (NH_4), ppm	10.18 (16.54)*
Ci (O.C) (%)	0.117 (0.451)*
D_o (m^2 s^{-1})	$6.478x10^{-10}$
k_d (L/kg) (NH_4)	3 (NH_4)
μ_{mi} (day^{-1})	0.0025
k_f (L/kg)	100 (P)
n_f	0.6 (P)
C:N (humus)	10:1
C:P (humus)	50:1
Ψ_e (kPa)	-0.38
b	1.81

* These numbers represent the concentration in the municipal waste mixed layer through the soil column.

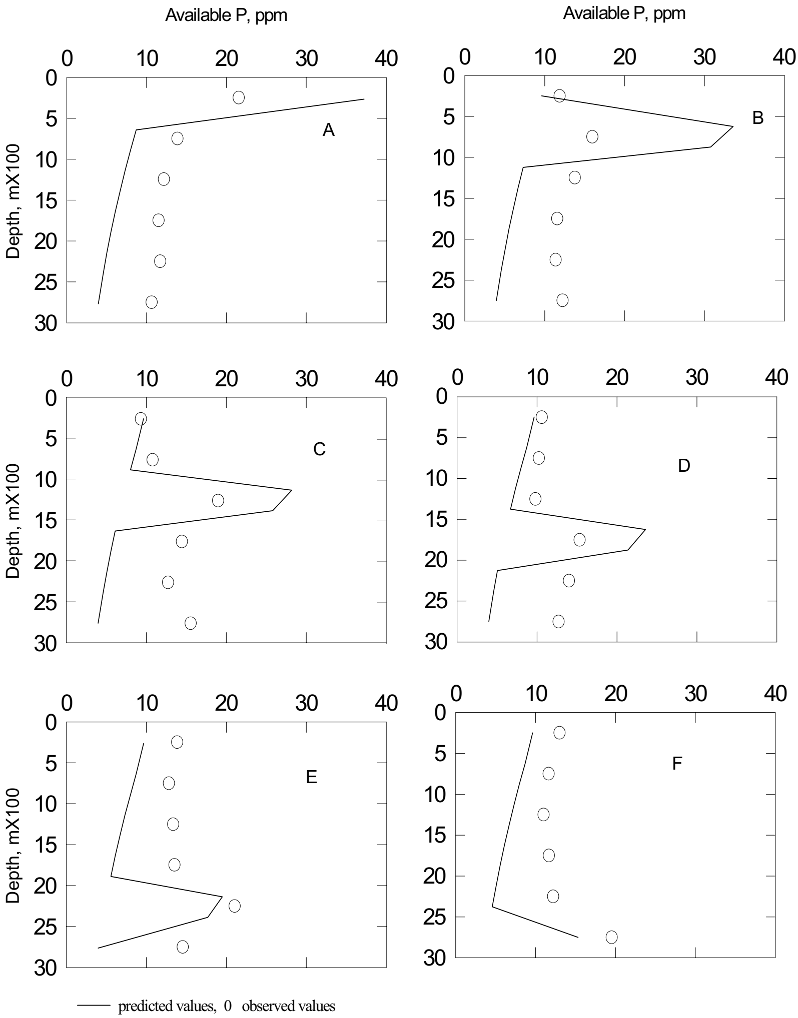

Figure 1: The predicted and observed available phosphorous. A, B, C, D, E, and F referred to the depths of municipal waste layer 0.0-0.05, 0.05-0.1, 0.1-0.15, 0.15-0.2, 0.2-0.25, and 0.25-0.3 m, respectively.

3 Results and discussion

3.1 Predicated and measured nutrients concentration

Measured and predicted available phosphorous, P, concentrations are compared in Figure 1 for time of 60 d. Initial soil carbon, nitrogen and phosphorus concentration are provided to the model as input variables (Table 3). The predicted values of P followed the trend of the observed values. Both observed and predicted values showed the greatest P concentration in the municipal waste layer. The great value of P is due to two reasons. The first reason is the high initial concentration of P in the municipal layer. The second reason is the low movement of P that is controlled by the presence of calcium carbonate (3.72%) and low soil water content. The average means of P within most of the soil column were lower than the initial P concentration (14.25 ppm) which indicated that some of labile P is bounded to the soil particle. The model overestimated the predicted P in the layer of municipal waste. This might be due to the high mineralization coefficient ($0.0025d^{1}$) used for the simulation followed by high released P. The soil posses' low percentage of water contents that results low unsaturated hydraulic conductivity. So, movement of phosphorous in soil is limited well. It is expected that some of the released phosphorus will be bound to the calcium carbonate present in soil. When the municipal waste layer was either at the 0-0.05 or 0.05-0.10 m-depth, the predicted P was greater than the observed. This discrepancy is due to the lower movement of water by model or the high mineralization rate of SOM. In general, the measured value of phosphorous showed little difference within the soil column. Phosphate forms are more soluble Ca^{2+} and Mg^{2+} at pH values near neutrality, and difficulty soluble Ca^{2+} compounds at higher pH Bohn et al. [20]. Phosphate availability also tends to decrease at high soil pH, because of precipitation as insoluble calcium phosphate compounds Bohn et al. [20]. The long-term capacity of most soils to adsorb phosphate is an order of magnitude greater than the amounts of phosphate added as fertilizer Bohn et al. [20].

Figures 2 and 3 show the predicted, and observed ammonium and nitrate concentrations, respectively. Both the predicted and observed values followed a similar trend. The municipal waste layer possessed the greatest concentration of either form of nitrogen. The predicted values of all forms of nitrogen showed good agreement with the observed ones within all soil columns of most of the treatments. When the municipal waste layer was located in the 0-0.05 layer, the LEACHM model overestimated all of the nitrogen forms in this layer. This discrepancy in the first layer in nitrogen concentration can due to the high convective transport calculated by the model for the nitrate and the low nitrogen loss in gaseous phases. The amount of N_2 and N_2O lost during oxidation is normally small and often negligible Bohn et al. [20]. The average mean concentration of ammonium within all soil column treatment was less than the initial concentration of ammonium (11.24 ppm), while nitrate concentration means were greater than the initial concentration (24.57 ppm). Some of ammonium might be converted to nitrate as a result of nitrification process. Relative to other exchangeable cations, however, the amount of exchangeable NH_4 is almost invariably small. Ammonium ions are much less mobile than NO_3 and are much less likely to be lost through denitrification, although ammonia readily volatilizes from the surface of alkaline soils. In the desert soil, for example, the absence of water greatly hinders the oxidation rate of organic materials at soil surface Bohn et al. [20]. Nitrogen may also be lost to the atmosphere as NH_3; which may occur whenever NH_4 is present at the soil surface, especially at high pH (above 7.0) and high temperature by volatilization when surface applied Barber [21]. The nitrate concentration was high when the municipal waste was located at either depth of 0.0-0.05 or 0.05 -0.10 m. So, high concentration

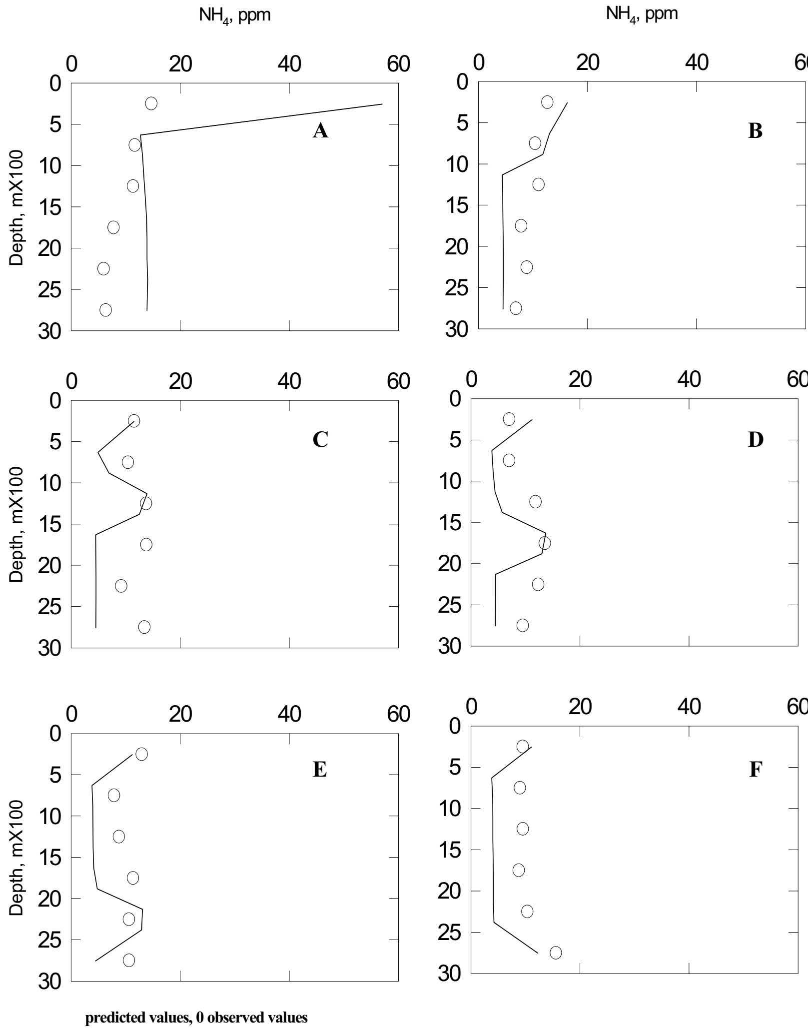

Figure 2: The predicted and observed amonium. A, B, C, D, E, and F referred to the depths of municipal waste layer 0.0-0.05, 0.05-0.1, 0.1-0.15, 0.15-0.2, 0.2-0.25, and 0.25-0.3 m, respectively.

of NO_3 in the top layers might be due to the high convective flux of nitrate and the high nitrification processes in these layers. The NO_3 solutions flowed through the soil column almost as quickly as the water because they are not adsorbed by soil constitutes Bohn et al. [20]. They also reported that in well–aerated soils, with adequate moisture, moderate temperature, NH_4 are converted to NO_3 in a matter of weeks. Cumulative net nitrogen that was mineralized was linearly related to the square root of time Barber [21]. Values for the mineralization potential varied from 20 to 300 mg/kg of air-dry soil. This potential was 5 to 40% of total nitrogen in the soil, with an average 18% for 39 soils studied. The mineralization rate constant averaged 0.054 ± 0.009/week. Barber [22] measured the reduction in soil organic matter content in plots that were followed for six years; the rate was 2.4% per year while Larson et al. [23] obtained a value of 1.9% per year in Iowa.

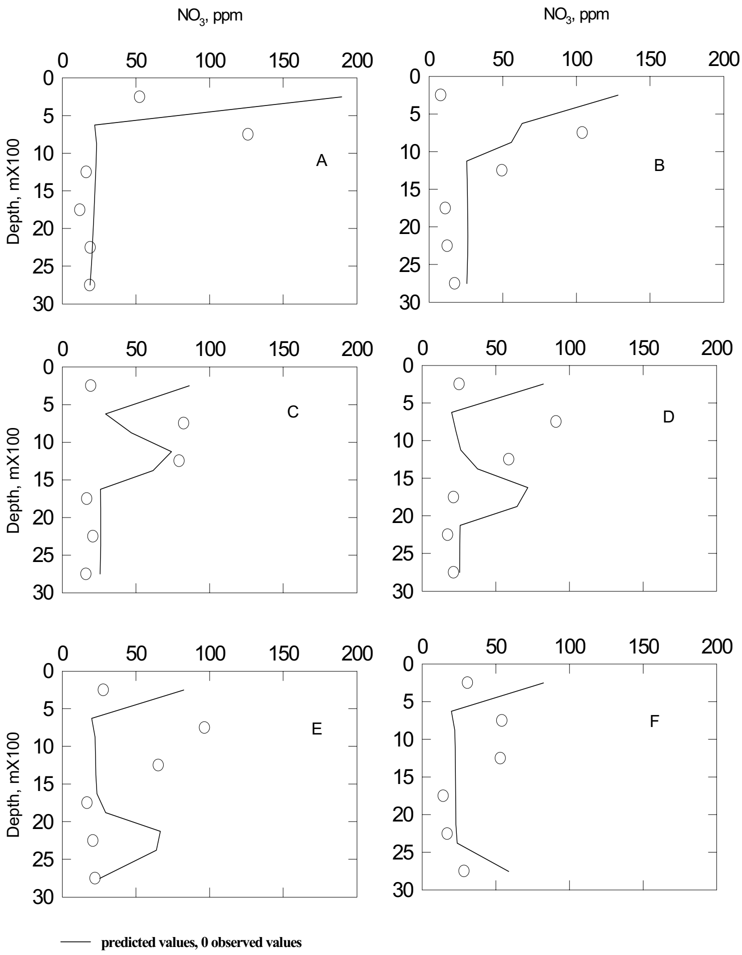

Figure 3: The predicted and observed nitrate. A, B, C, D, E, and F referred to the depths of municipal waste layer 0.0-0.05, 0.05-0.10, 0.1-0.015, 0.15-0.20, 0.20-0.25, and 0.25-0.3 m, respectively.

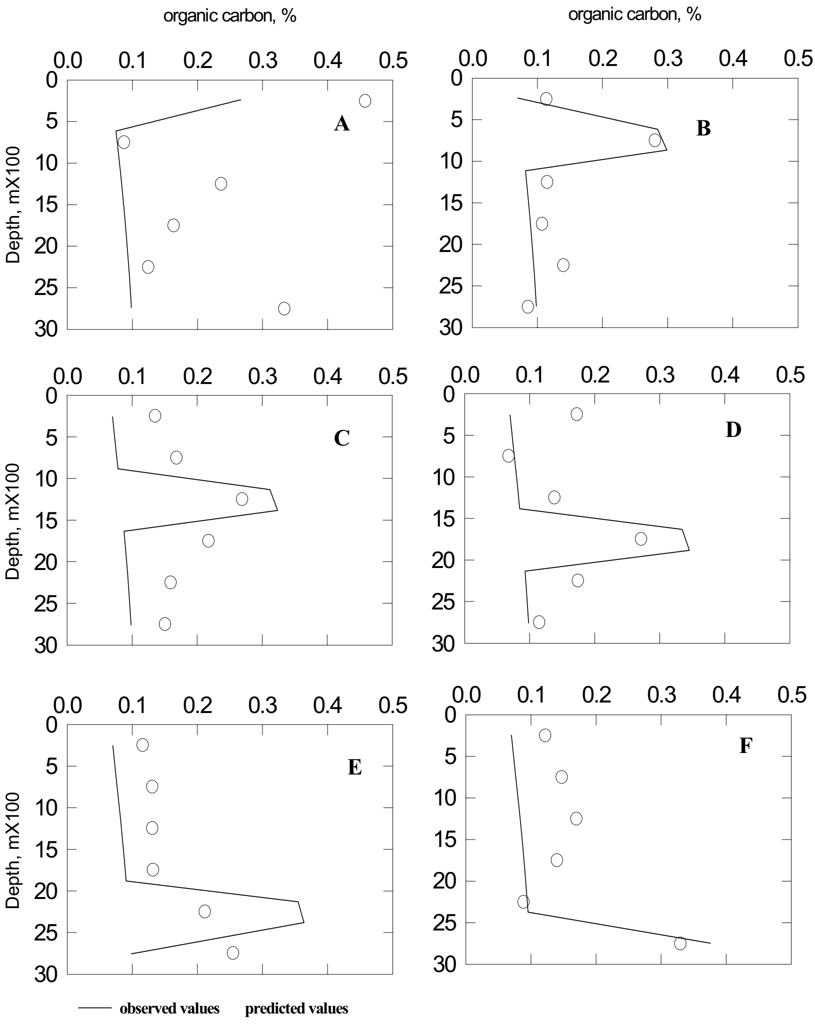

Figure 4: The predicted and observed organic carbon. A, B, C, D, E, and F referred to the depths of municipal waste layer at 0.0-0.05, 0.05-0.1, 0.1-0.15, 0.15-0.2, 0.2-0.25, and 0.25-0.3 m, respectively.

Figure 4 shows the calculated and observed organic carbon. The LEACHM model predicted organic carbon values in close agreement with the observed values within most of the soil columns. The mean of organic carbon was little less than the initial organic carbon (0.172%) in four treatments. The difference between the means of theses treatments and the initial organic carbon was small because of the soil water content was small during running period of the experiment and the duration of experiment was short (2 months). Parton et al [3] reported that the rate of decomposition of soil organic matter depends on soil types; soil texture (silt + clay content); rainfall; soil temperature; Potential evapotranspiration; initial soil organic carbon content and the maximum decomposition rate parameter. Parton et al. [24] found that CENTURY model simulated soil C and N levels within 25% of the observed values. Field losses of total C averaged 28% over the life of an old ant nest (between 30 and 60 yr according to Coffin and Lauenroth [25]. Gilmanov et al. [26] found that CENTURY model reproduced the seasonal, mid-term, and in some cases, long-term dynamics in aboveground biomass in a wide range of grassland ecosystems across the former USSR. They attributed model discrepancies to changes in species composition and short-term responses to intermittent rainfall that are missed by the monthly time step of the model. However, decay constants were not always in agreement with conceptual models. The authors' estimated values of active soil organic matter (SOM) turnover (k = 0.007-0.035) were lower than predicted by the three theoretical models van Veen and Paul [4], Parton et al. [5], and Jenkinson [6] to which they made comparisons (k = 0.29-0.66). Gilmanov et al. [26] used CENTURY model output to calculate simulated SOM losses over time due to removal of plants. Over 60 yr, simulated total soil C decreased 63% from 2074 g/[m.sup.2] to 828 g/[m.sup.2]. Their measured loss of total C on ant mounds over an estimated 30-60 yr was approximately 28%. The CENTURY model predicted an annual (k-based) loss of total C of 1.6-2.1%, while measured loss rates (k-based) range from 0.5 to 1.1%.

3.2 Predicted and measured soil water status

The predicted and measured soil water content distributions in nonisothermal columns are shown in Fig.5. The predicted water content distributions matched the observed values only in trend. There are two distinguished zones for the observed distribution of the soil water contents. The first zone was located in the upper 0.2 m while the second zone was located in the lower 0.1 m layer. The water content changed abruptly from the upper to the lower zones. The mean soil water contents calculated from the observed data were 4.79, 4.17, 4.90, 4.56, 3.58 and 3.69% for the treatments 1, 2, 3, 4, 5, and 6, respectively. The model overestimated the soil water content in the upper zone while it is underestimated the soil water content in the lower zone. The means of soil water (approximately 4.9%) obtained from the predicted values were greater than the observed means. Overestimation of the predicted soil water content is due partially to underestimation of water vapor movement. Similar results were reported by Nassar et al. [27] and Al-Salamah [19]. Several researchers found that temperature gradient has great effect on water flow Globus [28] and Nassar et al. [29].

Figure 6 shows the stored soil water after 60 d as a function of the depth of mixed layer. The stored water content decreased as the depth of mixed layer increased. For example, the stored water contents were 0.0062 and 0.0042 m/column when the mixed layer at 0-0.05 and 0.25-0.30 m, respectively. The stored soil water as a function of the mixed layer depth described using a liner relation. The function described 72% of the variations. Similar results were reported by Al-Salamah and Nassar [30] when municipal waste was located at the top, mixed in the top layer or located at subsurface layer. According to cumulative evaporation reported by Al-Salamah and Nassar [30], the four application aspects followed the order: the bare soil > mixed layer> subsurface layer> top layer.

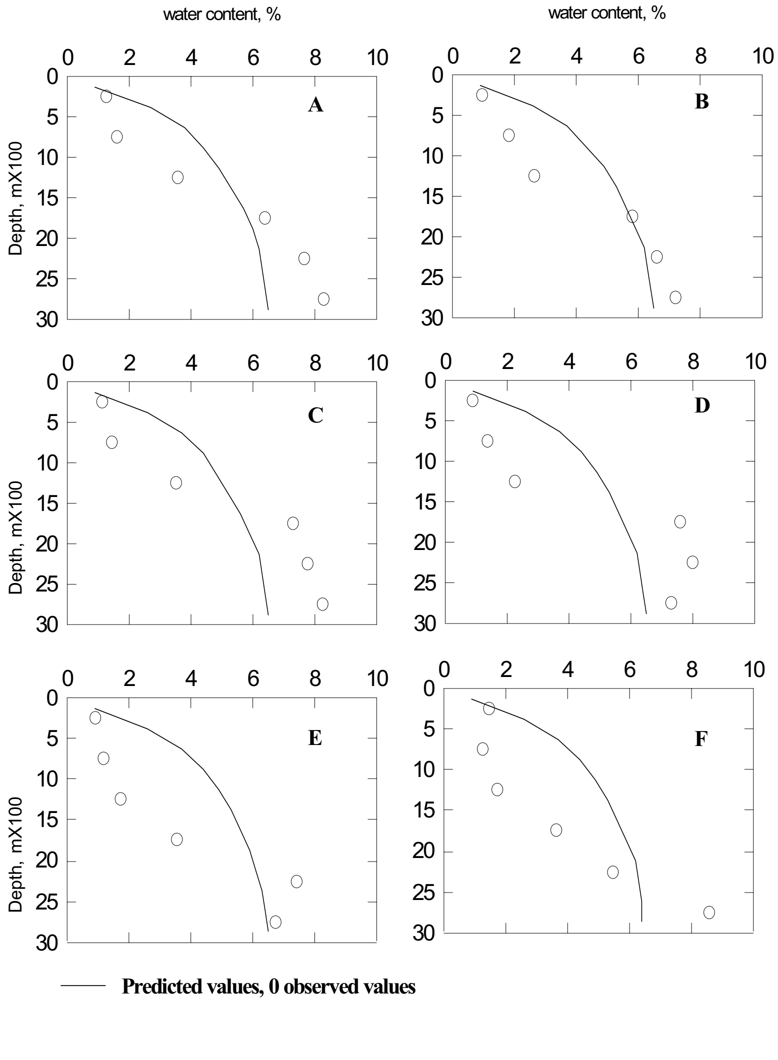

Figure 5: The predicted and observed volumetric water content. A, B, C, D, E, and F referred to the depths of municipal waste layers 0.0-0.05, 0.05-0.10, 0.1-0.15, 0.15-0.20, 0.20-0.25, and 0.25-0.30 m, respectively.

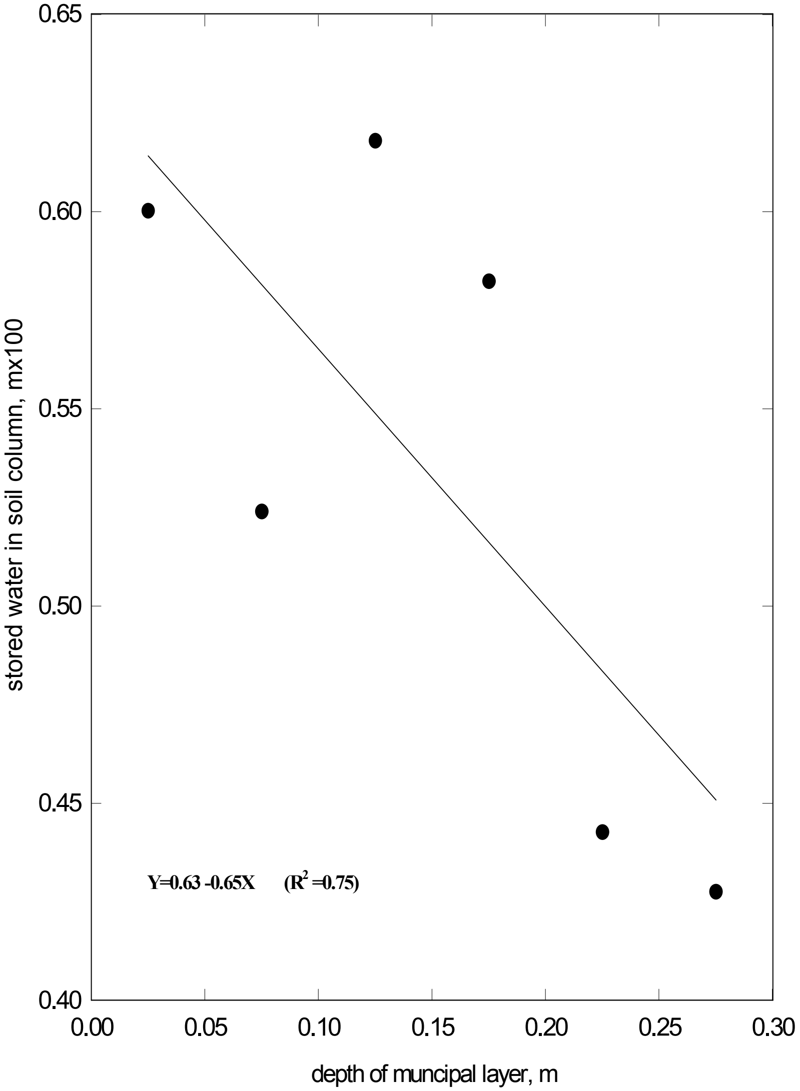

Figure 6: Effects of municipal wastes depth on the stored water in a soil column.

3.3 Model evaluation

Table 4 shows the observed mean, mean error (ME) and root mean square error (RMSE) for the P, NH_4, NO_3, Organic carbon and water content (Figures 1–5). The observed means of P ranged from 12.11 to 14.86 ppm and ME ranged from 2.71 to 5.94 ppm. The ratio of ME to the observed mean concentration of P varied from treatments to other. This ratio ranged from 0.22 to 0.39. The lower ratio indicates that high performance of the model for P prediction. The observed means for NH_4 ranged from 9.61 to 12.01 ppm and the ME ranged from 1.82 to 4.63 ppm. The ratio of the ME to the observed mean was 0.18 to 0.38. It can be concluded that the performance of the LEACHM model in the prediction of NH_4 is similar to the prediction of P. The ratio of the ME to the observed means ranged from 0.03 to 0.21, 0.14 to 0.47 and 0.01 to 0.32 for the NO_3, organic carbon content and water content, respectively. It can be concluded that the performance of the LEACHM model in prediction of nitrate is better than the prediction of either water content or organic carbon content. The RMSE values behaved similarly to the ME for P, NH_4, NO_3, organic carbon, and the water content. It can be concluded that the model described the mechanisms of organic matter decomposition and P, NH_4, NO_3 and water flows appropriately under the conditions of the present study.

Table 4: The observed mean, mean error (ME) and root mean square error (RMSE) for the data presented in Figures 1–5. The symbols are shown in the figures.

Figure No	Param-eters	Figures symbols					
		A	B	C	D	E	F
Fig. (Av..P) (ppm)	Mean	13.56	12.81	13.63	12.11	14.86	13.19
	ME	2.345	2.218333	3.6875	2.7195	5.9425	4.671667
	RMSE	8.43041	8.725044	7.531261	6.101289	6.507956	4.899321
Fig. 2 (NH_4) (ppm)	Mean	9.63	9.61	12.01	10.21	10.34	10.45
	ME	-11.2157	1.828167	4.639667	3.119667	3.504167	3.8685
	RMSE	18.04968	4.004083	5.881585	5.159043	4.683364	4.667167
Fig. 3 (NO_3) (ppm)	Mean	40.56	33.61	38.96	39.02	41.43	32.79
	ME	-8.645	-15.0242	-6.06	-3.46667	1.04	-5.595
	RMSE	70.32737	54.05466	33.64538	43.02575	45.71885	30.90611
Fig. 4 (O. C) %	Mean	0.233	0.140	0.183	0.156	0.162	0.166
	ME	0.116	0.019	0.059	0.029	0.033	0.034
	RMSE	0.142	0.030	0.080	0.065	0.096	0.057
Fig. 5 (Water) %	Mean	4.79	4.17	4.9	4.56	3.58	3.68
	ME	-0.1285	-0.71105	0.025667	-0.32728	-1.29512	-1.2055
	RMSE	1.554069	1.421457	1.701784	1.968665	2.110211	2.193406

4 Summary and conclusion

A greenhouse experiment was achieved for monitoring organic matter decomposition, P, NO_3, NH_4 and water transfer in unsaturated soil column. Municipal wastes material as a source of organic matter was mixed with soil and located at different depths within the soil columns. The soil columns were one-end open and buried vertically in a bare soil field in Al-Qassium region, KSA. Distilled water was added several times during the course of the experiment. Presences of municipal waste mixed layer close to the soil surfaces reduced water losses in comparison to lower depth. The LEACHM model was used to predict soil organic matter decomposition, and P, No_3, NH_4 and water content within the soil columns. The predicted and observed variables were similar in trends. However, there were some discrepancies between the observed and predicted values especially in the upper portion of soil columns. The application of this model may be very useful for the monitoring soil organic matter decomposition and the fate of nutrients (P, C, and N) in soil. This model can be a good tool in the fertilization strategies for plants. It is also useful for monitoring nitrate transfer and its deterioration effects on water resources.

References

[1] Froelich, P. N., Klinkhammer, G. P, Bender, M. L., Luedtke, N., Heath, G. R., Cullen, D., Dauphin, P., Hammond, D., Hartman, B. & Maynard, V., Early oxidation of organic matter in pelagic sediments of the eastern equatorial Atlantic; suboxic diagenesis, Geochimica et Cosmochimica Acta Pergamon, Oxford, 43, pp. 1075-1090, 1979.

[2] Drever, J.I., The Geochemistry of Natural Waters, Prentice-Hall, Inc., Englewoods Cliffs, N.J., pp. 388, 1982

[3] Parton, W.J, Stewart, J. W. B. & Cole, C. V., Dynamics of C, N,P and S in Grassland Soils - A model. Bio-geochemistry, 5, pp. 109-132, 1988.

[4] van Veen, J.A & Paul, E. A., Organic C dynamics in grassland soils-I. Background information and computer simulation. *Can.J.Soil Sci.,* 61, pp. 185-201, 1981

[5] Parton, W. J., Schimel, D. S., Cole, C. V. & Ojima, D. S., Analysis of factors controlling SOM levels in Great Plains grasslands. *Soil Science Society of America Journal*, 51, pp. 1173-1178, 1987

[6] Jenkinson, D. S., The turnover of carbon and nitrogen in soil. *Philosophical Transactions of the Royal Society of London*, B 329, pp. 361-368, 1990

[7] Hutson, J. L. & Wagenet, R. J., LEACHM: Leaching Estimation And Chemistry Model: A process based model of water and solute movement transformation, plant uptake and chemical reactions in the unsaturated zone. Continuum Vol.2, Version 3. Water Resources Inst., Cornell University, Ithaca, NY, 1992.

[8] Johnsson, H., Bergstrom, L., Janson, P.E. & Paustian, K., Simulated nitrogen dynamics and losses in a layered agricultural soil. *Agriculture, Ecosystems and Environment*, 18, pp. 333-356, 1987.

[9] Shaviv, A. & Shachar , A. N., A kinetic-mechanistic model of phosphorus sorption in calcareous soils. *Soil Sci*., 148, pp. 172-178, 1989.

[10] Olsen, S. R., Cole, C. V. & Dean, A., Estimation of available phosphorus in soils by extraction with sodium bicarbonate, U. S. Dept. Agric., Cir. 939, 1954.

[11] Jackson, M. L., Soil Chemical Analysis. Prentice Hall of India Private Limited, New Delhi, 1967.

[12] Nelson, D. W. & Sommers, L. E., Total carbon, organic carbon, and organic matter. In p.539-579. In A.L. Page et al. (ed). Methods of Soils Analysis, Part 2. Agronomy 9: Madison, WI, 1982

[13] Page, A.L., Miller, R.H. & Kenney, D.R., Methods of Soils Analysis, Part 2. Agronomy 9, Madison, WI, 1982.

[14] Campbell, G. S. A simple method for determining unsaturated conductivity from moisture retention data, *Soil Sci*., 117, pp. 311-314, 1974.
[15] Nassar, I. N. & Horton, R., Transport and fate of volatile organic chemicals in unsaturated, nonisothermal salty porous media: 1- Theoretical Development, *J. Hazardous Materials*, B69, pp. 151-167, 1999.
[16] Lyman, W. L., Reehl, W. F. & Rosenblatt, D. H., Handbook of chemical property estimation methods. McGraw-Hill Book Co., Inc., New York, N. Y., 1982.
[17] Milly, P.C.D., A simulation analysis of thermal effects on evaporation from soil, *Water Resour. Res*., 20, pp. 1087-1098, 1984.
[18] Klute, A. & Dirksen, C. D., Hydraulic conductivity and diffusivity: Laboratory methods. In Method of Soil Analysis. Part 1. (2nd Edn) (Edited by A. Klute) Chap. 28, Agron. Monogr. 9. ASAS and SSSA, Madison, WI, 1986.
[19] Al-Salamah, I.S., Simulating the Fate and Transport of Pesticide in Unsaturated Soil: a Case Study with Glyphosate Isopropylammonium, International Conference on Monitoring, Simulation and Remediation of the Geological Environment, 5 - 7 July 2004. Segovia, Spain.
[20] Bohn, H. L., McNeal, B. L. & O'Connor G. A., Soil Chemistry, 2nd ed. John Wiley & Sons, Inc. New York, Toronto, 1985
[21] Barber, S. A., Soil Nutrient bioavailability, A Mechanistic Approach, John Wiley& Sons, Inc. New York, Toronto, 1984.
[22] Barber, S. A., Corn residue management and soil organic matter, *Agron. J.,* 71, pp. 625-628, 1979.
[23] Larson, W. E., Clapp, C. E., Pierre, W. H. & Morachan, Y. B., Effects of increasing amount of organic residues on continues corn: II. Organic carbon, nitrogen, phosphorus and sulfur, *Agron. J*., 64, pp. 204-208, 1972.
[24] Parton, W.J., Scurlock, J.M.O., Ojima, D.S., Gilmanov, T.G. & Scholes, R.J., Observations and modeling of biomass and soil organic matter dynamics for the grassland biome worldwide, *Global Biogeochemical Cycles*, 7,4, pp. 785-809, 1993.
[25] Coffin, D. P. & Lauenroth, W. K., Vegetation associated with nest sites of Western Harvester ants (Pogonomyrmex occidentalis Cresson) in a semiarid grassland, *American Midland Naturalist*, 123, pp. 226-235, 1990.
[26] Gilmanov, T. G., Parton W.J. & Ojima, D. S., Testing the CENTURY ecosystem level model on data sets from eight grassland sites in the former USSR representing a wide climatic/soil gradient, Ecological Modelling, 96, pp. 191-210, 1997.
[27] Nassar, I. N., Shafey, H. M. & Horton, R.,. Heat, water, and solute transfer in unsaturated soil: II- Compacted soil beneath plastic cover, *Transport in Porous Media*, 27, pp. 39-55, 1997.
[28] Globus, A. M., Physics of Non-Isothermal Soil Moisture Transfer. Hydrometeorological Publ., Leningrad (Russian) pp. 171-191, 1983.

[29] Nassar, I. N., Globus, A. M. & Horton, R., Simultaneous soil heat, and water transfer. *Soil Sci.*, 154, pp. 465-472, 1992.
[30] Al-Salamah, I. S. & Nassar, I. N., Utilization of municipal garbage for soil evaporation suppression and as a source of plant nutrients*, J. Saudi Soc. Agric. Sci.*, 1, pp. 62-77, 2002.

Section 3
Water quality

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Advances in mathematical modelling of biofilm structures

C. S. Laspidou[1], A. Kungolos[2] & P. Samaras[3]
[1]*Hydromechanics and Environmental Engineering Laboratory, Department of Civil Engineering, University of Thessaly, Greece*
[2]*Department of Housing and Regional Development, University of Thessaly, Greece*
[3]*Technological Institute of Western Macedonia, Department of Pollution Control Technologies, Greece*

Abstract

Biofilms are found everywhere in natural waters forming complex microbial communities, playing an important role in ecosystem processes in streams and lakes and making their characterization a major research question. Biofilm models are commonly used as simulation tools in engineering applications and as research tools to identify and fill gaps in our knowledge of biofilm processes. Recently, mathematical biofilm models are becoming more complex and use advanced computational tools to predict the new structure formed under a set of given conditions, as well as the microbial species that comprise the biofilm structure. Moreover, advanced biofilm models incorporate the hydrodynamics of the fluid surrounding the growing biofilm and translate it into forces that act on the biofilm and cause detachment. Thus, they enhance their predictive capability over traditional biofilm models and enable research scientists and engineers to evaluate the relevance of biofilm heterogeneities to their function. An overview of the features of the unified multi-component cellular automaton (UMCCA) model is presented. The UMCCA model describes quantitatively the complexity of biofilms for all biofilm components: active bacteria, inert biomass, and extracellular polymeric substances (EPS). It also includes original donor substrate, two types of soluble microbial products (SMP), and oxygen. The UMCCA model captures all trends observed experimentally regarding biofilm density by employing the novel idea of biofilm consolidation, according to which the biofilm packs itself to a higher density over time. The UMCCA model can also be used to describe biofilm mechanical properties variable in time and space, making it possible to predict where it is likely to fail, or detach.
Keywords: biofilm modelling, extracellular polymeric substances, EPS, biofilm density, inert biomass, active biomass, soluble microbial products, SMP.

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1 Introduction

Biofilms are ubiquitous in nature and play a critical role in various biological processes in streams and lakes. Factors responsible for the abundance and growth of periphyton a natural biofilm found in streams and lakes, have been studied previously Hepinstall and Fuller [1], Rier and Stevenson [2], Sobczak [3]. Such factors that may affect microbial communities in streams and lakes are dissolved organic matter and inorganic nutrients. Biofilms are layerlike aggregations of microorganisms and their extracellular polymeric substances (EPS) attached to a solid surface. Some biofilms are viewed as “good” and we try to promote their accumulation. Examples are those exploited in fixed-film processes used to treat contaminated water, wastewater and air, or those that attach to stream beds and aquatic vegetation leading to self-purification of water bodies, and those responsible for engineered or intrinsic bioremediation of contaminated groundwater. Other biofilms are viewed as “bad,” and we try to remove or prevent them. Bad biofilms include those that foul ship hulls, thereby increasing friction loss and corrosion and those that accumulate in pipes in water distribution systems possibly carrying pathogens or corroding them. Thus, microbial biofilms can be of great benefit but they can also be a nuisance. Humans may want to control or enhance their presence, both in technical and natural systems. Therefore, it is extremely important to understand the structure and function of biofilm communities as well as the mechanisms regulating biofilm processes.

Biofilms are highly heterogeneous and diverse. They could be thick or thin, “fluffy” or dense, forming a solid mat or having finger-like protrusions in the overlying fluid and having clusters or streamers of biomass intermingled with open channels. They could include several different types of microbial species, such as heterotrophs and autotrophs. Even when biofilms do not include different microbial types, they are still heterogeneous since bacteria always produce EPS and “inert” or dead biomass Laspidou and Rittmann [4]. Biofilm research faces invariably the challenge of understanding complex relationships between physical, chemical and biological processes occurring at very different spatial and temporal scales. The best tool available for integrating the plethora of experimental observations in a rational environment is mathematical modeling. Mathematical models are no more than a simplified representation of reality consisting of sets of equations and algorithms containing the information needed to simulate a system. Models can lead to a deeper understanding of the underlying principles and to the potential of making predictions, so they are valuable computational tools.

Microbial biofilms have been simulated by mathematical models for the last three decades Noguera *et al* [5]. The initial models described biofilms as uniform steady-state films containing a single type of organism, governed exclusively by one-dimensional mass transport and biochemical transformations Rittmann and McCarty [6]. Later, layered dynamic models were developed Wanner and Gujer [7], which included multi-species interactions within the biofilm, but were not able to represent the characteristic structural heterogeneity that has been recently elucidated through experimental observations. The development of advanced microscopy techniques like Confocal Scanning Laser Microscopy (CSLM) has allowed researchers to really see what biofilms look like; thus, it became obvious that they do not grow in flat layers, but have channels and pores, and mushroom-like or tulip-like protrusions in the fluid, depending on the conditions the biofilm is grown under (Figure 1). It was therefore shown that the models that wanted the biofilms growing in layers were not really capturing the structural heterogeneity of biofilms; this realization motivated an explosion in the development of mathematical models during the past decade. This explosion was also motivated by the advances in computing capabilities that made it possible to perform the massive calculations that the new models needed, in a short period of time.

2 Objectives of biofilm modeling

During biofilm development, a large number of phenomena occur simultaneously and interact over a wide range of length and time scales. Nutrients are being taken up and converted and the biofilm expands as a result of bacterial growth and production of EPS. Chemical species are continuously transported to and from the biofilm system by physical processes such as molecular diffusion and convection. Fluid flow influences biofilm growth in two ways: first, by determining the concentrations of available substrates and products and second, by shearing the biofilm surface with a detachment process Rittmann and Laspidou [8]. In the case of multi-species systems, microorganisms of different species interact in complex relationships of competition or cooperation. All these phenomena create a dynamic picture of the biofilm structure. Mathematical models can prove useful because they allow testing of hypotheses and, in addition, can direct experimental efforts to complex regions of operation that need to be further investigated. In addition, biofilm models are frequently used by practicing engineers as a simulation tool to analyze the performance of biofilm processes. The models provide engineers with the means to evaluate the significance of several parameters, allowing them to search for explanations of performance problems. Thus, it is possible to formulate hypothetical modifications in operation and to simulate process behavior in response to operational changes before full-scale implementation.

In relation to research applications, mathematical models can provide useful insights into the evaluation of biofilm parameters, especially when used in combination with experimental results. As for practical engineering applications, the current objectives of biofilm modelling include biofilm engineering, real-time control and applications in education. Therefore, models can help engineers gain an insight into the interactions between the processes involved in biofilm formation so that it would be possible to design the biofilm structure and its function. For example, the environmental conditions can be manipulated to generate dense biofilm structures that will be easily separated from a liquid phase (as in a fluidized bed reactor), or multi-layered biofilms that would block corrosion of ship hulls, or rough biofilm structures with high capacity for removal of particulate material.

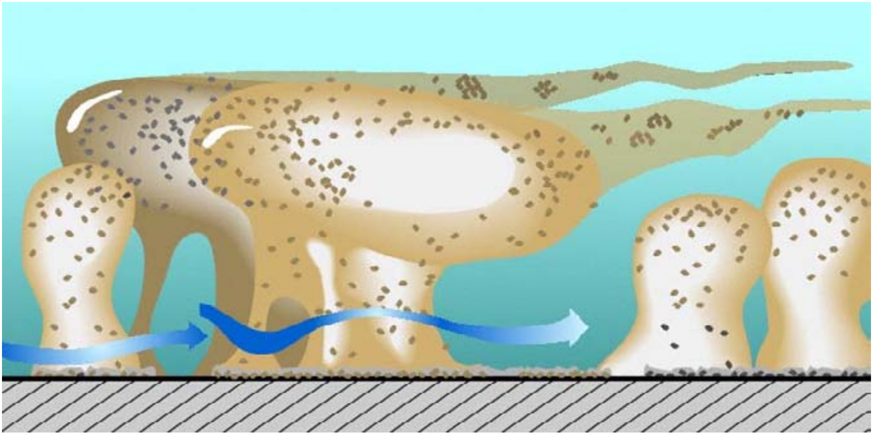


Figure 1: Conceptual illustration of the heterogeneity of biofilm structure, showing bacterial clusters, streamers, and water channels (adapted from a picture by the Center for Biofilm Engineering, Montana State University-Bozeman).

But why is it important to know the biofilm structure in the first place? The structure of biofilms plays a very important role in their activity. Together with the liquid flow, the shape of the biofilm surface influences the transport of all solutes from the bulk liquid to the microbial cells where the bioconversions occur. The structure, in turn, is itself defined by the biofilm activity. Bacterial cell growth and division, as well as EPS production and secretion, together with external factors such as shear forces defined by hydrodynamics and other mechanical forces shape the biofilm structure Picioreanu *et al* [9]. Depending on these factors, the biofilm develops at a variable density, i.e. it is either loose and "fluffy" or dense and compact. Naturally, biofilm density plays a critical role in biofilm detachment. Therefore, in a cyclical almost way, the hydrodynamics apply forces on the biofilm and cause detachment thus forming its surface, which in turn plays a role in substrate transport, defines biofilm activity and growth, which changes the biofilm surface and once again plays a role in hydrodynamics (Figure 2).

3 The UMCCA model

A sophisticated mechanistic model of biofilm structures is the Unified Multi-Component Cellular Automaton (UMCCA) model, which predicts quantitatively all biofilm components: active bacteria, inert (or dead) biomass produced by death and decay, and EPS. An overview of the model is presented in Laspidou and Rittmann [10]. This model is the biofilm adaptation of the multi-component mathematical model that quantifies the previously developed unified theory that reconciles the apparently disparate findings about active and inert biomass, EPS and other microbial products Laspidou and Rittmann [4, 11]. The unified theory provides a set of mathematical equations that describe the development of all solid and soluble species in the biofilm. The UMCCA model represents a growing biofilm using a cellular automaton (CA) approach in which the biofilm grows in a two-dimensional domain of compartments.

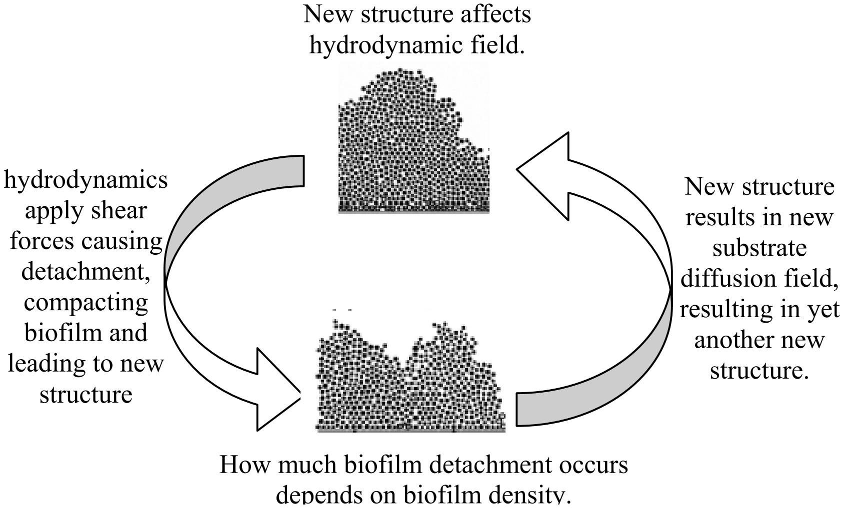
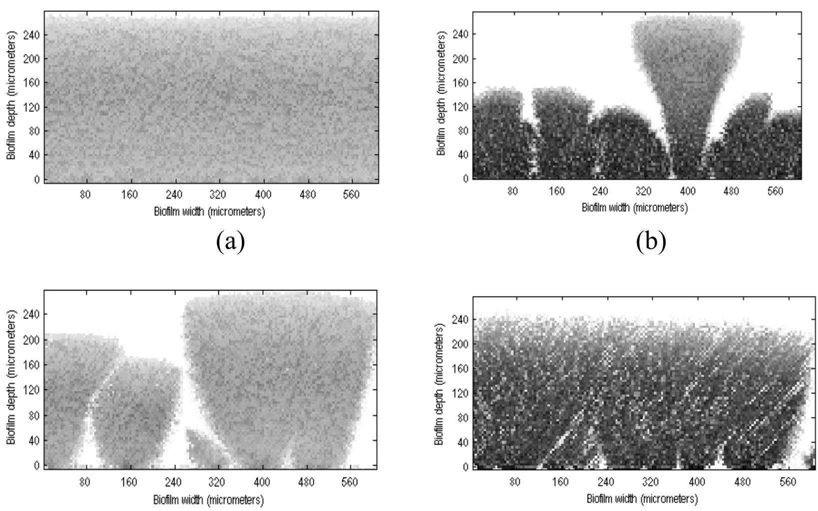


Figure 2: Hydrodynamics and substrate diffusion interact in forming the biofilm structure.

One of the unique features of the UMCCA model is that it can predict a variable biofilm density throughout the biofilm column. This is important as experiments show that the biofilm that is closest to the substratum has the highest density, while the top layers remain “fluffy” and porous, with the bottom of the biofilm being as much as 5 to 10 times denser than the top Bishop *et al* [12]. Biofilm porosity follows opposite trends, suggesting that biofilm accumulates near the substratum with a closer packing arrangement than at the top. In order for UMCCA to capture these density differences, it employs the theory of *consolidation*, an idea borrowed from the fields of geotechnical engineering Rittmann and Laspidou [13, 14]. According to this theory, when pressure is applied to a bed of irregularly sized solids, the solids gradually pack to a higher density, or consolidate. The solids do this to minimize porosity and achieve a lower energy state. When the applied pressure is combined with vibration consolidation is accelerated. Because a biofilm is a matrix of particles, UMCCA includes equations that allow it to consolidate, or gradually pack to a higher density, due to the differential forces, such as friction, which have developed as a result of the fluid flowing over it. Thus, the bottom biofilm layers, those closest to the substratum undergo consolidation for the longest time; therefore, the bottom layers should have the highest densities and the lowest porosities, which is what Bishop *et al* [12] observed in their experiments.

The UMCCA model is a hybrid differential and discrete cellular automaton (CA) model; thus, solutes are represented in a continuous field by reaction-diffusion mass balances, while solids are mapped in a discrete cell-by-cell fashion that uses a CA algorithm. The CA algorithm is a set of rules that determines where to place the newly-formed biomass, how to expand the biofilm front by "pushing" neighbouring cells when new biomass grows in. The model includes seven major variables: active biomass, EPS and inert biomass, as well as original donor substrate, two kinds of soluble products and oxygen. The model's general objective is to describe quantitatively the heterogeneity of biofilms.



(a) (b)

(c) (d)

Figure 3: Sample UMCCA model outputs of composite density. (a) Standard conditions (24.5 days of simulated time), (b) low substrate concentration (221 days), (c) high detachment rate (30 days) and (d) low oxygen conditions (195 days).

Figure 3 shows some sample model outputs of composite density. These are presented in a shading format, with a shade of grey being associated with a composite density: a black compartment represents a biofilm location that is filled completely (100%) with a combination of solid species, while a white compartment represents an empty compartment. Figure 3 shows that different conditions produce different physical structures. Biofilms may grow in "mat" formations (Figure 3(a)), in "mushrooms" (Figures 3(b) and (c)) and in mat formations with "holes" close to the substratum (Figure 3(d)). Figure 3(b) and (d) show biofilms with much higher densities at the bottom, while Figures 3(a) and (c) show lower and more uniform densities throughout the column of the biofilm. But why are the biofilm structures in Figure 3 so different so different with respect to density and heterogeneity? Before this question is answered, an analysis of the composite density graph in Figure 3(a) is presented (shown again in Figure 4(d)).

Figure 4 shows a breakdown of the individual solid species that comprise the biofilm density (active biomass, EPS and residual inert biomass) for the composite density in Figure 3(a), which corresponds to a simulation of biofilm after 24.5 days. Figure 4(a) tells us that active biomass is close to zero at the bottom of the biofilm, since it had enough time to decay almost completely to residual inert biomass, which is what is shown in Figure 4(b). Active biomass is also at a low density at the top of the biofilm; here, the reason is that the biofilm in these rows is young and has had no time to fill in all the space and consolidate. Hence, the density of active biomass peaks at about one-quarter of the depth of the biofilm, where substantial synthesis has had time to occur, but decay is not yet dominant. Residual biomass, shown in Figure 4(b) is produced only after active biomass is synthesized and has had enough time to decay. Figure 4(c) shows that EPS follows the active-biomass profile. The total composite density, Figure 4(d), is a composite of the three biomass component. Its maximum is close to the middle of the biofilm. The composite density for this case is not especially high, compared to Figures 3(b) and (d), since the age of this biofilm is modest, only 24.5 days.

The graph of composite density in Figure 3(b) is very different from that in Figure 3(a), since it is a result of a run with lower substrate concentration. The slower biofilm growth rate, due to the lower substrate concentration, required 221 days of simulated time. At the bottom of the biofilm, substrate is nearly zero and active biomass has had a long time to decay almost totally to inert biomass. In Figure 3(b), it appears that a cluster protrudes higher than the rest of the biofilm; this happens because, once a cluster manages to protrude, it is exposed to a higher substrate concentration than the other clusters are, it gets a growth-rate advantage and then it keeps growing faster than the rest of the biofilm. Figure 3(c) shows that a high-detachment-rate run took 30 days to fill the domain. The run was relatively short, because the composite density is lower overall, which means that less biomass is needed to fill the domain. The biofilm has a lower proportion of inert biomass (the highest-density solid species), because the loss of active biomass is shifted from decay to detachment. Figure 3(d) is a low-oxygen run that terminated in 195 days. In the absence of oxygen, active biomass growth ceases, while decay (not an oxygen-dependent process) proceeds. As a result, most of the biofilm is almost completely inert, with "holes" at its bottom that were unable to fill up with biomass owing to the lack of oxygen.

4 UMCCA and mechanical properties: the next phase

A new feature that was added to the UMCCA model is the ability to link a physical property—like composite density or the densities of active biomass, EPS, and inerts—to the mechanical properties of the biofilm, namely the

Young's modulus E and Poisson ratio ν (reference to the meaning of these new terms). In this way, we can use the UMCCA model to predict mechanical properties for the biofilm, and the mechanical properties can vary in time and in space. The significance of this feature is that we may gain insight into the issue of biofilm detachment, which can be viewed as a mechanical "failure" of the biofilm structure. Physical causes of detachment include forces acting on the biofilm structure itself, which most probably come from the fluid surrounding and moving around the biofilm and exerting shear stresses on it. Since the biofilm is heterogeneous, it is reasonable to assume that mechanical properties are not uniform either; thus, an analysis that predicts the mechanical properties throughout the biofilm is what is necessary to understand when, where, and why biofilm detachment occurs.

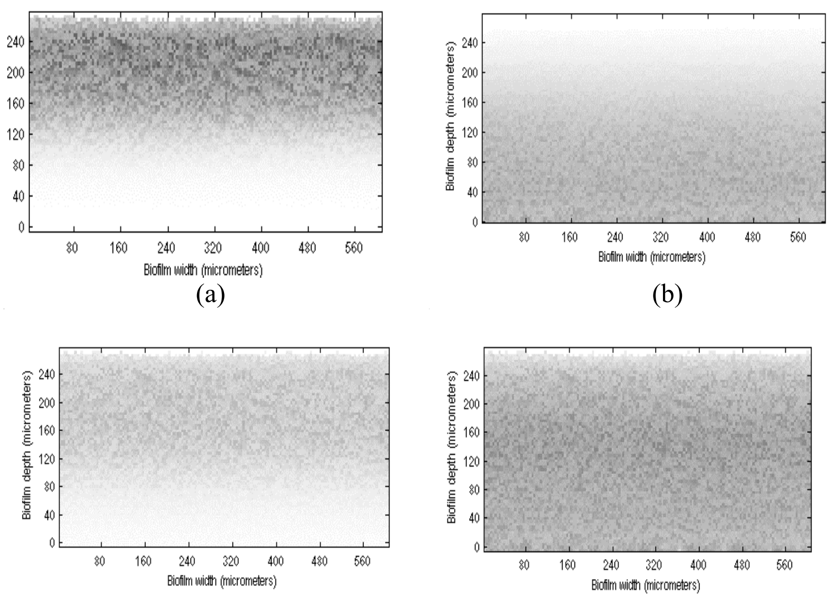


Figure 4: UMCCA model outputs of (a) active biomass (b) residual dead biomass (c) EPS and (d) the corresponding graph of composite density for the simulation also shown in Figure 3(a).

Once the variable mechanical properties of the biofilm from the UMCCA outputs are computed, they can be imported to a finite-element software used in structural mechanics, such as ABAQUS. When the forces acting on the biofilm and boundary conditions are provided, ABAQUS can be used to show how biofilms deform and what the internal stresses in the biofilm are. Locations where failure is likely can be predicted because the internal stress at that location exceeds the biofilm's yield strength (tensile or compressive). Limited tensile strength data were published by Ohashi *et al* [15] and can be used as a comparison level for such studies. A preliminary case study motivated from the experiments in [15] is presented in Laspidou *et al* [16].

5 Conclusions

Multi-dimensional modeling of biofilm structures constitutes a valuable tool for the investigation of the biofilm's structure as a result of its activity and the diffusion – reaction coupling. An overview of a quantitative biofilm model—the UMCCA model—that generates realistic biofilm structures formed under a set of given conditions relevant to natural ecosystems is presented. The UMCCA model incorporates the substrates and hydrodynamics of the fluid surrounding the growing biofilm and simulates the growth of biofilm structures predicting the different species that comprise the biofilm. It describes quantitatively the complexity of biofilms for all biofilm components: active bacteria, inert biomass, and extracellular polymeric substances (EPS). It includes original donor substrate, such as dissolved organic matter in a stream or lake, two types of soluble microbial products (SMP), and oxygen. The UMCCA model also employs the novel idea of biofilm consolidation, according to which the biofilm packs itself to a higher density over time. It can also be used to describe biofilm mechanical properties variable in time and space, making it possible to predict where it is likely to fail, or detach.

References

[1] Hepinstall, J.A. & Fuller, R.L., Periphyton reactions to different light and nutrient levels and the response of bacteria to these manipulations. *Arch. Hydrobiol.*, **131**, pp. 161-173, 1994.

[2] Rier, S.T. & Stevenson, R.J., Effects of light, dissolved organic carbon, and inorganic nutrients on the relationship between algae and heterotrophic bacteria in stream periphyton. *Hydrobiologia*, **489**, pp. 179-184, 2002.

[3] Sobczak, W. Epilithic bacterial responses to variation in algal biomass and labile dissolved organic carbon during biofilm colonization. *J. Norht Am. Benthol. Soc.*, **15**, pp. 143-154, 1996.

[4] Laspidou, C.S. & Rittmann, B.E., A unified theory for extracellular polymeric substances, soluble microbial products, and active and inert biomass. *Water Research,* **36**, pp. 2711-2720, 2002.

[5] Noguera, D.R., Okabe, S. & Picioreanu, C., Biofilm Modeling: Present Status and Future Directions. *Water Science and Technology*, **39(7)**, pp. 273-278, 1999.

[6] Rittmann, B.E. & McCarty, P.L. Model of Steady-State Biofilm Kinetics *Biotechnology and Bioengineering*, **22**, pp. 2343-2357, 1980.

[7] Wanner, O & Gujer, W., A multispecies biofilm model *Biotechnology and Bioengineering*, **28**, pp. 314-328, 1986.

[8] Rittmann, B. E. & Laspidou, C. S. Biofilm Detachment. *The Encyclopedia of Environmental Microbiology*, ed. G. Bitton, John Wiley & Sons, Inc., New York, 2001.
[9] Picioreanu, C., Xavier, J.B. van Loosdrecht, M.C.M., Advances in mathematical modeling of biofilm structure, *Biofilms*, **1(4)**, pp. 301-313, 2004.
[10] Laspidou, C.S. & Rittmann, B.E., Modeling biofilm complexity by including active and inert biomass and EPS. *Biofilms*, **1(4)**, pp. 285-291, 2004.
[11] Laspidou, C.S. & Rittmann, B.E., Non-steady state modeling of extracellular polymeric substances, soluble microbial products, and active and inert biomass, *Water Research* **36**, pp. 1983-1992, 2002.
[12] Bishop, P.L., Zhang, T.C. Fu, Y.-C., Effects of biofilm structure, microbial distributions and mass transport on biodegradation processes, *Water Science and Technology*, **31**(1), pp.143-152, 1995.
[13] Laspidou, C.S & Rittmann, B.E. Modeling the development of biofilm density including active bacteria, inert biomass, and extracellular polymeric substances, *Water Research* **38(14-15)**, pp. 3349-3361, 2004.
[14] Laspidou, C.S. & Rittmann, B.E., Evaluating trends in biofilm density using the UMCCA model, *Water Research*, **38(14-15)**, pp. 3362-3372, 2004.
[15] Ohashi, A., Koyama, T., Syutsubo, K., Harada, H., A novel method for evaluation of biofilm tensile strength resisting to erosion. *Water Science and Technology*, **39(7)**, pp. 261-268, 1999.
[16] Laspidou, C.S., Rittmann, B. E. Karamanos, S.A. Finite-element modeling to expand the UMCCA model to describe biofilm mechanical strength, *Water Science and Technology*, **52(7)**, pp. 161-166, 2005.

Ecotypes in south-central Spain rivers: the interactions among land use and pollution

C. Llácer, J. L. Moreno & J. de las Heras
Centro Regional de Estudios del Agua,
Universidad de Castilla-La Mancha, España

Abstract

A previous physiographical river typology was carried out in south-central Spain by applying System B of the Water Framework Directive (2000/60/CE). The classification distinguished four main river ecotypes: calcareous headwaters, siliceous rivers, plain rivers and large rivers. Physico-chemical water quality was assessed during period 2001-2003 to determinate natural hydrochemical composition and the relationship between land uses and pollutants, mainly nutrients. The river ecotypes corresponded to distinct hydrochemical types. Relatively unpolluted sites between river ecotypes showed no differences in turbidity, DQO, TOC, dissolved oxygen, nitrate, nitrite and phosphate. However, differences in EC, TSS and ammonium concentration between ecotypes were recorded. Correlation and regression analyses showed a strong relationship among land uses and nutrient concentration. Additionally, nitrate concentrations increased significantly when a higher percentage of land was dedicated to agriculture and less to forest. Adequate fertilizer management to avoid high surpluses, and therefore increase depuration efficiency by sewage treatment plant are absolutely necessary measures to reach the environmental objectives set by the Water Framework Directive.
Keywords: stream ecotypes, water quality, nutrients, agriculture, Water Framework Directive, Castilla-La Mancha, Spain.

1 Introduction

Freshwater ecosystems are greatly influenced by the characteristics of catchment area, as geology, geomorphology and climate. Although, human activities carried out in river catchments can be the most important modifier of the natural chemical composition of freshwater [1, 2]. Agricultural activities, urban sewage

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and industrial waste have been identified as the most important sources of pollution in Mediterranean countries [3–8].

Agricultural activities potentially result in multiple impacts to nearby streams: e.g. alteration or loss of riparian vegetation and changes to water chemistry through chemical application to crop. These impacts are considered the main issues for almost every water category regarding pollution in Europe. But, in a great majority of the European countries, nutrient loading by agricultural runoff is the main problem in achieving the environmental objective of Water Framework Directive (2000/60/CE). It is well known that drainage from agricultural land produces large amounts of nitrate and phosphate Hynes [1]. Identification and quantification of those impacts are necessary for an adequate management of land and water resources in river catchments Petts and Calow [9].

The Region of Castilla-La Mancha is dominated by intensive agriculture (>50% of the total area) and nitrogen, phosphorus, and potassium-based fertilizers are generously applied to vineyards, cereals and corn crops during the last few decades Domínguez [10]. Until now, in the region, the chemical quality of streams that reflected high concentrations of nutrients has been assessed Moreno *et al*. [11]. Also, stream water quality has been monitored with elaborate biological indicators based on macrophytes communities Moreno *et al*. [12, 13] to assess the trophic status. Many studies have established the relationship between land use and pollutants in rivers [14–17]. This relationship is important for integrated, sustainable management and protection of Castilla-La Mancha freshwater resources, particularly in terms of new ecological targets for water quality management under the new directive (WFD).

The aims of this research are: (1) to describe the natural hydrochemical composition for four river ecotypes and (2) to determinate the relationship between land use and pollutants.

2 Methodology

2.1 Study area

This study was carried out in headwaters and middle reaches of five main Spanish river basins located in south-central Spain corresponding to the limits of the administrative region of Castilla-La Mancha, fig. 1. Climatic conditions correspond to Mediterranean regime, with marked continental conditions. Castilla-La Mancha (CLM) is a region with an extensive area (79.463 km^2, 15.7% of Spain) and very low population density (22.8 inhabitants km^{-2}) respect to the national averages (71 inhabitants km^{-2}). Agriculture constitutes the 53% of land use in Castilla-La Mancha, whereas in Spain it occupies only 37% of the national surface. As for lithology, we can distinguish the western zone dominated by quartzite, slate and granite (siliceous rocks), and the eastern areas formed by limestone, dolomite, sandstone and conglomerates (calcareous rocks). Sedimentary fills are located in the central area (“La Mancha” Plateau) and the main river valleys which are filled of Tertiary detritic sediments, marls, and gypsum.

An abiotic stream classification based on the most statistically significant non-human influenced environmental characteristics was used to subdivide the region into relatively homogeneous environmental zones Moreno *et al.* [11]. Thus, four river ecotypes were established: Calcareous Headwaters (CALH), Siliceous Rivers (SILRIV), Plain Rivers (PLRIV) and Large Rivers (LARIV). CALH and SILRIV include mainly headwaters whereas PLRIV and LARIV include middle and low reaches.

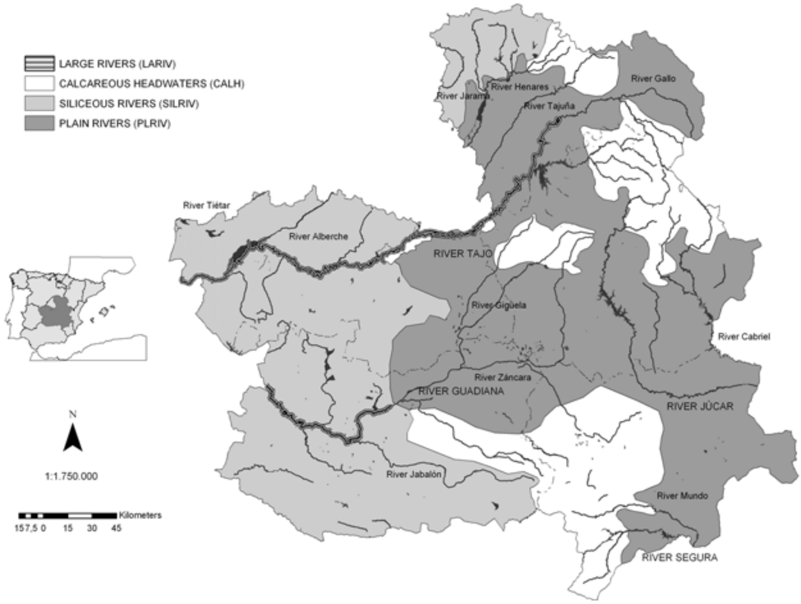

Figure 1: Study area showing main south-central Spanish rivers and boundaries of river ecotypes Moreno *et al.* [11].

2.2 Physico-chemical and land use

A total of 152 sites were selected in order to construct the basic monitoring network in the Castilla-La Mancha. Sites were visited once, twice or three times during period 2001-2003, with winter being the season with the least amount of available data. Electric conductivity, pH and dissolved oxygen were measured *in situ* using adequate sensors (Multiline P4 WTW). Water samples were collected in polyethylene bottles (500 ml) and were kept in the refrigerator (4ºC) until the water was analysed. The concentration of nutrients (NO_3^-, NO_2^-, NH_4^+, PO_4^{-3}) was determined photometrically with MERCK kits (Spectroquant®). Total suspended sediments (TSS), chemical oxygen demand (COD), total organic carbon (TOC), turbidity and major ions (HCO_3^-, Cl^-, SO_4^{-2}, Ca^{+2}, Mg^{+2}, K^+, Na^+) were obtained using standard procedures in the laboratory APHA [18].

Land use data were obtained by means of Geographical Information System (GIS) software (ESRI® ArcMap™ 8.2). Land uses were summarized into three

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categories from Corine Land Cover 1:100.000 (CLC 1990): agriculture, urban and forest.

2.3 Statistical analyses

The SPSS ver.11.5 (SPSS Inc., 2002) statistical software was used for statistical descriptors, analysis of variance and correlations. Non-parametric statistical procedures were applied when normality of residuals and homogeneity of variances did not match with parametric criteria. Simple regression was carried out using Statgraphics Plus v5.1. and data were transformed using $\log_{10}$ and percentage data with √arcsine to achieve normality assumptions and homocedasticity. Scatter Plots show transformed data.

3 Results

3.1 Physico-chemical water quality

In this case, relatively unpolluted streams were selected to determinate the water quality in natural conditions within the five river ecotypes. Selected criteria for unpolluted rivers corresponded to those established thresholds for Mediterranean Spanish rivers, $N\text{-}NH_4^+ < 0.5$ mg l^{-1} and $P\text{-}PO_4^{-3} < 0.05$ mg l^{-1} Bonada *et al.* [19]. Then, the river ecotypes showed distinct rock type were investigated the natural hydrochemical characteristics by comparing their ion composition and ion concentration. Calcareous rocks were dominant in CALH, siliceous rocks in SILRIV and detritic rocks in sedimentary valleys (PLRIV and LARIV). Major ions defined two main ion water types: calcium-bicarbonate and calcium-sulphate, fig. 2. CALH and SILRIV ecotypes include mainly headwaters and they were calcium-bicarbonate. Calcareous rivers showed higher significant concentration of major ions (bicarbonate, sulphate, calcium and magnesium) than siliceous rivers, showing the lower dissolution rates. LARIV and PLRIV were calcium-sulphate type, they showed a high similarity in ion composition but a high significant difference in sulphate concentration with respect to the other two ecotypes ($H_{3,41} = 24.33$, $p<0.001$). Large and plain rivers were located mainly on sedimentary detritic rocks and therefore they shared many physico-chemical features.

Mean values of pollutants for river ecotypes corresponding to relatively unpolluted streams, are shown in table 1. The analysis of variance between ecotypes only showed significant differences in electric conductivity, total suspended solids and ammonium concentration. The ecotype CALH presented a high nitrate concentration in natural conditions. The ecotype SILRIV showed the lowest electric conductivity and the highest large rivers (LARIV). The result of Tukey test did not fond differences in conductivity values between CALH and PLRIV, whereas significant differences in conductivity were found between SILRIV and LARIV, table 1. The concentration of ammonium in calcareous and siliceous rivers was lower than plain and large rivers showing differences between upland and lowland rivers. The LARIV ecotype presented the highest value of total suspend solid. Significant differences were observed between headwater and middle reaches (CALH and SILRIV) but there were no differences in suspend solid with plain rivers (PLRIV). Large Rivers and Plain Rivers could be considered the same water condition respect to pollutant indicators because no differences were found between pollutant parameters.

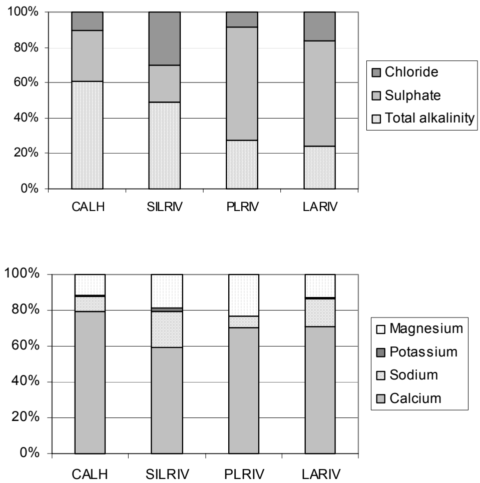

Figure 2: Major ion composition for the four river ecotypes.

Table 1: Mean values and standard deviation of some parameters of unpolluted sites for river ecotypes. Analyses of variance: asterisk indicates significance level (* $p<0.05$; ** $p<0.01$;*** $p<0.001$) and letters mean group forms of the Tukey test ($p<0.05$).

		Calcareous Headwaters			Siliceous Rivers			Plain Rivers			Large Rivers		
parameter	units	n	Mean ± SD		n	Mean ± SD		n	Mean ± SD		n	Mean ± SD	
Nitrate	mgN-NO_3^- l^{-1}	31	3.208 ± 3.877		8	0.938 ± 0.800		17	1.502 ± 1.251		10	0.903 ± 0.422	
Nitrite	mgN-NO_2^- l^{-1}	31	0.008 ± 0.006		8	0.006 ± 0.005		17	0.011 ± 0.007		10	0.014 ± 0.009	
Ammonium**	mgN-NH_4^+ l^{-1}	31	0.046 ± 0.049	a	8	0.019 ± 0.018	a	17	0.082 ± 0.007	b	10	0.090 ± 0.078	b
Phosphate	mgP-PO_4^{-3} l^{-1}	31	0.019 ± 0.018		8	0.031 ± 0.014		17	0.022 ± 0.016		10	0.025 ± 0.012	
Conductivity$_{25^{\circ}}$***	µS cm^{-1}	28	666.6 ± 191.9	a	8	148.0 ± 150.5	b	15	843.1 ± 413.0	a	10	1029.9 ± 532.3	c
COD	mg l^{-1}	14	38.6 ± 34.7		5	21.0 ± 8.4		10	48.0 ± 27.8		5	39.6 ± 20.3	
Dissolved oxygen	mg l^{-1}	24	9.4 ± 1.6		8	9.2 ± 0.9		14	9.8 ± 1.9		10	8.0 ± 1.7	
TSS*	mg l^{-1}	27	5.4 ± 8.5	a	8	4.9 ± 6.2	a	14	7.2 ± 8.0	ab	9	17.2 ± 18.9	b
TOC	mg l^{-1}	21	1.7 ± 1.0		7	2.9 ± 1.7		12	1.5 ± 0.4		6	2.3 ± 2.2	
Turbidity	U.N.F.	27	1.2 ± 1.6		8	28.6 ± 77.0		14	2.4 ± 2.6		10	6.8 ± 14.4	

3.2 Land use and pollutants

All sites, polluted and unpolluted, were taken into account to investigate the relationships between land uses and pollution. The correlation analysis (Spearman) indicated that land uses were associated with certain pollutants, mainly nutrient concentrations. Significant correlations between land use and pollutant parameters, are shown in table 2. For the total study area (CLM), significant positive correlations were found between the percentage of urban area and nitrite, ammonium and phosphate levels. Agriculture was correlated with nitrogen forms and forested land showed a negative correlation with nitrate and phosphate concentrations. In all river ecotypes, agricultural use was correlated with nitrate and nitrite, whereas forest use presented negative correlation with nitrate concentration. The highest correlation coefficient was recorded in calcareous headwaters for nitrate and agriculture ($r = 0.637$, $p < 0.001$). Urban use was correlated to some nutrient in all ecotypes except for calcareous headwaters, where no correlation was found with nutrients. Phosphate and nitrate were positively correlated to urban use only in case of large rivers.

Table 2: Spearman coefficients between land use types and pollutant parameters.

		CALH			SILRIV			PLRIV			LARIV			CLM		
		n	r	p	n	r	p	n	r	p	n	r	p	n	r	p
Urban	NH_4^+		ns			ns		59	0.272	*	26	0.429	*	196	0.306	***
	NO_3^-		ns			ns			ns		28	0.378	*		ns	
	NO_2^-		ns		47	0.289	*	64	0.298	*		ns		207	0.384	***
	PO_4^{3-}		ns			ns			ns		25	0.400	*	185	0.224	**
	TOC		ns		28	0.586	*	38	0.452	**	16	0.758	**	142	0.421	***
	COD	41	0.346	*		ns			ns			ns			ns	
	TSS	72	0.242	*		ns			ns			ns			ns	
	Turbidity	73	0.345	**	32	0.364	*		ns			ns		171	0.302	**
Agriculture	NH_4^+		ns		41	0.351	*		ns			ns		196	0.207	**
	NO_3^-	80	0.637	***	47	0.395	**	77	0.268	*	28	0.442	*	232	0.235	***
	NO_2^-	68	0.474	***	47	0.566	***	64	0.404	**	28	0.540	**	207	0.449	***
	PO_4^{3-}		ns		43	0.289	*		ns		25	0.479	*		ns	
	TOC		ns		28	0.419	*	38	0.405	*	16	0.763	***	142	0.351	***
	COD		ns			ns			ns			ns			ns	
	TSS		ns			ns			ns			ns		169	0.158	*
	Turbidity	73	0.263	*	32	0.554	**		ns			ns		171	0.260	**
Forest	NH_4^+		ns			ns			ns			ns			ns	
	NO_3^-	80	-0.632	***	47	-0.328	*	77	-0.268	*	28	-0.506	*	232	-0.227	**
	NO_2^-	68	-0.469	***	47	-0.599	***	64	-0.395	**	28	-0.547	**		ns	
	PO_4^{3-}		ns		43	-0.419	**		ns		25	-0.505	*	185	-0.170	*
	TOC		ns		28	0.419	*		ns			ns		142	0.192	*
	COD		ns			ns			ns			ns			ns	
	TSS		ns			ns		45	-0.367	*		ns			ns	
	Turbidity		ns			ns		45	-0.387	**		ns			ns	

With respect to other pollution indicators, the correlation and significant were lower than nutrient concentrations, in table 2. Agricultural and urban land use were significantly correlated with TOC in all river ecotypes except calcareous headwaters. On the other hand, the percentage of urban area showed a correlation with COD, TSS and turbidity in the ecotype CALH. Total suspend solid and turbidity presented a negative correlation with forest area only in plain rivers

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(PLRIV). In the case of Castilla-La Mancha, the increase of agricultural and urban land use presented significant correlation with TOC and turbidity.

Table 3: Results of linear regression between land use and nutrient concentrations.

	URBAN	AGRICULTURE	FOREST
NH_4^+	R^2= 0.55 p<0.001	R^2= 0.53 p<0.001	R^2= -0.34 p<0.001
NO_3^-	R^2= 0.34 p<0.001	R^2= 0.62 p<0.001	R^2= -0.6 p<0.001
NO_2^-	R^2= 0.48 p<0.001	R^2= 0.55 p<0.001	R^2= -0.33 p<0.001
PO_4^{3-}	R^2= 0.6 p<0.001	R^2= 0.38 p<0.001	R^2= -0.40 p<0.001

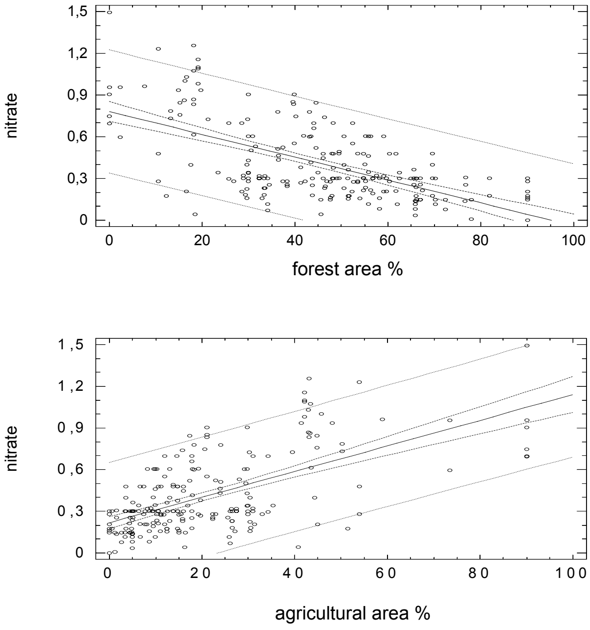

Figure 3: Scatter plots for the concentration of nitrate versus the increase of agricultural and forest percentages. It indicates the regression line, the 95% confidence interval and the 95% prediction interval.

Finally, the regression analyses recorded a strong trend in the percentage of land use type and nutrients levels, in table 3. Concentrations of nitrogen compounds were higher correlated to the increase of land dedicated to agriculture. These results fit in with Spearman correlation for the Region of Castilla-La Mancha, as shown in table 2. For urban use, however, there were significant correlations with phosphate and ammonium levels. A negative significant correlation was obtained between forested area and all nutrients, mainly nitrate concentrations. Figure 3 plots the highest significant correlation values that corresponded to the percentage of agricultural and forested area against nitrate concentrations. Scatter Plots revealed a linear relationship with a positive significant correlation when the percentage of agriculture increases and negative relation with decreasing forested area.

4 Discussion

The water quality of streams can be expected to differ between ecoregions in terms of major ions, nutrients, organic matter and silt loading because it is a product of atmospheric inputs, climatic conditions, land use practices, an specially catchment bedrock [20, 21]. However, Harding *et al.* [22] found little correspondence between stream water chemistry and ecoregions in South Island, New Zealand. In this study, the river ecotypes corresponded to distinct major ion composition and major ion concentration. The calcium-bicarbonate type dominated siliceous and calcareous headwaters (SILRIV and CALH) whereas calcium-sulphate type dominated downstream (PLRIV and LARIV), probably due to the contribution of sulphates by the weathering of sedimentary rocks (Allan [2]). However, Sala [7] concluded that the high sulphates values were a consequence of agriculture as well as industrial and domestic inputs.

The relationships between land use and nutrient concentration in this study produced important conclusions. Urban land use increased phosphorus and nitrogen levels in rivers, agricultural use mainly increased nitrogen compounds, and forested land showed negative correlations with nitrate and phosphate. This negative correlation supports the opinion that forest catchments periodically retain larger amounts of nutrients [23, 24]. In siliceous rivers, agriculture originated ammonium and phosphate pollution due to high densities of livestock (>2500 heads x 1000). At calcareous headwaters (CALH) the highest correlation with nitrate was recorded ($r = 0.632$, $p<0.001$). In this river ecotypes forested land dominates, agricultural land use of territory is bellow 50%, and population is low. Therefore, the absence of large urban areas in calcareous headwaters leads one to attribute the high nitrate values to agricultural activities Bellos *et al.* [6]. In the case of small upland catchment in Indonesia, it appears to have a larger impact due to diffuse pollution associated with land use than the point source impact Walsh *et al.* [25]. Plain and large rivers were located in sedimentary valleys and plateaus with the largest catchment areas and the highest percentage of catchment dedicated to agricultural and urban use, therefore indicating that human impact is greatest in these river ecotypes (PLRIV and LARIV). Ammonium was correlated to urban land use only in plain and large rivers, thus indicating high population densities. Agricultural use was also correlated to nitrite and nitrate in both river ecotypes receiving high nutrient loading by agricultural run-off, industry and domestic sewages. However, phosphate was correlated to agriculture only in the case of large rivers (LARIV) and siliceous rivers (SILRIV), both river ecotypes showing high livestock densities and high agricultural development.

Study results highlight the close association between land use types and nutrient levels in rivers. As, in other study where anthropogenic land uses primarily defined nutrient baseflow chemistry patterns at regional and watershed levels Dow *et al.* [17]. Nitrate has shown a strong relationship with land use types because nitrogen compounds are easily oxidised to nitrate and contribute to nitrate leakage into rivers. The increase of nitrate concentrations was better explained by the increase in agricultural percentage (R^2 = 0.384) and the decrease in forests (R^2 = 0.367), fig. 3. These results reflect that the nitrogen fertilization of cropland produce surpluses which are then emitted to water channels. Extensive areas in Castilla-La Mancha have been affected by groundwater pollution caused by nitrates Berzas *et al.* [26], which also influence nitrate levels in rivers. The European Environment Agency (EEA) found a close relationship between nitrogen load and the surplus of nitrogen applied to agriculture for large European river catchment EEA [27]. On the other hand, the forested area buffers the levels of nutrients providing an effective reduction of nitrate load. The surface runoff of nutrients movement to streams can be reduced by riparian forest Schultz *et al.* [24]. According to EEA [27], discharge of phosphorus from point sources in Europe has decreased significantly during the past 30 years, mainly due to improved purification of urban wastewater. In this study, phosphate was strong related to urban land use, showing the need of tertiary treatment in the sewage treatment plants to reduce the load of nutrient discharge into rivers.

5 Conclusions

Since 1993, crop lands increased 5% but production has doubled in Castilla-LA Mancha, which means an important increase in nutrient fertilization (Regional Agriculture Database). The use and abuse of nitrogen and phosphorus fertilizers have caused high nutrient loads to flow into Central Spanish rivers, with additional serious pollution produced by urban sewage. Adequate management of fertilizers to avoid high surpluses and therefore increase depuration efficiency by sewage treatment plants are absolutely necessary measures to reach the environmental objectives set by the Water Framework Directive. Our results are of relevance for integrated, sustainable management and protection of Castilla-La Mancha freshwater resources and ecosystems.

References

[1] Hynes, H.B.N., *The Ecology of Running Waters*. Liverpool University Press, Liverpool, 1970.

[2] Allan, J.D., *Stream Ecology: Structure and Function of running waters.* Kluwer Academic Publishers, Dordrecht.

[3] Prenda, J. & Gallardo, A., Self-purification, temporal variability and the macroinvertebrates community in small lowland Mediterranean streams receiving crude domestic sewage effluents. *Archiv für Hydrobiologie* **136**, pp.159-270, 1996.

[4] Fytianos, K., Siumka, A., Zachariadis, G.A. & Beltsios, S., Assessment of the quality characteristics of Pinos River, Greece. *Water, Air, and Soil Pollution*, **136**, pp. 317-329, 2002.

[5] Koukal, B., Dominik, J., Vignati, D., Arpagaus, P. Santiago, S., Ouddane, B. & Benaabidate, L., Assessment of water quality and toxicity of polluted Rivers Fez and Sebou in the region of Fez (Morocco). *Environmental Pollution*, **131 (1)**, pp. 163-172, 2004.

[6] Bellos, D., Sawidis, T. & Tsekos, I., Nutrient chemistry of River Pinios (Thessalia, Greece). *Environment International*, **30 (1)**, pp.105-115, 2004.

[7] Sala, M., Hydrogeomorphological assessment of surface and groundwater quality in the Ridaura stream, Catalan Ranges, NE Iberian Peninsula. *Land Degradation and Development*, **15 (3)**, pp. 311-323, 2004.

[8] Dassenakis, M., Scoullos, M., Foufa, E., Krasakopoulou, E., Pavlidou, A. & Kloukiniotou, M., Effects of multiple source pollution on a small Mediterranean river. *Applied Geochemistry*, **13 (2)**, pp. 197-211, 1998.

[9] Petts, G. & Callow, P., (eds). *River restoration*. Blackwell Science: Oxford, 1996.

[10] Domínguez, A., *Tratado de fertilización*. Mundi-Prensa: Madrid, 1997.

[11] Moreno, J.L., Navarro, C. & De Las Heras, J., Abiotic ecotypes in south-central Spanish rivers: Reference conditions and pollution, *Environmental Pollution*, **143**, pp. 388-396, 2006.

[12] Moreno, J.L., Navarro, C. & De Las Heras, J., Propuesta de un índice de vegetación acuática (IVAM) para la evaluación del estado trófico de los ríos de Castilla-La Mancha: Comparación con otros índices bióticos, *Limnetica*, **25 (3)**, pp. 821-838, 2006.

[13] Moreno, J.L., Navarro, C. & De Las Heras, J., Propuesta de un índice de vegetación acuática (IVAM para la evaluación rápida del estado trófico de los ríos de Castilla-La Mancha: comparación con otros índices bióticos, *Tecnología del Agua*, **143**, pp. 388-396, 2006.

[14] Ometo, J.P.H.B., Martinelli, L.A., Ballesteri, M.A., Gessner, A., Krusche, A.V., Victoria, R.L. & Williams, M., Effects of land use on water chemistry and macroinvertebrates in two streams of the Piracicaba river basin, south-east Brazil. *Freshwater Biology*, **44**, pp. 327-337, 2000.

[15] Donner, S., The impact of cropland cover on river nutrient levels in the Mississippi River Basin, *Global Ecology and Biogeography*, **12 (4)**, pp. 341-355, 2003.

[16] Schilling, K.E. & Spooner, J., Effect of watershed-scale land use change on stream nitrate concentration, *Journal of Environmental Quality*, 35 **(6)**, pp. 2132-2145, 2006.

[17] Dow, C.L., Arscott, D.B., Newbold, J.D., Relating major ions and nutrients to watershed conditions across a mixed-use, water-supply watershed, *Journal of the North American Benthological Society*, **25 (4)**, pp. 887-911, 2006.

[18] APHA, *Standard methods for the examination of water and wastewater*. American Public Health Association, Washington D.C., 1989.

[19] Bonada, N, N. Prat, A. Munné, M. Rieradevall, J. Alba-Tercedor, M. Alvarez, J. Avilés, J. Casas, P. Jáimez-Cuéllar, A. Mellado, G. Moya, I. Pardo, S. Robles, G. Ramón, M.L. Suárez, M. Toro, R. Vidal-Abarca, S. Vivas y C. Zamora-Muñoz, Criterios para la selección de condiciones de referencia en los ríos mediterráneos. Resultados del proyecto GUADALMED. *Limnetica*, **21 (3-4)**, pp. 99-114, 2002.

[20] Stumm, W. & Morgan, J.J., *Aquatic chemistry*. John Wiley and Son, New York, 1981.

[21] Gibbs, R.J., Mechanisms controlling world water chemistry. *Science*, **170**, pp. 1088-1090, 1970.

[22] Harding, J.S., Witerbourn, M.J. & Wayne, F.M., Stream faunas and ecoregions in South Island, New Zealand: do they correspond? *Archiv für Hydrobiologie*, **140 (3)**, pp.289-307, 1997.

[23] May, L., House, W.A., Bowes, M. & McEvoy, J., Seasonal export of phosphorus from a lowland catchment: upper River Cherwell in Oxfordshire, England *Science of Total Environment*, **269**, pp. 117-130, 2001.

[24] Schultz, R.C., Isenhart, T.M., Simpking, W.W. & Colletti, J.P., Riparian zones forest buffers in agrosystems- lessons learned from the Bear Creek Watershed, central Iowa, USA *Agroforestry Systems*, **61 (1)**, pp. 35-40, 2004

[25] Walsh, C.J., Gooderham, J.P.R., Grace, M.R., Sdraulig, S., Rosyidi, M.I. & Lelono, A., The relative influence of diffuse- and point-sources disturbances on a small upland stream in East Java, Indonesia: a preliminary investigation *Hydrobiologia*, **487**, pp. 183-192, 2002.

[26] Berzas, J.J., García, L.F., Martín-Álvarez, P.J., & Rodríguez, R.C., Quality assessment and chemometric evaluation of a fluvio-lacustrine system: Ruidera Pools Natural Park (Spain). *Water, Air and Soil Pollution*, **155**, pp. 269-289, 2004.

[27] EEA, *Source apportionment of nitrogen and phosphorus inputs into the aquatic environment*, Report No 7/2005. European Environment Agency, Copenhagen, 2005.

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Health impacts of the 2005 flood events on feedlot farm families in southern Alberta, Canada

M. P. Acharya[1], R. G. Kalischuk[1], K. K. Klein[1] & H. Bjornlund[1,2]
[1]*Department of Economics and School of Health Sciences, University of Lethbridge, Canada*
[2]*University of South Australia, Australia*

Abstract

In the summer of 2005, southern Alberta received heavy rainfall that led to extensive flooding. Four separate flooding events severely affected several rural communities, roads, bridges, and businesses within the region; specifically, the flooding affected farm families living on Canada's largest feedlot operations in the area. This article explores the health-related impacts of the 2005 flood events on families who reside on feedlot farms in southern Alberta. Utilizing random sampling, an in-person survey was conducted between September and December 2005 with 33 affected feedlot farm families living in the Lethbridge Northern Irrigation District, home of the greatest number of large beef feedlot operations in Canada. Three percent of participants (and 12% of their family members) reported flood-related health problems. However, 63% of participants (and 58% of their family members) reported flood-related mental health problems, and 40% (and 24% of family members) indicated that they experienced isolation and helplessness. Only 9% of the participants accessed health services. Most participants reported that their communities were more helpful in dealing with their flood-related health problems than were public and service delivery sectors. A majority (91%) of participants reported that their family was helpful, followed by friends (64%), neighbors (42%), church (39%), and employees (36%). Among the affected feedlot farm families, a small percentage experienced physical health problems but a much larger percentage had mental health problems. However, only a few utilized health care services. This information will be important to health care leaders and policy makers as they plan and develop efficient and effective health care support for farm families exposed to flooding events.
Keywords: 2005 flood events, feedlot farm families, health impacts, southern Alberta, Canada, health care services, mental health problems, isolation and helplessness.

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1 Introduction

Southern Alberta in western Canada, a semi-arid region with high summer temperatures and low rainfall [1, 2], in 2005 experienced the worst and costliest natural disaster in the history of that province [3–6]. Three major floods in the month of June and further torrential rainfall from September 9 to 11 produced 534 mm of rain by the end of September [4, 7, 8]. In terms of precipitation and flood-related human fatalities, this region's 2005 flooding is not comparable to the flood disasters that took place in western and southern India (for example, in the city of Mumbai alone there was 944 mm of rainfall in 24 hours which affected 2 million people with at least 200 fatalities) or southern China (heavy rainstorms during the third week of June killing 170 people) in the summer of 2005 [6]. Nonetheless, heavy rain storms in southern Alberta kept thousands of people out of their homes [4], on evacuation alert and left others without electricity for at least five days, particularly during the early June flood [8]. Also, there were major destructions to the region's infrastructure and services. For example, several bridges and roads were damaged; in many towns sewage and water supply systems struggled to cope with the deluge of rain; small businesses were destroyed; and agricultural activities, specifically farm families living on Canada's largest feedlot operations within the region were either flooded or cut off from their neighbors. According to reports, the June 9th rainfall and flooding alone caused over C$100 million in damages [8] in southern Alberta, prompting several municipalities to declare states of emergency along the Oldman River, including municipality district (MD) of Pincher Creek, MD of Willow Creek, County of Lethbridge and City of Lethbridge [3, 4]. In addition to issuing states of emergency, flood warnings and flood watches, on June 9, 2005 the Government of Alberta through Alberta Municipal Affairs implemented a Disaster Recovery Program (DRP) called the 2005 Southern Alberta Disaster Recovery Program (SADRP). SADRP, the largest program of its kind in the province's history [7], was introduced to help residents of this region recover from losses caused from severe rain storms [5, 9]. To date, C$73.1 million has been paid with an estimated total program cost of C$162.7 million once final municipal infrastructure and emergency operations costs are received from southern Alberta [7, p. 1]. Based on the above reports, officials have described the 2005 flooding as a "one-in-200 year event" [8].

In addition to the destruction of critical infrastructures, flood-related natural disasters have adverse effects on human health [10–16], particularly among survivors [17] and displaced persons' health [18]. The most common health problems reported among flood victims are: trauma caused by lives lost mainly from drowning [19], increased incidence of infectious diseases, especially acute gastrointestinal infections because of faeco-oral cycling from disruption of sewage disposal [10, 20] or untreated sewage disposal [17]; vector-borne and rodent-borne infections, such as malaria, yellow fever, dengue fever, West Nile fever, Hantavirus and leptospirosis [14, 15, 19, 21]; wound infections or injuries [10, 22]; and mental health problems such as anxiety, depression, sleeplessness and post-traumatic stress [18, 21, 23]. In recent years, several studies have been conducted to review health impacts of flooding worldwide [10], however, most have focused on developing countries [10, 11] and on flooding caused after hurricanes or tropical cyclones [12, 14, 17, 18]. The limited literature obtainable on developed countries is primarily from the United States [12, 17], Europe [18, 20] and Australia; and there is a deficiency in literature that documents flood-related health issues from rural Canada, especially semi-arid southern Alberta.

In Canada, the few studies available on flood events show that the incidence of waterborne infection is highest in rural areas with intensive agricultural production and especially intensive livestock operations during periods of extreme precipitation [2, 22, 24]. In addition to high incidence level of waterborne infection in rural watersheds [14], recent health reports on Canadians suggests that mental health concerns of farm families [20] and the health status of people who live in rural and remote communities is poorer [25] than the rest of the Canadian population [21, 26], and many have limited access to health care services [27]. However, much of these health studies of Canadians have not included health impacts of flooding. It is well known that individuals living in high or middle income countries who experience natural disasters develop excessive physical, mental and emotional stress [13], especially anxiety and depression [10]. This article explores the health-related impacts of the 2005 flood events on families who reside on large feedlot farms in southern Alberta and also describes the responses to recent flood-related health problems by the feedlot farm families. The specific objectives of this article are to:

- determine the types of health problems experienced by the feedlot farm families living in Lethbridge Northern Irrigation District (LNID);
- examine LNID feedlot farm families' responses to these health problems; and
- identify the most and least helpful types of resources of the participants for coping with flood-related health problems.

2 Research location and methods

The purpose of this research was to gain a deeper understanding of the impact of 2005 flood and what resources were most and least helpful to cope with flood-related health challenges of affected feedlot farm families residing in LNID located in southern Alberta. This research was conducted one month after the September flood and three months after the June flood events. A questionnaire was developed to elicit data with regard to the 2005 flood experiences. For example, the health challenges experienced by the feedlot farm families, their responses associated with these particular flood events, and resources most and least helpful for dealing with flood-related health challenges. This research was approved by the University of Lethbridge Faculty of Arts and Science Ethics Committee. All participants signed informed process consents and confidentiality was maintained.

LNID is located in the southwestern part of the province of Alberta, between the city of Calgary and Lethbridge, and provides water from the Oldman River to over 50,000 ha through irrigation canals. The irrigation district's responsibility relates directly to water quantity management through their delivery of water to irrigators and those with whom the district has agreements to supply water. The LNID is home to about 30 of the largest feedlot operations of Canada [28].

Using an interviewer-administered questionnaire, data were collected from a random sample of 33 feedlot owners and/or employees (females n=8; males n=25) between the ages of 27 and 70. Feedlot families' contact information was obtained from the LNID office. Most, if not all participants in this research reported living within the LNID during the 2005 flood events. Interviews were conducted from October to December 2005 in the participant's homes lasting 1-2 hours. Finally, all interviews were audio-taped and transcribed verbatim.

3 Results and discussion

3.1 Survey participants

Sixty feedlot households were contacted, of which 55% (33/60) consented to participate and were recruited. In this study most participants were male (76%) and the median age was in the 40-59 years of age group (Table 1). There was a considerable variation in educational level. The majority of the participants had more than high school education (73%), were married (91%) at the time of interview and more than 58% said that their household had four or more members. At the time of the 2005 flood, almost all participants (94%) owned their farm and 79% of them were engaged primarily in feedlot activities with 67% managing more than 10,000 beef cattle per year.

Table 1: Background Information of the Participants (N=33).

Characteristics	N	%	Characteristics	N	%
Gender			Education		
Male	25	76	High School or <	9	27
Female	8	24	Post-High School	24	73
Age			Main Agricultural Activity		
39 years and <	8	24	Feedlot	26	79
40 to 59 years	21	64	Field-crop	4	12
60 years and >	4	12	Other(dairy & swine)	2	6
Marital status			Farming Status		
Married	30	91	Owner	31	94
Other	3	9	Employee	2	6
# of People live at home			Feed Animals per year		
Three or <	14	42	Less than 10,000	11	33
Four or >	19	58	10,000 – 20,000	14	43
			21,000 – 30,000	5	15
			31,000 – 70,000	3	9

3.2 Flood-related feedlot operation problems

To assess the impact of flooding on feedlot operations, participants were asked to describe the types of problems they experienced on their feedlot during heavy rainfalls. Almost all participants admitted that the 2005 flood events severely damaged their feedlot operation. For example, all talked about how the flooding caused damages to their pens, caused their lagoons to overflow (91%) and blocked effluent drainage systems (88%). In fact, persistent rainfall made it difficult for most to access and clean pens (88%). In addition to structural damages to pens and lagoons, many talked about loss in animal performance because of flood-related illnesses. Loss of grain, crop damage, loss of bedding supplies, labor shortage and damages to storage and office space were also reported by participants. In fact, the majority of participants talked about their feedlot damages in terms of financial loss. This is because it is the main source of household income for every individual in this study. Moreover, every participant concluded that they will never be able to know their financial loss, and many suggested that they did not want to experience another natural disaster after their experience with BSE crisis.

Table 2: Percentage and types of flood-related problems participants experienced in their feedlot operation during the 2005 flood events.

Types of Problems Experienced	Respondents % Indicated
Wastewater & Flood Runoff Management	
-Pen Problems (i.e. overflow, messy, damaged, etc.)	97
-Lagoon Overflow	91
-Access	88
-Equipment problems (i.e. destroyed from the flood, not available or accessible)	88
-Drainage Problem	88
-Lagoon Full	79
Animal Health, Illness & Death	
-Animal Illness (primarily foot rot)	97
-Animal Performance	97
-Loss of Animal	67
Animal Feed & Bedding Supplies	
-Crop Damage (i.e. forages)	94
-Animal Feed loss	85
-Grain loss	79
Labour Issues	70
Damages to Storage & Office Space	94

3.3 Health challenges

Previous studies provide detailed accounts of the impact of floods on human health from waterborne diseases to mortality [10, 13, 15, 17, 19, 21]. Table 3 displays the types of health challenges experienced by the participants and their family members. While flood-related diseases were reported by 3% of participants and 12% of family members, flood-related mental health problems were reported by 63% of participants and 58% of family members. The most common mental problem by both the participants and family members are: stress (85%), anxiety (85%), anger (67% and 64%), and helplessness (73% and 58%). Only a minority (6%) of participant believed that they alone experienced depression. Social and behavioral problems, including isolation and helplessness, were experienced by 40% of participants and 24% of family members.

Though the majority of participants and their family members experienced mental health concerns, only 9% of participants accessed health services. Family physicians, counselors, and psychologists were most commonly sought in addressing theses concerns. Counseling services and medical treatments were utilized most frequently. No differences in health service utilization pattern were observed based on age, education or farm size. However, the findings somewhat supports earlier data on rural Canadians underutilization of health services as a group [25, 27, 29, 30]. This may have some adverse long term health consequences on the flood-affected individuals.

Table 3: Percentage and types of health challenges experienced by the participants and their family members during the 2005 flood.

Flood-Related Health Challenges	Participants	Family Members
Experienced	% Affected	% Affected
Medical Problems		
Disease	3	12
Physical Problems		
-Tired	73	42
-Sore back	64	36
Mental/Emotional		
-Stress	85	85
-Anxiety	85	85
-Anger	67	64
-Depression	6	-
-Helplessness	73	58
Social/Behavioral		
-Isolation	18	15
-Sleeplessness	61	33

3.4 Behavioral response

The cultural characteristics of rural and remote communities influence the experience of natural disaster management. Similar to previous studies concerning the health of rural Canadians, this study explored what resources the participants found to be most and least helpful in dealing with their health issues related to the 2005 flood events. Most participants (91%) reported that the family or a family member was a helpful source of support in dealing with flood-related health challenges, followed by friends (64%), neighbors (42%), church (39%)

and employees (36%). In other words, the participants found that their communities were more helpful in assisting participants to deal with their flood-related health problems than public and service delivery groups.

Resources that were noted as least helpful included Natural Resource Control Board (36%), a regulatory body that is responsible for the management of Intensive Livestock Operations in the province. Similarly, 33% did not find LNID very useful during the recent flooding, followed by the County of Lethbridge (30%) and media (21%).

Table 4: Resources most and least helpful to the participants while dealing with 2005 flood-related health issues.

Type of Resources	Most Helpful	Least Helpful
Community Categories	(%)	(%)
Family	91	3
Friends	64	0
Neighbors	42	27
Staff/Employees	36	15
Church	39	0
Other Resources – i.e. Sports, Casino, etc.	21	6
Public & Service Delivery Group		
Public[1]	3	27
Industry[2]	42	6
Insurance &telephone Companies	21	0
Natural Resource Control Board (NRCB)	12	36
Lethbridge Northern Irrigation District (LNID)	18	33
County of Lethbridge (CL)	24	30
All levels of Government	3	21
Researcher	6	0
Information Technology		
Media (i.e. television, newspaper, internet & radio)	6	21
[1]Public = people who do not live on farms; [2]Industry = Livestock and Cattle Producers.		

4 Conclusion

This research is the first to investigate the types of health challenges faced by feedlot farm families during the 2005 flood in southern Alberta. The results of this research are highly relevant for future research that addresses the public health impacts following flooding in livestock intensive areas. For instance, although the participants lived in an intensive livestock community, they reported relatively low levels of immediate, short- or long-term infectious disease effects or occurrence of clinically relevant pathology. Instead, the majority of participants experienced some form of mental/emotional health problems. Furthermore, the results demonstrate that the participants and their family members found their community to be most helpful, and the service delivery sector and the mass media the least helpful resources. This may be due to the duration and severity of flood-related health problems, access to service facilities, or the stigma associated with mental health challenges. Further research is needed to identify factors associated with under-utilization of service delivery. One limitation associated with the present study is that the findings are based on only one livestock sector and only one irrigation district in southern Alberta. It would be interesting to know if the findings would be similar in other livestock sectors, irrigation districts and regions within the province of Alberta. Additional studies are planned to investigate these issues.

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References

[1] Government of Alberta. Alberta, www.en.wikipedia.org/wiki/Alberta, 2006.

[2] Hyland, R., J. Byrne, Selinger, B., Graham, T., Thomas, J., Townshend, I. & Gannon, V. Spatial and temporal distribution of faecal indicator bacteria in southern Alberta, Canada. *Water Quality Research Journal of Canada,* 38, pp. 15-32, 2003.

[3] Alberta Environment. June 2005 Flood Events in Albert. *Presentation to the Alberta Environment Conference*, 2006.

[4] Carpenter, G., *Floods in Alberta,* Canada. Report Date, 30th June 2005, www.guycarp.com

[5] Government of Alberta. Disaster Recovery Programs are available on the Municipal Affairs, www.municipalaffairs.gov.ab.ca, 2006.

[6] World Meteorological Organization (WMO). *WMO Statement on the Status of the Global Climate in 2005*, World Meteorological Organization: Geneva, Switzerland, 2005.

[7] Alberta Municipal Affairs. *Flooding in Alberta in 2005 – An Overview*. Alberta Municipal Affairs: Edmonton, Alberta, 2006.

[8] Canadian Broadcasting Corporation (CBC). 2005 sets rain and flood records. *CBC News, www.cbc.ca/canada/story/2005/12/29/weather-records051229.html,* 2005.

[9] Alberta Agriculture, Food and Rural Development. *2005 Alberta Disaster Recovery Program – Flooding Component*, Government of Alberta: Edmonton, Alberta, 2005.
[10] Ahern, M., Kovats, R. S., Wilkinson, P., Roger, F. & Matthies, F., Global health impacts of floods: Epidemiologic evidence. *Epidemiologic Reviews,* 27, pp. 36-46, 2005.
[11] Bissell, R., Delayed-impact infectious disease after a natural disaster. *The Journal of Emergency Medicine*, 1, pp. 59-66, 1983.
[12] Diaz, J. H., The public health impact of hurricanes and major flooding. *The Journal of the Louisiana State Medical Society*, 156(3), pp. 145-50, 2004.
[13] Greenough, G., McGeehin, M., Bernard, S. M., Trtanj, J., Riad, J. & Engelberg, D., The potential impacts of climate variability and change on health impacts of extreme weather events in the United States. *Environmental Health Perspectives*, 109(2), pp. 191-198, 2001.
[14] Gubler, D. J., Reiter, P., Ebi, K. L., Yap, W., Nasci, R. & Patz, J. A., Climate variability and change in the United States: Potential impacts on vector and rodent-borne diseases. *Environmental Health Perspectives*, 109(2), pp. 223-233, 2001.
[15] Ivers, L. C. & Ryan, E. T., Infectious diseases of severe weather-related and flood-related natural disasters. *Current Opinion in Infectious Diseases,* 19*,* pp. 408-414, 2006.
[16] Rose, J. B., Epstein, P. R., Lipp, E. K., Sherman, B. H., Bernard, S. M. & Patz, J. A., Climate variability and change in the United States: Potential impacts on water and foodborne diseases caused by microbiologic agents. *Environmental Health* Perspectives, 109(2), pp. 211-221, 2001.
[17] Reacher, M., McKenzie, K., Lane, C., Nichois, T., Kedge, I., Iversen, A., Hepple, P., Walter, T., Laxtn, C. & Simpson, J., Health impacts of flooding in Lewes: A comparison of reported gastrointestinal and other illness and mental health in flooded and non-flooded households, *Communicable Disease and Public Health*, 7(1), pp. 39-46, 2004.
[18] Keene, E. P., Phenomenological study of the North Dakota flood experience and its impact on survivors' health. *International Journal of Trauma Nursing*, 4(3), pp. 79-84, 1998.
[19] Vasconcelos, P., Flooding in Europe: A brief review of the health risks. *Eurosuveillance*, 11(4), www.eurosurveillance.org/ew/2006/060420.asp, 2006.
[20] Schmidc, C. W. Lessons from the flood: Will Floyd change livestock farming? *Environmental Health Perspectives*, 108(2), pp. A74-A77, 2000.
[21] Ohi, A. & Tapsell, S., Flooding and human health: The dangers posed are not always obvious. *British Medical Journal*, 321, pp. 1167, 2000.
[22] Health Canada. Notifiable diseases annual summary. *Canada Communicable Disease Report,* 27, pp. 2765, 2001.
[23] World Health Organization and Unicef. *Water for Life: Making it Happen*, *WHO*/UNICEF Joint Monitoring Programme for Water Supply and Sanitation, France, 2005.

[24] Gannon, V. P. J., Graham, T. A., Read, S., Ziebell, K., Muckle, A., Mori, J., Thomas, J., Selinger, B., Townshend, I. & Byrne, J. Bacterial pathogens in Southern Alberta, Canada. *Journal of Toxicology and Environmental Health Part A,* 67, pp. 1643-1653, 2004.
[25] Leiper, B. D., Rural women's health issues in Canada: An overview and implications for policy and research. *Canadian Women's Studies, 24*(4), pp. 109-121, 2005.
[26] DesMeules, M., Pong, R., Lagacé, C., g, D., Manuel, D., Pitblado, R., Bollman, R., Guernsey, J., Kazanjian, A. & Koren, I. 2006. *How healthy are rural Canadians? Canadian population health initiative*, Canadian Institute for Human Information, Ottawa, 2006.
[27] Romanow, R., Building on values: The future of health care in Canada, Ottawa: The Romanow Commission Report, 2002.
[28] North American Environmental Law and Policy (NAELP). *Comparative Standards* for *Intensive Livestock Operations in Canada, Mexico and the United States*, Ếditions Yvon Blais: Montreal, 2003.
[29] Thomlinson, E., McDonagh, M. K., Crooks, K. B. & Lees, M., Health beliefs of rural Canadians: Implications for practice. *Australian Journal on Rural Health*, 12, pp. 258-263, 2004.
[30] Thurston, W., E., Glundell-Gosselin, H. J. & Pose, S., Stress in male and female farmers: An ecological rather than an individual problem. *Canadian Journal of Rural Medicine*, 8(4), pp. 247-254, 2003.

Groundwater quality and sustainability in granitised-fractured aquifers, Pallisa district, eastern Uganda

J. Nyende
Water Engineering Section, Department of Civil and Building Engineering, Kyambogo University, Kampala, Uganda

Abstract

The sustainability of the granitised aquifer system of the Kyoga catchment has been threatened by the ever increasing population that will lead to over exploitation of the groundwater resource. Uganda's freshwater is considered a key strategic resource which is vital for sustaining life, promoting development and maintaining the environment. The increasing pressure from domestic, industrialization and urbanization, agricultural as food security and hydropower uses due to population and industrial growth combined with environmental degradation and extreme poverty are relatively putting pressure on the quantity and quality of water resources. Groundwater quality in Uganda is originally influenced by the rainwater in some areas and the Nile, which recharge the aquifers from where boreholes are drilled and dug up wells. Boreholes data show the highest degree of mineralisation with relative enrichment of nearly all tested elements and is due to the ability of aggressive groundwater to decompose the relatively fresh minerals in the bedrock fractures. Plots of boreholes situated in these aquifers of Pallisa represent a regime where there is calcium enrichment, which is typical of lime dosing to neutralize acid waters. The plot of water contains approximately 70% Ca, 70% Mg, 80% TAlk, 10% Cl and 10% SO_4. This is typical of calcium/magnesium bicarbonate waters. Higher concentrations of aluminium are observed in the regolith while low levels in the bedrock. The significantly lower level of aluminium in the bedrock aquifer provide further support for the notion of relatively limited hydraulic interaction between the two units. An urgent need for a holistic approach to water resources management is required to protect the available water resources and to satisfy the sometimes-conflicting demands while ensuring sustainable water resources development. Collection and analysis of water resources related data and information is one of the priority areas for the water resources management sub-sector.
Keywords: groundwater quality, sustainability, rock interaction, fractured-bedrock, wastewater, basement complex, Eastern Uganda, degree of mineralisation, sodium absorption ratio, regolith, granitised.

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1 Introduction

The populations of most development countries are growing at a high rate. It is likely that the total use of groundwater will increase even when conservation measures are taken. Groundwater therefore, has become increasingly popular in Pallisa district. Water being a finite resource, and with the increasing population in the district coupled with deficiency of safe water points, there is a need to identify more safe sources of supply of this limited resource. Therefore, due to high demand for the invaluable resource, work on providing adequate good quality water is essential. However, despite the widespread increase in development of groundwater resources, there has been a population increase that has caused an increase in several activities – agricultural irrigation, livestock and domestic water supply within Pallisa district.

The challenge facing planners, implementers and policy makers in Uganda now, is to ensure sustainable groundwater exploitation, utilisation, prevention of exhaustive abstraction and groundwater-related pollution. It should be realised that the priority in groundwater is directly emphasised on the sustainability of the resources with respect to both quantity and quality. Knowledge of the spatial and temporal characteristics of groundwater systems and their interactions with the environment provides the basis for such sustainable development and environmentally sound planning and management of groundwater resources.

The study on groundwater chemistry in the Naivasha area explained that the quality of groundwater has deteriorated due to high level of nitrate from agricultural activities as well as high-level fluoride (Morgan [1]). The Rural Water and Sanitation Project (RUWASA) and Small Towns Water and Sanitation Project under Directorate of Water Development (DWD) set up water supply projects that were intended to bring safe water to millions of Ugandans in the rural communities by use of groundwater. The focus however, was on delivery (construction of water systems). This has been at the expense of equity and sustainability.

2 Site description

Pallisa district is situated in the eastern part of Uganda and it is neighboured by Iganga district to the south-west, Kamuli district to the west, Tororo district to the south, Soroti district to the northwest, Kumi district to the north and Mbale district to the east. It occupies an area of 1956 km^2 with a population of 357656 according to the 1991 census. The density of the area is 229 people per square kilometer. It is located between latitude 33° 25″ East and 34° 09″ East and Longitude 0° 50″ North and 1° 25″ North. The District Headquarters are located at Pallisa, with major towns of Budaka, Kamuge, Kibuku and Butebo. A location map of the area under investigation is as in Figure 1 below.

2.1 Geology

The geology of Pallisa is generalised and can be described as undifferentiated gneisses including elements of partly granitised and metamorphosed formations.

It consists of however, a gneissic complex formation. Gneiss and granitic formations of the Pre-Cambrian predominates which are usually referred to as gneiss complex. The northern part is largely underlain by older, wholly granitised or medium to high-grade metamorphic formations. However, thin pleistocene deposits are the most widespread representatives of the post-Cambrian deposition. Volcanic activity occurred during the Lower Miocene in eastern Uganda (Elgon, Moroto, Kadam and Napak), which in some areas are underlain by sediments possibly of cretaceous age, and during the late pleistocene over a small area.

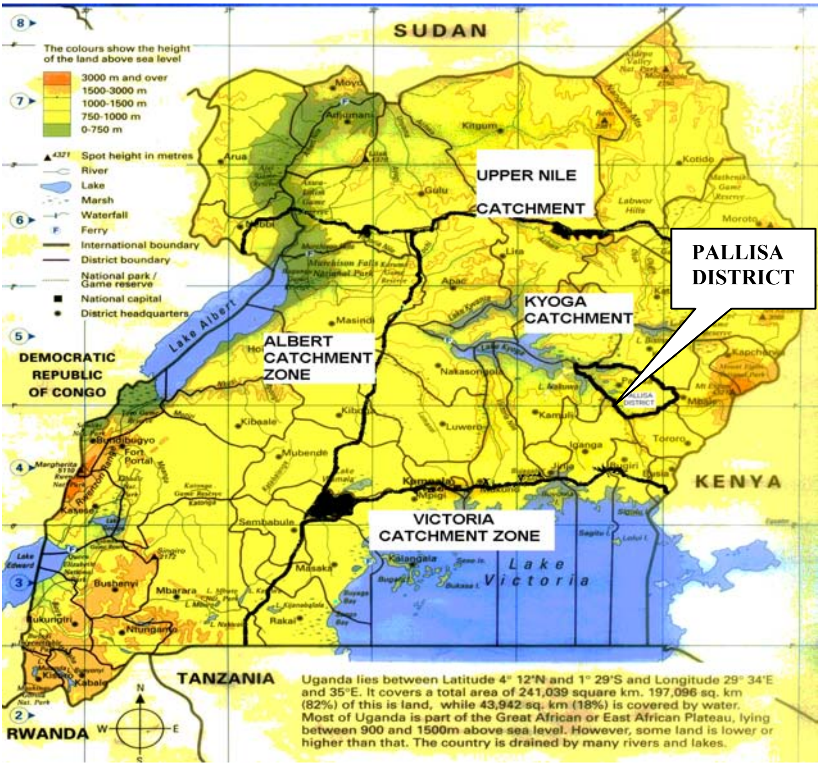

Figure 1: Location of the study area within Uganda; Source: NCDC, Uganda [2].

3 Hydrogeology

In much of this study area, groundwater is widely available and fairly free from sediment and biological impurities that frequently plague surface waters. Considering the rural population of Pallisa district, which relies exclusively on groundwater as the only potable water source, many thousands of boreholes which have been put into production were constructed in 1930s. Until very recently, the preferred method of well construction has been to drill relatively deep wells that fully penetrate the overlying regolith, or "weathered zone" and rely on fractures in the competent underlying rock to provide an adequate well yield.

Throughout Pallisa district, crystalline basement rocks are extensively concealed by the regolith, which is the result of intense chemical weathering. The extent of the chemical weathering, and hence development of the regolith, depends on the nature of the basement rock including its age, structure and lithology, as well as climate and relief. Key [3], found out that chemical weathering is enhanced by joints, fractures and coarse grains in the bedrock that expose a greater surface area to groundwater, which is the principal weathering agent. According to Briggs et al [4], the high rainfall and temperature of tropical climates serve to increase the rate at which chemical weathering processes occur as a result of hydrolysis, oxidation and dissolution. Taylor et al [5] described the deeply weathered crystalline rocks that form important aquifers for public water supply throughout low-latitude regions of Africa, South America, and Asia, and that these aquifers have considerable heterogeneity and produce low well yields.

There are two units that form an integrated aquifer system in which transmissivity lies (5-20 m^2/d) and porous weathered mantle that provides storage to underlying bedrock fractures (T = 1 m^2/d). The thickness and extent of the more productive weathered-mantle aquifer are functions of contemporary geomorphic processes. Generally, a lateritic, sub-humus soil horizon rests over a clay unit formed from secondary weathering of basement rock fragments. The water table tends to lie at the base of the clay where fragments of the parent bedrock pre-dominate the bedrock surface. As an indicator in the evaluation process, the water resource is monitored, managed and exploited in a sustainable and equitable manner (Nyende and Hodgson [6]).

3.1 Groundwater chemistry

Hydro-chemical evaluation of groundwater from the regolith and basement aquifers were undertaken in Pallisa district and the groundwater samples analysed for minor and major elements. This was to enable the quality of groundwater presently being pumped from the fractured bedrock aquifer to be compared to the quality of water, as per the WHO [7] guidelines that could be expected from the development of the regolith aquifer. See figure 1 below.

In terms of organic chemistry, the conductivity values (and thus degree of mineralisation) are significantly lower for springs than for dug wells and lower in boreholes. This is explained by the fact that spring water trajectories have been intensively leached through many years of flow whereby the water in these aquifers shows relatively manor enrichment in organic substances. Spring water show the lowest PH values, which demonstrates that water, in spite of the low PH, has caused limited decomposition of aquifer minerals. Boreholes on the other hand, show the highest degree of mineralisation with relative enrichment of nearly all tested elements. This is due to the ability of aggressive groundwater to decompose the relatively fresh minerals in the bedrock fractures.

Table 1: Average values for water quality parameters from dug wells, protected springs and boreholes in Pallisa district.

PARAMETER	AV. DUG WELLS (Nos. =24)	AV. PROTECTED SPRINGS (Nos. =28)	AV. BOREHOLES (Nos. = 38)	WHO 1984 GUIDELINES
PH	6.8	5.8	6.7	6.5-8.5
Conductivity, µS/cm	390	113	737	-
Total iron, mg/l Fe^{2+} + Fe^{3+}	**0.5**	0.3	**0.8**	0.3
Manganese, mg/l Mn^{2+}	0.10	0.05	0.07	0.1
Alkalinity, mg/l $CaCo_3$	119.5	38.8	186.1	-
T-Hardness, mg/l $CaCo_3$	124.6	48.4	229.8	500
Calcium, mg/l Ca^{2+}	41.0	20.7	94.1	-
Magnesiun, mg/l Mg^{2+}	9.1	5.2	22.1	-
Bicarbonate, mg/l HCO_3	140.5	51.1	216.3	-
Carbon dioxide, mg/l CO_3	97.5	126.5	165.6	-
Sodium, mg/l Na^+	25.3	6.4	60.6	200
Potassium, mg/l K^+	5.6	1.6	5.1	-
Chloride, mg/l Cl-	22.1	9.3	73.4	250
Sulphate, mg/l SO_4^{2-}	27.8	12.8	57.5	400
Phosphate, mg/l PO_4^{2-}	1.3	0.4	0.8	-
Nitrate, mg/l NO_3^-	2.2	2.6	3.7	10
Fluoride, mg/l F^-	0.7	0.14	0.8	1.5
TAlk, mg/l	186	173	211	-
% Water points with E.Coli count > 0	34	13	5	0
Aluminum, mg/l	**0.46**	**0.35**	**0.5**	0.2
Turbidity, NTU	0.8	0.15	0.39	5
TDS, mg/l	460	350	375	1000

The figures in bold, table 1 indicate the values not in accordance with WHO guidelines.

3.2 Groundwater quality

The groundwater quality in the catchment was assessed by electrical conductivity (EC) and sodium absorption ratio (SAR), see figures 3 and 4 and table 2 below.

Table 2: Water quality in Pallisa district.

Salinity Hazard EC (μ S/cm)			Sodium Hazard SAR	
C1	< 25	L	S1 < 10	L
C2	25 – 75	M	S1 < 10	L
C3	75 – 225	H	S1 < 10	L
C4	> 255	VH	S1 < 10	L

L – Low; M – Medium; H – High; VH – Very High

Figures 2, 3, 4, 6, 7 and 8 show the chemistry composition of some of the drilled boreholes in Pallisa district. The water quality in almost all the boreholes in Pallisa district is relatively safe for drinking purposes.

Figure 4 shows the salinisation of the boreholes shown is generally very low, but the sampling points on the right of the diagram has a very high electrical conductivity as one moves away from River Mpologoma. (See the position of borehole CD2641, Fig.5.) The decrease in salinity is due to dilution along or near the Mpologoma River.

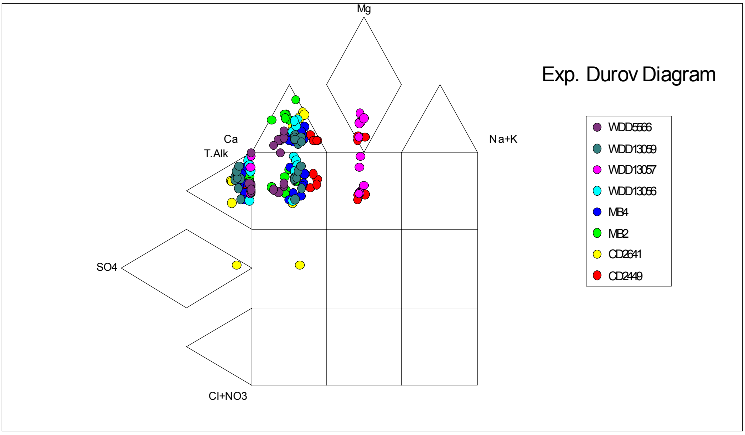

Figure 2: Concentrations of different ions in the expanded durov of all the boreholes drilled in Pallisa.

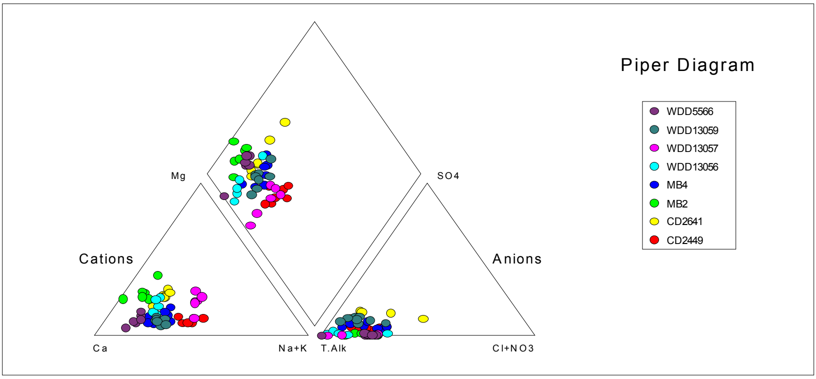

Figure 3: Piper diagram of groundwater from the boreholes in Pallisa, lying in both the regolith and the basement complex (bedrock).

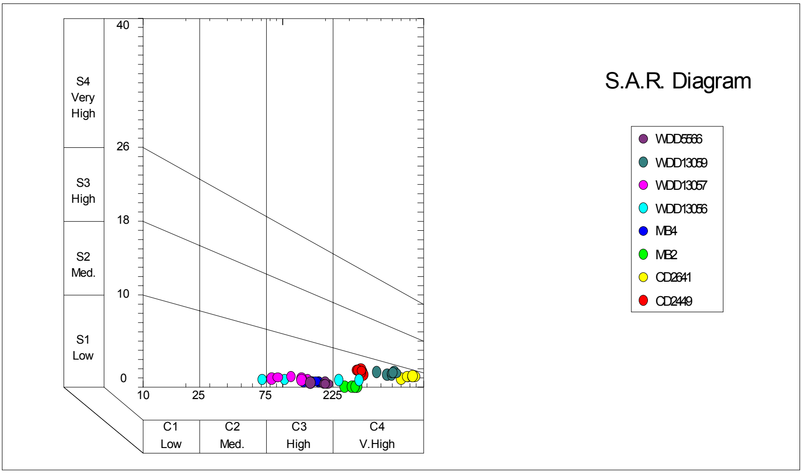

Figure 4: Hydrograph of sodium adsorption ratio to conductivity in µS/cm.

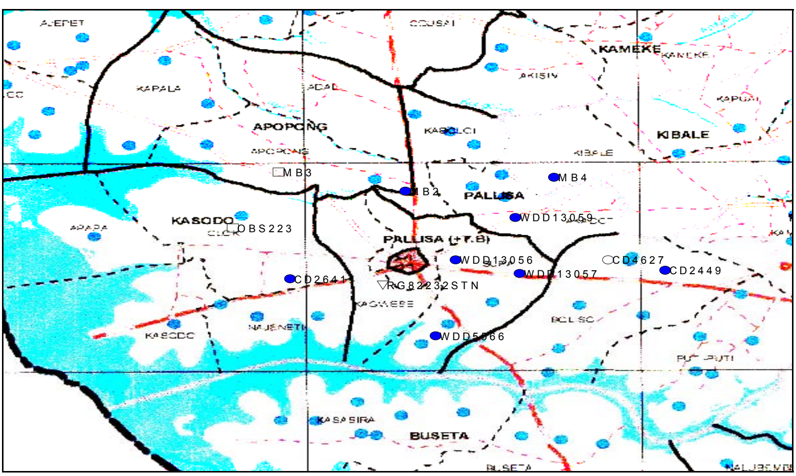

Figure 5: Positions of some of the drilled boreholes around Pallisa town. Source: RUWASA. Scale: 1: 400 000.

4 Discussion of the results

Figures 2, 3 and 4 shows piper, hydrograph and durov plots of boreholes situated in both the regolith and the basement complex (bed rock) of Pallisa district.

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Looking at the figures 1 and 2, it can be observed that the boreholes situated in these aquifers of Pallisa represent a regime where there is calcium enrichment, which is typical of lime dosing to neutralize acid waters. In the piper diagram, plot of water contains approximately 70% Ca, 70% Mg, 80% TAlk, 10% Cl and 10% SO_4. This is typical of calcium/magnesium bicarbonate waters. So both the regolith and bedrock groundwaters are dominated by the carbonate ions. All in all the groundwaters in Pallisa district are unpolluted and therefore generally suitable for drinking except for few areas with high concentration of iron. Although water rock interactions in both aquifer units may lead to parallel geochemical evolution of groundwater, the data at least do not deny a hydraulic connection. It is known with the exception to correlation is aluminium. Higher concentrations of aluminium are observed in the regolith while low levels in the bedrock. The higher levels of aluminium are the product of highly active weathering in the regolith, in that the bedrock aquifers receives recharge primarily in areas where the aquifer units are sufficiently linked to allow the water to move through the regolith and into the bedrock with minimal geo-chemical interaction. Where hydraulic interaction is poorly developed, the groundwater will tend to remain in the regolith and the geo-chemical weathering is able to proceed without inhibition.

The significantly lower level of aluminium in the bedrock aquifer provide further support for the notion of relatively limited hydraulic interaction between the two units.

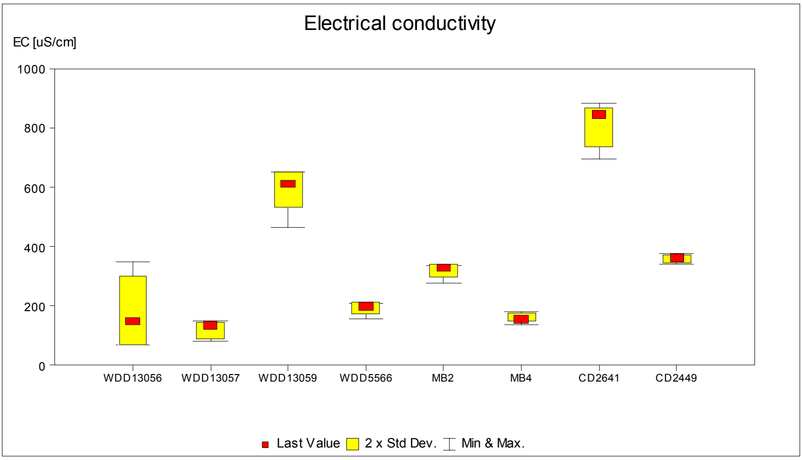

Figure 6: Box and whisker plot of electrical conductivity of the boreholes drilled to less than 800 μS/cm. within the basement complex.

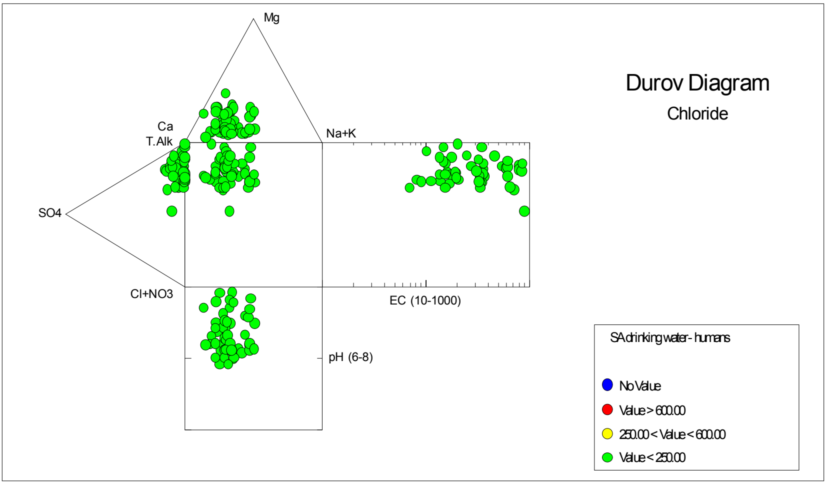

Figure 7: Concentrations of chloride of all the boreholes tested in Pallisa.

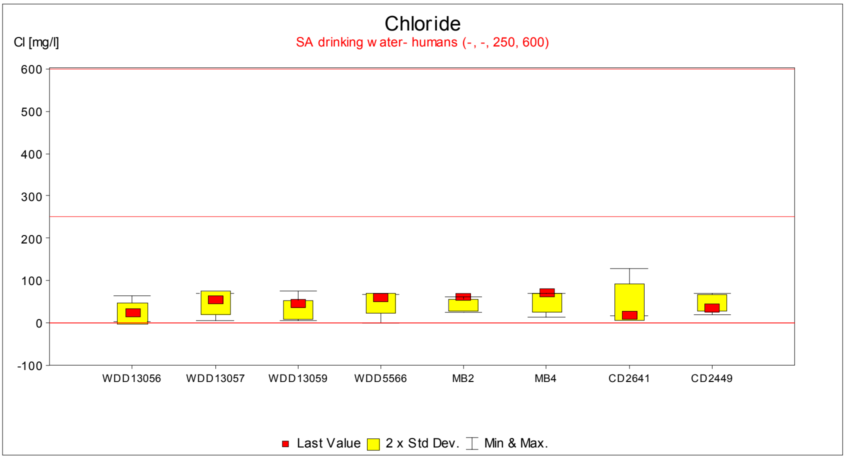

Figure 8: Box and whisker plot of chloride concentration (mg/l) of boreholes drilled in Pallisa basement complex.

5 Conclusion

From a human health point of view, most of the groundwaters are generally acceptable in terms of their inorganic water quality. Aesthetically, however, many groundwaters show excessive levels of aluminium, chloride, iron, manganese, zinc and hardness in a limited number of wells, thus substantiating concerns raised by the consumers nearby some of these wells.

Where health standards are exceeded, nitrate and chromium are the usual problems. Evidence of groundwater quality deterioration is associated with the corrosion of the borehole casings and the raising mains, or the seepage of the wastewater into shallow wells from domestic wastes due to insufficient sanitary practices by humans and livestock. Wastewaters are generally responsible for elevated concentrations of chlorides and nitrates, while corroded pipe work increases the contents of iron, zinc and manganese. However, elevated aluminium, iron, manganese and to some extent chromium, are also associated with natural weathering of the aquifer matrix.

Ongoing work as part of the principal author's research aims to finalise the chemical composition of groundwater in Pallisa and a further study in the sodium absorption ratio (SAR) of the soil to be re-evaluated. Given the enormous variations in conductivity over short distances, there is need to study the hydrogeology of the area at a high resolution. This will enable the study and understanding of the spatial variation of hydrogeology and its causes.

Acknowledgements

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The author would like to thank Professor F.D.I Hodgson particularly for his contribution in this research, Eng. J.C Kasiita, and Edison Niwagaba for the tremendous work provided. I thank the Departments and the personnel of the DWD Library and WRMD Library, Entebbe, RUWASA and Kyambogo University Library for their assistance and guidance.

References

[1] Morgan, N.E., Groundwater chemistry and quality assessment of Lake Naivasha area, Kenya. M.Sc. Thesis, International Institute for Aerospace Survey and Earth Sciences, Enschede (ITC).1998. pp.16.

[2] National Curriculum Development Center, Uganda: Geological, soils and natural vegetation maps of Uganda. *Scale* 1:7 000 000 .2001.pp 17.

[3] Key, R.M., An introduction to crystalline basement of Africa. In: Wright, E.P and Burgess W.G (eds), Hydrogeology of crystalline basement aquifers in Africa geological society, London, Publication No. 66. 1992, pp.29 – 57.

[4] Briggs, D., P. Smithson and T. Ball, Fundamentals of Physical Geography (Can. Ed.) Copp Clerk, Ontario, Canada. 1989. pp.594

[5] Taylor, R.G., K.W.F. Howard, J. Karundu and T. Callist, Distribution and seasonability of groundwater recharge to the regolith basement aquifer system in Uganda. GSA abstracts vol.25, No. 6 (September 1993).

[6] Nyende J. and Hodgson F.D.I, Evaluation of groundwater resource potential of Pallisa district in Eastern Uganda. *Institute for groundwater studies, University of the Free State RSA. 2003.* pp.11

[7] World Health Organization, Guidelines for Drinking-water Quality, Vol. 1, Recommendations. *World*

Sediment transport in sewers

R. Celestini[1], G. Silvagni[2], M. Spizzirri[1] & F. Volpi[2]
[1]*Acea Ato2 SpA, Rome, Italy*
[2]*Department of Civil Engineering, University of Rome, "Tor Vergata", Italy*

Abstract

Solid transport in combined sewers is characterized by a succession of two phenomena: erosion of sediment from the pipe bottom and deposition of solid matter transported by the flow during the sequential phases of a storm wave.

The alternation of these phenomena in a combined sewer network causes the deposition of sand and organic material during the time between two successive storm waves.

Dry weather flow generally cannot activate near-bed solids transport, so that the sediment remains in sewer trunks until the first storm flow occurs.

After long periods of permanence at the sewer pipe bottom, especially during summer, organic material can undergo a process of decomposition. In this case, wet weather flow created by the first storm event often causes erosion and transport of solid material in a decomposed state to the receiving aquatic systems, with the risk of anoxia.

This phenomenon is being analysed in two different hydrographic basins in Rome, with on-site monitoring of two combined sewer networks, whose dry weather flow and first storm flow, according to designed dilution ratios, is conveyed to the Roma Nord wastewater treatment plant and whose combined sewer overflow (CSO) spills are received by the river Aniene, one of the main tributaries of the river Tevere.

This experimental activity is aimed at both locating, characterizing and quantifying sediment deposits in such sewer networks, and defining a sediment transport model to predict the position of sewer trunks with high probability of deposit and to estimate mean values of sediment volume.

Keywords: combined sewerage, sewer sediments, transport model, water quality.

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1 Introduction

The increasing sensitivity towards problems in the solid transport phenomenon at international level, has given rise to: the development of experimental studies aiming to define self-cleaning design criteria for different types of sewer pipes; the definition of predictive models to localise and quantify sediment deposit in sewer networks, aiming to support the management of the whole system.

However, the analysis of solid transport in sewers is still characterised by a significant uncertainty related to the high number of variables on which the phenomenon depends; the location and type of sewerage, the nature of solid material, the granulometric composition, the presence of cohesion, the size, shape, material and slope of sewer pipes, are but few variables indispensable for a detailed description of the system.

The movement of solid material in sewer trunks is the result of different forms of transport by flow, all contributing to a global solid flow. The different types of solid transport can be simplified into the following three phases (Crabtree et al [1], Gent et al [2]):

- suspended transport of middle and small-sized particles, generally at the same rate or slightly slower than the flow rate;
- a dissolved phase transport of very small particles, at the same rate as the flow rate and completely diluted in flow;
- bed and near bed transport, characterised by middle and large-sized particles, which move under the effect of higher flow conditions.

Although it is possible to classify such phenomenon as shown, varying the instantaneous flow conditions (i.e. altering the energetic content), a single solid particle can pass from one type of transport to another. The great range of sizes and materials of common particles, as well as the high flow variability over time, are all elements that contribute to making the real boundaries between different phases even less definable.

Recent studies, aimed to adapt classical solid transport models for natural channels to sewer systems, have highlighted the significant dissimilarity between the two cases. Such difference makes the classical transport models of the first half of the twentieth century practically inadequate to approach the analysis of solid transport in sewerage.

The main differences between the two cases include characteristics such as:

- composition, size and nature of transported materials; in particular, the presence of cohesion strongly influences the resistance of solid deposit to abrasion;
- flow characteristics, which can change rapidly especially in combined sewer pipes;
- shape and size of pipe sections; large rectangular river sections can be considerably different from sewer sections, which are often small and circular or egg-shaped;
- action on flow by macro roughness of sewer beds.

Experimental results have also shown how the granulometry of sediments in sewer beds highly influences the amount of eroded and re-suspended material under fixed flow rates. The presence of granular matter in beds composed of fine solid material, for example, can increase erosion thanks to the abrasive action of sand in motion, by reducing the cohesion created by clay particles (Tait et al [3]).

The great variability of such factors is the main cause of a remarkable uncertainty in qualitative and quantitative solid transport analysis. A proper description of this phenomenon requires a complex model and an appropriate calibration of its input data, based on experimental studies and a high number of on-site measurements. Solid transport problems in natural basin drains are here analysed thanks to a simplified simulation method which represents a valid compromise between a realistic description of transport phenomena and quick data processing. Such an approach is based on the relation between transport capacity by different flow rates and quantity of solid material conveyed by storm water into sewer pipes from the whole basin surface.

The model here illustrated has been calibrated for the sewer system of Cesarina catchment, a large basin located east of the centre of Rome, on the right side of the river Aniene (2015 ha).

2 The model

The solid material heterogeneity, as well as the high variability of hydrodynamic flow conditions during a storm wave, implied the need for a simplified model founded on the separation between hydrodynamic simulation of the sewer network and analysis of solid transport development.

Starting from such a hypothesis, all calculations are based on the following consequential steps:

- characterisation of hydrodynamic conditions into sewer pipes during storm waves (U.S. EPA [4]);
- evaluation of the amount of solid material accumulated on the basin surface (Alley and Smith [5]);
- evaluation of flow transport capacity for each storm event (Verbanck [6, 7]);
- estimation of deposited solid material according to a balance between entering solid material and transport capacity in each sewer trunk (Silvagni et al [8]).

This balance, continuatively developed over time and space, is aimed to quantify solid transport during different phases of a storm wave, as well as deposits on pipe bottoms.

2.1 Hydraulic modelling of the sewer network

The flow capacity to re-suspend particles deposited on the pipe bottom, and to transport them within the water solution, is strictly related to the flow shear stress on the pipe walls and to the mean flow velocity in each pipe section.

Therefore, to simulate the alternation of erosion and deposit phenomena, it is fundamental to know how hydrodynamic characteristics of flow vary in the sewer network over time and space. For the examined sample basin, the hydrodynamic SWMM engine [4] has been used with the Horton infiltration model to solve the complete De Saint Venant equation under the hypothesis of mono-dimensional flow conditions.

The analysed sewer network is mainly composed of egg-shaped pipes (3.0x3.75 m and 4.50x5.04 are the sizes of the final segments) with an inferior semi-circular cunette, while the final trunk's section is box-shaped, 3.80x4.90 m, with a trapezoidal-shaped bottom. This last segment, 300 m long, ends with a Combined Sewer Overflow (CSO) chamber which is connected downstream to the Roma Nord wastewater treatment plant. Calibration is based on field data collected in the terminal section of the main egg-shaped pipe, where a level gauge and an automatic water sampler have been installed. First results show a substantial agreement between computed and registered data (fig. 1 and 2).

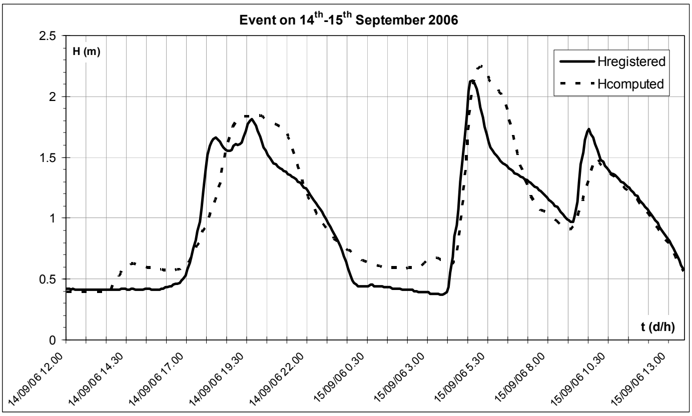

Figure 1: Event on 14th-15th September, 2006 – hydraulic model's calibration.

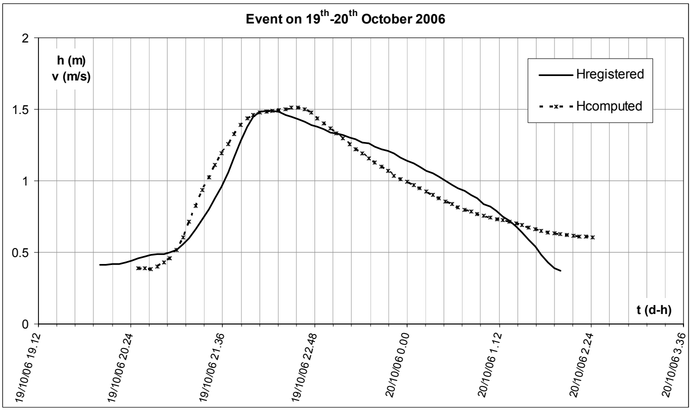

Figure 2: Event on 19th-20th October, 2006 - hydraulic model's calibration.

2.2 Superficial solid storage and transport capacity

The main sources of solid material transported by combined sewer flow during a storm event are both particles deposited on the basin surface during a dry period, and re-suspended particles deposited on sewer beds during the decreasing phase of the previous storm wave, scarcely eroded by dry weather flow.

Referring to experimental studies conducted by Alley and Smith [5], storage of solid material on the basin surface was evaluated with the following expression:

$$M_{acc} = M_{acc,lim}\left(1 - e^{-\frac{Disp \cdot \Delta t_s}{24}}\right) + M_r e^{-\frac{Disp \cdot \Delta t_s}{24}} \quad \text{(kg).} \qquad (1)$$

where: *Accu*, accumulation rate of solid material which depends on basin urbanisation (kg/ha/d); *Disp*, dispersion coefficient given by 0.08 d^{-1}; *S*, basin surface (ha); *Peim*, percentage of impermeable area; M_r, residual mass on the basin surface at the end of the last storm event (kg).

The experimental activity on the road drains conducted in the land of Comune di Roma during the 1999 – 2002 biennium (Silvagni [10, 11]), as well as the first experimental results of field study in the sewer system, allowed the definition of characteristic accumulation rates for the analysed network.

Table 1: *Accu* and *Peim* values.

Basin	Peim	Accu (kg/ha/d)
Cesarina stream	15%	6
Cinquina stream	40%	6
Urban	65%	10

The catchment, with a global surface of 2015 ha, is composed of two large non-urbanised areas (Cesarina basin, 1200 ha; Cinquina basin, 640 ha) and of a densely populated urban area (175 ha).

Under the hypothesis of isolated and sufficiently intense storm events, it is assumed that solid material deposited on the basin surface is completely washed away, and eroded material reaches the sewer trunks with the modulating action of road drains. Such a global mass of conveyed material equals the following limit:

$$M_{acc,lim} = \frac{Accu}{Disp} Peim \cdot S \quad \text{(kg).} \qquad (2)$$

Under this hypothesis, the concentration in flow of solid material washed away from the basin surface and introduced in the sewer system during a storm event equals:

$$C_s = \frac{M_{acc,lim}}{V_{ev}} \quad \text{(kg/m}^3\text{).} \qquad (3)$$

where V_{ev} is water volume globally introduced into the sewer through road drains of an examined segment during the storm event (m^3).

A sensitivity analysis of characteristic variables, conducted on a wide range of experimental studies, allowed the selection of the Verbanck [6, 7] formulation for solid transport capacity, deduced for sewer pipe shapes and sizes comparable to the ones in the Cesarina network.

The concentration of suspended solids transported by flow (C_v), under the hypothesis of non-cohesive material, can be evaluated with the following expression, using sedimentation velocity w_s (m/s), hydraulic radius R (m), gravitational acceleration g (m/s^2), relative density of sediments in water Δ, and tangential velocity on the bed of sediments $u_{*,b}$ (m/s):

$$C_v = \frac{1}{5.16} \frac{u_{*,b}^3}{g \Delta w_s R} \qquad \text{(adm)}. \tag{4}$$

$$u_{*,b} = \frac{u_m}{\frac{1}{k} ln\left(\frac{12H}{\varepsilon_{tot}}\right)} \qquad \text{(m/s)}. \tag{5}$$

where H is the depth of flow (m); u_m the mean flow velocity (m/s); k the Von Karmann constant, 0.40; ε_{tot} the global pipe roughness (mm).

2.3 Balance

For a certain storm event, the hydrodynamic simulation allows the calculation, for each segment, of the flow rate $Q(t)$, the depth of flow $H(t)$ and the mean flow velocity $V(t)$; in particular, the model takes into consideration the hydraulic conditions at the terminal section of each segment, considered more representative of solid transport capacity.

Solid material accumulated on the basin during the dry interval between two significant storm events and solid transport capacity by flow, are calculated with relations (2) and (4), under the simplifying hypothesis of non-cohesive material and immediate system response in terms of erosion and deposit. It is also assumed that solid flow entering one of the initial segments of the sewer network equals, at every time step, the expression:

$$Q_{s,ing}(t) = C_s \cdot Q(t) \qquad \text{(kg/s)}. \tag{6}$$

where C_s corresponds to solid material concentration of flow accessing the segment and it is considered constant during the same storm event, eqn. (3). When a sewer trunk downstream of the initial segments is analysed, solid flow conveyed from the upstream network is added to the solid rate coming from the road drains belonging to the said trunk.

The instantaneous balance of solid mass present in each sewer trunk was so computed, by calculating the amount of solid material deposited in each trunk, at the i time step, as:

$$M_{dep}(i) = M_{t=0} + \sum_{j=0}^{i} \left[Q_{s,ing}(j) - Q_{s,usc}(j) \right] \cdot \Delta t \qquad \text{(kg)}. \tag{7}$$

where $M_{t=0}$ is the solid mass present on the pipe bottom before a storm event occurs (kg) and $Q_{s,usc}(j)$ is the solid flow (kg/s) pouring out of the analysed segment, transported downstream. The latter equals flow transport capacity, eqn. (4), when there is storage of deposited material to erode, otherwise it is lower.

This function can be expressed as:

$$Q_{s,usc}(i)=\begin{cases} Cap_s(i) \text{ when } M_{t=0}+\sum_{j=0}^{i}\left[Q_{s,ing}(j)-Q_{s,usc}(j)\right]\cdot\Delta t+Q_{s,ing}(i)\cdot\Delta t > Cap_s(i)\cdot\Delta t \\ \dfrac{M_{t=0}+\sum_{j=0}^{i}\left[Q_{s,ing}(j)-Q_{s,usc}(j)\right]\cdot\Delta t+Q_{s,ing}(i)\cdot\Delta t}{\Delta t} \quad \text{otherwise} \end{cases} \tag{8}$$

where $Cap_s(i)$ is the solid rate that can be transported by flow, given by:

$$Cap_s(i)=C_v(i)Q(i)\cdot\rho_s \qquad \text{(kg/s).} \tag{9}$$

with ρ_s specific weight of solid material (kg/m^3).

Therefore, for each examined sewer segment it is possible to draw, over the duration of considered storm events, the patterns of the following functions:

- water flow $Q(t)$, depth of flow $H(t)$ and mean flow velocity $V(t)$;
- global solid transport capacity $Cap_s(t)$;
- entering solid flow $Q_{s,ing}(t)$ and solid flow conveyed downstream $Q_{s,usc}(t)$;
- instantaneous amount of deposited solid material along the whole analysed segment.

3 Initial results

Surveys conducted in the terminal trunk of the Cesarina sewer (box-shaped section 3.80x4.90 m) highlighted the presence of temporary deposits of solid material during dry periods between sequential storm events, whose maximum depth reached the sidewalk level (about 0.80 m).

The deposit is mainly made of non-cohesive sand, which is normally re-suspended and conveyed downstream by significant storm waves. The effect of foul flow on deposits of solid material has not been taken into account in this study.

The model is rapidly self-calibrated: in particular, under the initial hypothesis of sewer pipes completely free of deposits, after simulating the effect of the second storm event on the system, the M_{dep} output values become compatible with measured ones.

The relatively high CSO weir, which allows the storage of a large first storm water volume, as well as water pouring back into the CSO chamber from the river Aniene during major flood events, are causes of flow velocity decrease and a sudden reduction of solid transport capacity during both peak and minimum flow.

Under the hypothesis discussed above, considering for solid material the specific weight ρ_s= 1500 kg/m^3 and a sedimentation velocity w_s = 0.006 m/s, the following graphs show model's results for the terminal sewer trunk of Cesarina system, during the storm events on 14th-15th September 2006 (fig. 3), 19th October 2006 (fig. 4), 1st and 12th November 2006 (fig. 5 and 6).

In particular, simulated values for the amount of deposited material M_{dep}, whose patterns over time (t) are here shown, substantially match experimental data.

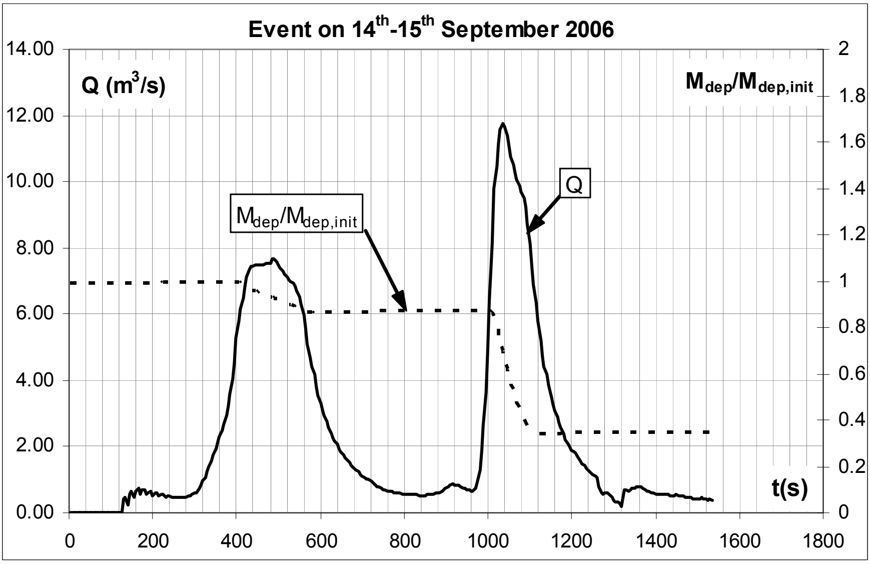

Figure 3: Event on 14th-15th September 2006 – Q(t) and $M_{dep}/M_{dep,init}(t)$ pattern.

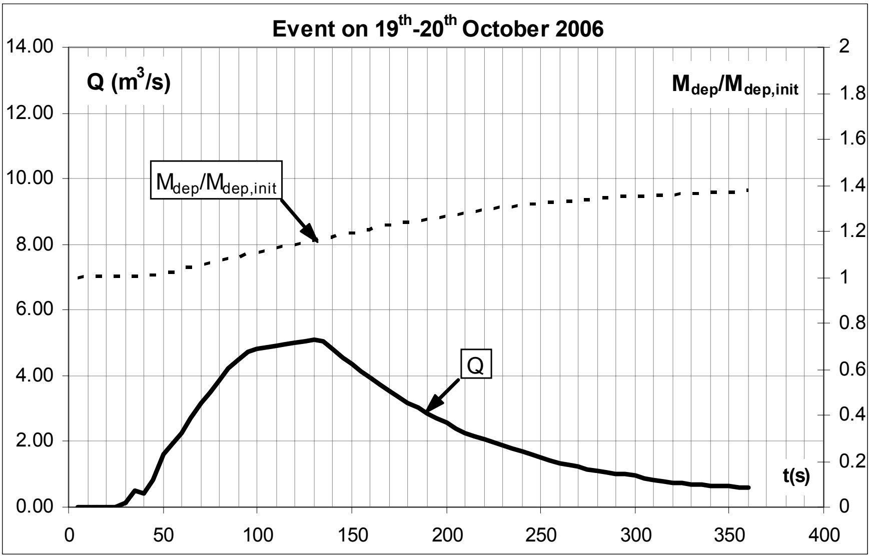

Figure 4: Event on 19th-20th October 2006 – Q(t) and $M_{dep}/M_{dep,init}(t)$ pattern.

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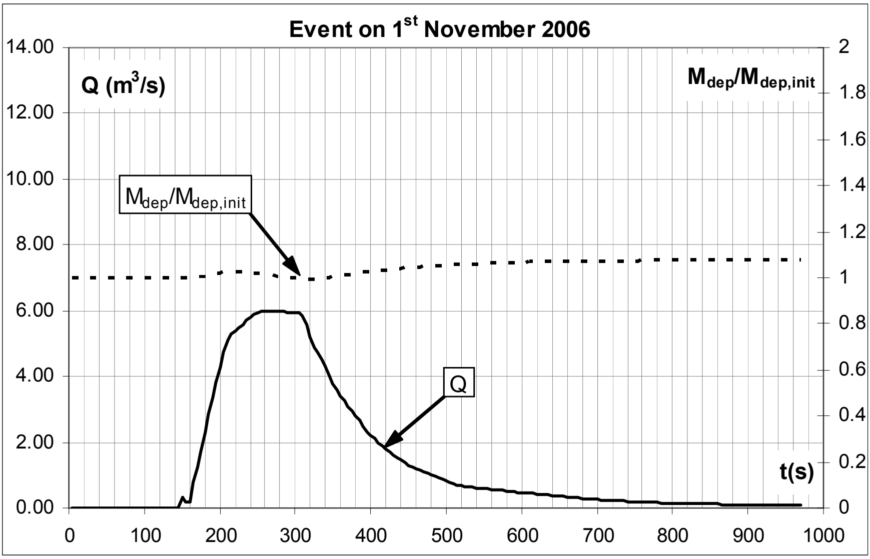

Figure 5: Event on 1st November 2006 – Q(t) and $M_{dep}/M_{dep,init}(t)$ pattern.

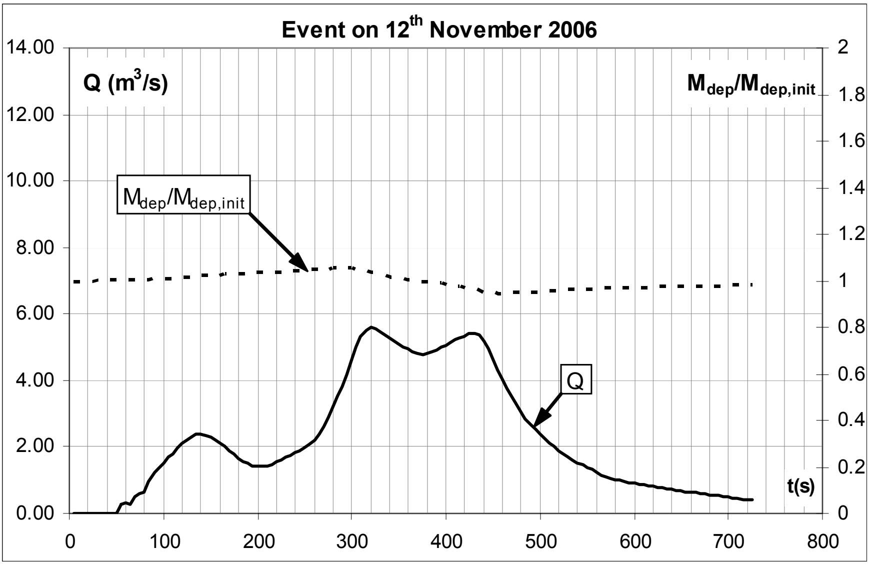

Figure 6: Event on 12th November 2006 – Q(t) and $M_{dep}/M_{dep,init}(t)$ pattern.

4 Conclusions

Solid transport in sewers has been studied thanks to a simplified model which, through the hydrodynamic simulation of a sewer network and the evaluation of flow transport capacity varying hydraulic characteristics, allows the location of all segments of the sewer network with the highest probability of deposit phenomena.

The practical application of such analysis to the sewerage of the Cesarina basin allowed the quantification of the effects of solid material deposit in the terminal trunk of the network, substantially confirmed by the results of on-site experimental research.

References

[1] Crabtree, R.W., Ashley, R.M., Gent, R.; *Mousetrap: modelling of real sewer sediment characteristics and attached pollutants*, Water Science and Technology: Vol. 31 No. 7, 1995;

[2] Gent, R., Crabtree, B., Ashley, R. M.; *A review of model development based on sewer sediments research in the UK*, Water Science and Technology: Vol. 33 No. 9, 1996;

[3] Tait, S. J., Rushforth, P. J., Saul, A. J.; *A laboratory study of the erosion and transport of cohesive-like sediment mixtures in sewers*, Water Science and Technology: Vol. 37 No. 1, 1998;

[4] United State Environmental Protection Agency (EPA); *Storm Water Management Model – User's Manual*, 2004;

[5] Alley, W.M. & Smith, P.E.; *Estimation of accumulation parameters for urban runoff quality modelling*, Water Resource Research: vol. 17, No. 6, pp. 1657-1664, 1981;

[6] Verbank, M. A.; *Assessment of sediment behaviour in a cunette-shaped sewer section*, Water Science and Technology: Vol. 33 No.9 pp.49 - 59,1996;

[7] Verbanck, M.A; *Computing near – bed solids transport in sewers and similar sediment – carrying open-channel flows*, Urban Water: pp.277 – 284, 2000;

[8] Silvagni, S., Celestini, R., Volpi, F.; *Stima dell'evoluzione del trasporto solido in fognatura*, VIII Simposio Italo – Brasileiro de Engenharia Sanitaria e Ambiental, September 17-22, 2006;

[9] Celestini, R., Silvagni, G., Volpi, F.; *Transport of solid material in sewer pipe – The sample catchment of Cesarina (Rome)*, submitted to the 32nd Congress of IAHR, July 1-6, 2007;

[10] Silvagni, G., Volpi, F., Rubrichi, G.; *Problematiche connesse al trasporto solido in fognatura*, XXVII Convegno di Idraulica e Costruzioni Idrauliche: vol. 3, pp. 295-302, 2000;

[11] Silvagni, G. & Volpi F.; *A model for the evaluation of silting process of gully – pots*, 2nd International Conference New Trends in Water and Environmental Engineering for Safety and Life: Eco-compatible Solutions for Aquatic Environments, Capri (Italy), June 24-28, 2002.

Influence of redox cycle on the mobilization of Fe, Zn, Cu and Cd from contaminated sediments: a laboratory investigation

L. T. Nguyen[1], T. Lundgren[2], K. Håkansson[3] & B. H. Svensson[1]
[1]*Linköping University, Sweden*
[2]*Envipro Miljöteknik AB, Sweden*
[3]*Geo Innova AB, Sweden*

Abstract

An investigation was set up to elucidate the dynamics of metals flows from a contaminated and eutrophic lake. Two sediment layers were incubated in flow-cells to study the mobilization of Fe, Zn, Cu and Cd during the anoxic and oxic conditions. pH, inorganic carbon (IC) and dissolved organic carbon (DOC) were monitored to assess their influences on the metal mobilization. Under anoxic conditions, Fe was rapidly released to the solution, but taken up during the oxic conditions. The Fe-mobilization was not affected by pH, but positively correlated with the increase in DOC during the anoxic period. In contrast, Zn, Cu and Cd were removed from the solution during the anoxic period and released to the solution during the oxic conditions. The change in pH during the oxic period may have contributed to the increase in concentrations of Zn, Cu and Cd. The results of the experiment show that periodical redox changes likely are among the factors controlling the mobilization of Fe, Zn, Cu and Cd from the contaminated sediments.
Keywords: sediment, flow-cell, metal, mobilization, redox condition.

1 Introduction

Changes in redox conditions play a significant role in mobilization of heavy metals from sediments. Under anoxic conditions, heavy metals usually occur as stable sulfides buried in the sediment layers, which results in low concentrations of metals in the water phase Calmano *et al* [3]. Imposing oxic conditions, the

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metal sulfides occurring under anoxic condition are oxidised and dissolved as ions or complexed by dissolved organic compounds. These may be transported or re-adsorbed on more reactive solid surface sites such as freshly-precipitated iron hydroxides or biotic material Calmano *et al* [2].

Lake Håcklasjön (250 km SE of Stockholm, Sweden) is a shallow, eutrophic lake and has elevated concentrations of Zn, Cu, Cd, Fe and organic matter in the sediment profile Eklund and Håkansson [6]. The contaminated lake sediments reflect the serious metal pollution history of the area, where the lake continuously and for a long period received the discharges from one of the biggest smelter in Sweden during the 18th and 19th centuries. The lake has been the recipient for untreated, but later treated sewage from town Åtvidaberg. The lake is now used as a dam for hydropower purpose. During operation periods, the power plant discharges about 4 m^3/s into the lake, causing strong water currents in the lake, resulting in the oxidation and resuspension of the surface sediments. During non-operation periods and, thus, low water flows the lake becomes stagnant and anoxic conditions are likely developed.

The main objective of our study was to examine the possible effects of a redox cycle caused by the hydropower plant on the mobilisation of the redox-sensitive metal Fe and the heavy metals Zn, Cu and Cd. As a part of this study, two sediment layers (the top 6-cm and the deeper 8-12 cm) were each set up in experimental flow-cells, where anoxic and oxic conditions were in turn introduced. The parameters pH, inorganic carbon (IC) and dissolved organic carbon (DOC) and redox were monitored to assess their influences on the mobilisation of the target metals.

2 Materials and methods

2.1 Field sampling

Sediment cores were collected from lake Håcklasjön into acrylic cylinders in July 2005. The collected cores were sealed with 20-cm height of overlying water and rubber stoppers on top to avoid intrusion of air. Lake water was taken at ~1 m depth by a water sampler to a 15-l plastic carboy. All the samples were kept airtight and cool during transport to the laboratory. Upon arrival, the lake water and sediment samples were stored dark in a refrigerator at 4^{o}C until they were treated and used.

2.2 Flow-cell design

Three-compartment flow-cells (l×w×h=36×11×11 cm, fig. 1), made of unstained polypropylene, were filled with sediment material in the middle compartment (22×6×11 cm). The two outer compartments functioned as reservoirs for an even flow of water above the sediments when used in circulation mode. Sediments were confined in the midle compartment (22×6×11 cm). A peristaltic pump was connected to each flow-cell via PVC tubing equipped with a Luer-lock joining with a 60-ml syringe for collection of overlying water. Each flow-cell was sealed with an airtight lid, followed by silicone gluing at the conjunctions.

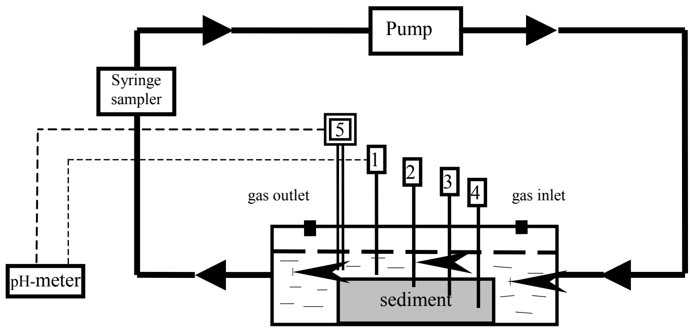

Figure 1: Schematic drawing of the flow-cell setup for the experiment. Platinum wires (1-4) and calomel reference electrode (5) connected to a standard pH-meter (PHM 83, Radiometer) were constructed and immersed in the overlying water (at 1-cm depth above the sediment surface) and in the sediments at 0.5-1, 1.5-2 and 3-3.5 cm.

2.3 Experimental procedure

The lake water was purged with N_2 gas (<1 ppm (v) O_2, AirLiquide, Sweden) over night to decrease dissolved O_2 before the start of the anoxic incubation and a continuous N_2 gas flow (20 ml/min) was applied to the flow-cells *cf.* fig. 1 to employ anoxic conditions. The gaseous content of oxygen was regularly checked with an oxygen analyzer (Servomex 570 A) via the gas outlet and stayed <0.1% during the anoxic experiment. All sample preparations were carried out in a glove box flushed with N_2 gas resulting in <0.4% O_2. The cells were incubated at 15°C in the dark. Construction for redox electrodes and measurements followed the procedure by Svensson and Rosswall [13].

Water standing above the sediment surface was siphoned off and the cores were sliced into: 0-0.5, 0.5-1.5, 1.5-2.5, 2.5-4, 4-6 and 8-12 cm sections. Slices from each level were lumped together in airtight N_2 pre-filled plastic containers, followed by homogenization under O_2-free condition. The top sediment layers (0-6 cm) were transferred in depth-order into one cell, while the deeper mixed-layer (8-12 cm) was placed into another cell to give 6-cm height. The cells were then filled with de-oxygenated water to a height of 3-cm above the sediments and the lids were sealed to the cells. While being at anoxic stagnant conditions for 21 days, water samples (60 ml each) were withdrawn on 6 occasions: at 3 h and 1, 3, 4, 11 and 21 days. In order to mix the stagnant water before sampling, a slow recirculation (~50 ml/min) was applied for about 10 minutes in both cells. Thereafter, oxic conditions were applied to both flow-cells by exchanging the N_2-flow for technical air (AirLiquide, Sweden). Simultaneously, water recirculation was applied in the two cells at a rate of ~1.4 l/min. Overlying water samples were sampled at 6 occasions at 1, 2, 4, 6, 8 and 11 days after the aeration start. Before sampling, the pumps were stopped for 4-5 minutes to enable stable redox readings and to let the suspended particles settled. After withdrawal of overlying water, an equivalent volume of filtered-lake water (deoxygenated during the anoxic and oxygenated during the oxic periods, respectively) was supplied to the flow-cells to conserve the initial solid/liquid ratio.

2.4 Sample analysis

Total contents of Fe, Zn, Cu and Cd in the sediments were determined using ICP-OES (Al-Control Laboratories, Sweden) after digestion in aqua regia according to standard method [18]. Blanks using Milli-Q water and certified reference samples using PACS-2 (National Research Council, Canada) were prepared in the same way as the samples to evaluate the analytical procedures and the quality of the obtained data. The accuracy was within 15% of the certified concentrations for all elements of interest. The concentrations of target elements in the blanks were below the detection limits of the analytical technique used. The water contents and estimation of organic matter contents (LOI) in the sediments were determined following standard methods [16, 17].

As soon as collected, the overlying water was filtered through 0.4 μm polycarbonate filters (Millipore) to determine the concentrations of dissolved Fe, Zn, Cu and Cd, dissolved organic carbon (DOC) and inorganic carbon (IC). All samples for metal determinations were preserved with concentrated HNO_3 to pH <2 and stored dark at 4^oC until analysis. The pH of the overlying water was determined using a pH-meter (PHM 83, Radiometer). Concentrations of IC and DOC were obtained using a TOC analyser (TOC-500, Shimazu). Concentrations of Fe and Zn were determined using flame atomic absorption spectrometry (AAS, Perkin Elmer 1100), whereas Cu and Cd were determined using AAS with furnace. Blanks were prepared and analysed in the same way as the samples to control the obtained data. The concentrations of the metals of interest in the blanks were below detection limits of the analytical technique used.

2.5 Statistical analysis

Pearson's correlation coefficients (r) were calculated and used in the correlation analyses to check the relations between the target parameters, using SPSS 11.5 for Windows (SPSS Inc., USA). The correlations were considered to be statistical significant at $r \geq \pm 0.7$ at the set $p \leq 0.05$ level, using a 2-tailed test with n=6.

3 Results and discussion

The original sediments showed lower pH-levels with depths (from pH 7.1 to 6.8, table 1) and the metal concentrations of Zn, Cd and Cu were high *i.e* class 4 to very high *i.e.* class 5 according to the 5-level scale classified by Swedish

Environmental Protection Agency [15]. The water content and organic matter content (LOI) of the sediments decreased slightly with depths, table 1.

The lake water shows high pH (pH 8.9), which is probably induced by the photosynthesis or chemosynthesis of aquatic plants during the sampling occasion in July Mayer *et al* [8]. This eutrophic lake Håcklasjön's water shows high concentration of DOC, about 2- to 6-fold more elevated than the average DOC concentrations in eutrophic lakes Drever [4]. The concentrations of Cd, Zn and Cu in the lake water were moderately high, falling into class 3 on the 5-level scale classified by Swedish Environmental Protection Agency [15].

Table 1: Physio-chemical compositions of the original samples used in the experiment. The metal contents in the sediments were calculated on dry weight basis.

Filtered lake water								
	pH	IC (mg/l)	DOC (mg/l)	Fe (µg/l)	Zn (µg/l)	Cu (µg/l)	Cd (µg/l)	
	8.9	10	12	380	42	6	0.1	
Sediments								
Depth (cm)	pH	Water cont. (%)	LOI (%)	Fe (µg/g)	Zn (µg/g)	Cu (µg/g)	Cd (µg/g)	S (µg/g)
0-0.5	7.1	82	33	44900	3930	640	17	21800
0.5-1.5	7.0	82	32	43500	2560	630	9	20600
1.5-2.5	6.9	80	32	44600	3850	690	16	22000
2.5-4	6.8	80	32	42200	4210	720	30	21200
4-6	6.8	81	32	45300	4280	730	20	24600
8-12	7.1	79	30	48100	5560	1220	29	29600

The results of the experiment are presented in fig. 2 and fig. 3. During 21 days of the anoxic stagnant incubation, the pH-pattern was similar for both sediment layers, fig. 2. However, once shifted to oxic conditions, the pH for the top layer decreased from 7.6 to 6.9, suggesting that oxidation of sulphides occurred, whereas the pH for the deeper layer increased from 7.5 to 8.

During the anoxic event, the concentrations of IC gradually increased from ~8 to ~20 mg/l in both sediment layers, fig. 2, suggesting that a decomposition of organic matter or a dissolution of carbonate minerals occurred. In contrast, the IC concentrations rapidly dropped from 18 to 4 mg/l for the top layer and from 20 to 9 mg/l for the deeper layer during the first two days after air introduction to remain stable for the rest of the experiment, fig. 2.

As for IC, the concentration of DOC for both layers gradually increased during the anoxic incubation, fig. 2, indicating that anaerobic degradation of organic matter was likely occurred in the sediments, *cf.* Salomons [10]. The concentrations of DOC rapidly decreased upon the initiation *i.e.* within the first 2 days of aeration.

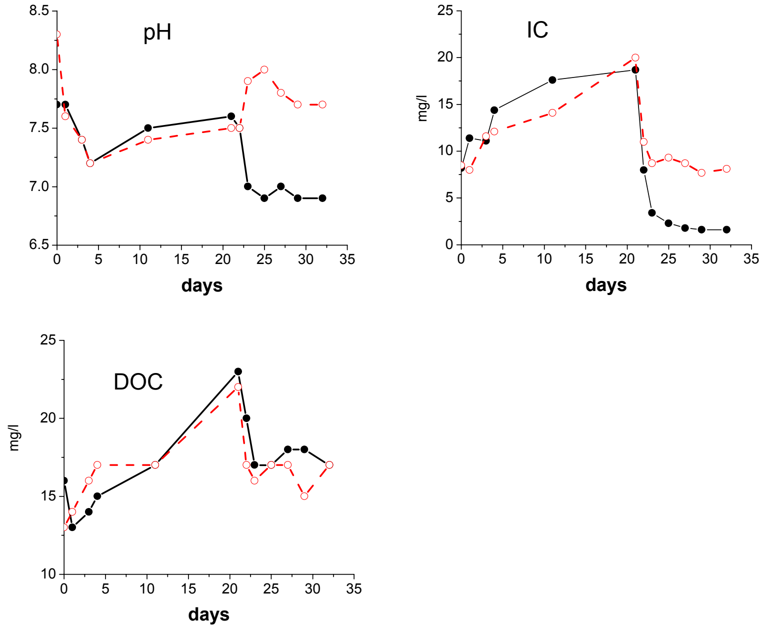

Figure 2: Evolution of pH, inorganic carbon (IC) and dissolved organic carbon (DOC) in solution during the experiment. Solid lines represent the top layer and dashed lines represent the deeper layer.

The Eh values were unfortunately not recorded during the anoxic period and the first sampling during the oxic conditions. However, after one day of aeration, the Eh value of the overlying water was recorded at +420 mV and remained stable between +410 and +480 mV during the rest of the oxidation, table 2. The Eh values of three sediment depths (0.5-1, 1.5-2 and 3-3.5 cm) showed a gradual increase from day 2 to day 8 after when they stabilised. Thus, the Eh readings showed that the sediment profiles were gradually oxidised after the introduction of air.

Table 2: Redox measurements (in mV) of overlying water and sediments during the oxidation period in the top 6-cm sediment layer.

	Depth (cm)	Day 1	Day 2	Day 4	Day 6	Day 8	Day 11
Overlying water	1	-	420	480	460	440	410
Sediments	0.5-1	-	150	190	170	250	250
	1.5-2	-	-150	120	163	340	340
	3-3.5	-	-350	-270	-265	-60	-80

3.1 Evolution of iron

Similar dissolved concentration patterns of increasing Fe were observed for both sediment layers, fig. 3. Within 21 days of anoxic incubation, the Fe concentrations gradually increased ~7- and 5-fold for the top and deep sediments, respectively. It is likely that a reductive dissolution of Fe oxides/hydroxides in the particulate phase occurred in both layers *cf.* Mitsch and Gosselink [9]. Also, in the presence of organic matter, ferric Fe could be removed by reductive reactions *cf.* Salomons and Förstner [11], which, thus, may be part of the picture in our experiment. This is supported by the correlation (r=0.80) found for the simultaneous increase in the Fe- and DOC-concentrations for both sediment layers. The changes in pH (pH 7.2-7.7 in the top and pH 7.2-8.3 in the deeper layers) did not seem to affect the release of Fe during the anoxic incubation, *c.f* fig. 2 and fig. 3.

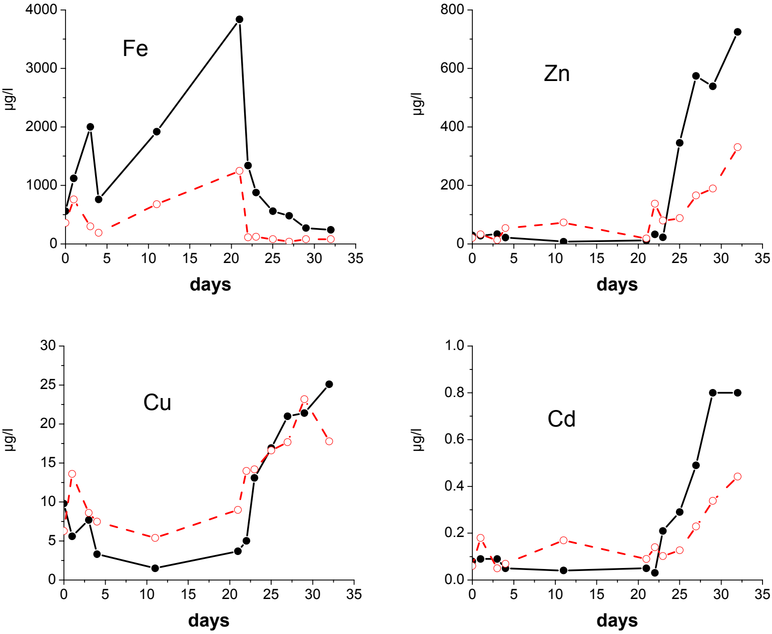

Figure 3: Evolution of Fe, Zn, Cu and Cd in solution during the experiment. Solid lines represent the top layer and dashed lines represent the deeper layer.

In contrast, Fe of both sediment layers (the top and the deeper) was rapidly removed from the solution within 11 days of the aeration, fig. 3. The dissolved Fe concentrations decreased ~16-fold in the top layer and ~22-fold in the deeper layer. Iron in solution is known to form a particulate phase through the oxidation reactions or the bacteria activities in presence of O_2 *cf.* Sunby *et al* [14].

However, opposite result was found in the laboratory investigations conducted by Saulnier and Mucci [12], who observed an outflux of Fe when Fe monosulphides were oxidised. Nevertheless, within a matter of days, Fe in their studies was rapidly removed from the solution, probably due to depletion of Fe monosulphides occurring in parallel with the fast oxidation of ferrous Fe through contact with O_2.

3.2 Evolution of zinc, copper and cadmium

Under the anoxic condition, Zn, Cu and Cd behaved differently to Fe, *i.e.* they were generally low, fig. 3. Zinc ranged ~10 and ~30 µg/l and ~15 and ~70 µg/l for the top and the deeper layers, respectively. The corresponding ranges for Cu were ~2 to 10 µg/l and ~6 and ~14 µg/l, and for Cd ~0.04 to ~0.1 µg/l and ~0.05 to ~0.2 µg/l. It is known that the formation of sulphide metals such as ZnS, Cu_2S and CdS probably occur in the presence of sulphur in the anoxic systems *cf.* Salomons and Förstner [11]. This is consistent with our finding as our sediment layers contain high S contents *cf.* table 1, which may govern the precipitation of the freshly-formed metal sulphides. The pH range (pH 7.2-8.3) within the anoxic conditions in both the top and the deeper layers seems not influence the removal of Zn, Cu and Cd from the solution to the particulate phase, fig. 2 and fig. 3.

In opposite, the concentrations of Zn, Cu and Cd for both the top and deeper layers increased throughout 11 days of the aeration, fig. 3. For the top layer, the dissolved metal concentrations rapidly increased ~6-fold for Cu and ~16-fold for Cd, whereas a progressive increase up to 60-fold was observed for Zn. The same pattern was observed for the deeper layer but at a lower extent, *i.e.* 2-fold increase for Cu, 5-fold for Cd and 17-fold for Zn. This is in agreement with Caetano *et al* [1], who observed a release of the sorbed-Cd and -Cu on Fe sulphides upon oxidation of these sulphides. In another study on re-oxidation of anoxic sediments, Fajtl *et al* [7] recorded a flux of Zn in the water phase as a result of the oxidation of Zn sulphides or its desorption from organic/mineral sorption sites during oxidation. On the other hand, Dutrizac and MacDonald [5] found that ferric iron resulting from the oxidation of ferrous Fe is a powerful oxidising agent. The agent can attack most of metal sulphides like ZnS, Cu_2S and CdS in the presence of bacteria and, as a consequence, the metals Zn, Cu and Cd were generated to the solution. Our experimental results agreed with their findings.

According to the correlation analyses, the pH decrease (from 7.5 to 6.9) in the top layer may have affected the release of Zn (r=-0.72), Cu (r=-0.91) and Cd (r=-0.76). No correlation was found between the pH and the dissolved metal concentrations for the deeper layer. The rapid release of Zn, Cu and Cd from both sediment layers is also likely enhanced by the water flow, which may have increased the diffusion rate of O_2 into the sediment depths as indicated by the evolution of redox conditions in the top layer, table 2. As soon as the sediments were in contact with O_2, the oxidation reactions occurred and the sediment pore-water become the medium for exchanging the resultant released ions from the sediment phase to the water phase *cf.* Saulnier and Mucci [12].

References

[1] Caetano, M., Madureira, M.J. & Vale, C., Metal remobilization during resuspension of anoxic contaminated sediment: short-term laboratory study. *Water, Air, and Soil Pollution*, **143**, pp. 23-40, 2003.
[2] Calmalo, W., Ahlf, W & Förstner, U., Exchange of heavy metals between sediment components and water. *Metal speciation in the environment,* eds J.A.C. Broekaert, S. Gucer, & F. Adams, Springer-Verlarg: Berlin, pp. 503-522, 1990.
[3] Calmano, W., Hong, J. & Förstner, U., Binding and mobilization of heavy metals in contaminated sediments affected by pH and redox potential. *Water Science & Technology*, **28 (8-9)**, pp. 223-235, 1993.
[4] Drever, J.I., *The geochemistry of natural waters: surface and ground water environments*, third edition, New Jersey: Prentice Hall Inc., 1997.
[5] Dutrizac, J.E. & MacDonald, R.J.C., Ferric ion as a leaching medium. *Minerals Science & Engineering*, **6 (2)**, pp. 59-100, 1974.
[6] Eklund, M. & Håkansson, K., Distribution of cadmium, copper and zinc emitted from a Swedish copperworks, 1750-1900. *Journal of Geochemical Exploration*, **58**, pp. 291-299, 1997.
[7] Fajtl J., Kabrna, M., Tichy, R. & Ledvina, R., Environmental risks associated with aeration of a freshwater sediment exposed to mine drainage water. *Environmental Geology*, **41**, pp. 563-570, 2002.
[8] Mayer, J.S., Davidson, W., Sunby, B., Oris, J.T., Lauren, D.J, Förstner, U., Hong, J. & Crosby, D.G., Synopsis of Discussion Session: The effects of variable redox potentials, pH and Light on bioavailability in dynamic water-sediment environments. *Bioavailability: Physical, Chemical, and Biological interactions*, eds. J.L. Hamelink, P.F. Landrum, H.L.Bergman & W.H. Benson, Lewis: USA, pp. 155-170, 1994.
[9] Mitsch, W.J. & Gosselink, J.G., *Wetlands*, third edition, John Wiley and Sons, Inc. USA, 2000.
[10] Salomons, W., Long-term strategies for handling contaminated sites and large-scale areas. *Biogeodynamics of pollutants in soils and sediments*, eds W. Salomons & W.M. Stigliani, Springer-Verlarg: Berlin, pp. 1-30, 1995.
[11] Salomons, W. & Förstner, U., *Metals in the Hydrocycle*, Springer-Verlarg: Berlin, 1984.
[12] Saulnier, I. & Mucci, A., Trace metal remobilization following the resuspension of estuarine sediments: Saguenay Fjord, Canada. *Journal of Applied Geochemistry*, **15**, pp. 203-222, 2000.
[13] Svensson, B.H. & Rosswall, T., In situ methane production from acid peat in plant communities with different moisture regimes in a subarctic mire. *Oikos*, **43**, pp. 341-350, 1984.
[14] Sunby, B., Anderson, L.G., Hall, P.O.J., Iverfeldt, Å., van der Loeff, R.M.M., Westerlund, S.F.G., The effect of oxygen on release and uptake of cobalt, manganese, iron and phosphate at the sediment-water interface. *Geochimica Cosmochimica Acta*, **50**, pp. 1281-1288, 1986.

[15] Swedish Environmental Protection Agency (SEPA), Environmental Quality Criteria for lakes and watercourses www.internat.naturvardsverket.se/.
[16] SS 028112: Determination of suspended solids in wastewater and their residue on ignition. Swedish Standard Institute, 1996.
[17] SS-EN 12880: Characterization of sludge-determination of dry residue and water content. Swedish Standard Institute, 2000.
[18] SS-EN ISO: 15586. Water quality-Determination of trace elements using atomic adsorption spectrometry with graphite furnace. Swedish Standards Institute, 2004.

Delayed impacts of land-use via groundwater on Lake Taupo, New Zealand

J. C. Hadfield[1], U. Morgenstern[2] & J. J. Piper[3]
[1]*Environment Waikato, Hamilton East, New Zealand*
[2]*GNS Science, Avalon, Lower Hut, New Zealand*
[3]*Tonkin and Taylor Ltd, Auckland, New Zealand*

Abstract

The near pristine quality of water in Lake Taupo has begun to deteriorate, largely as a result of farming. Groundwater investigations were undertaken to estimate potential future land-use impacts and the lag in effects. The lake is a sink for groundwater migrating indirectly via baseflow dominated streams and to a lesser extent by direct seepage. Land-use impacts are increasing as contaminated water progressively replaces older, higher quality groundwater.

The age of groundwater in Lake Taupo catchment was estimated by measuring the cosmogenic isotope tritium and atmospheric trace gases CFC and SF_6. Mean residence times of samples ranged from about 20 to 75 years. Higher nitrogen concentrations occur in groundwater with a greater fraction of water recharged since farm development some 40 years ago.

Numerical groundwater modeling predicts nitrogen mass loading to the lake from current land-use will continue to increase for a substantial period of time (> 100 years). This lag relates to the considerable time required to replace old pristine groundwater with nitrogen enriched water from farming. Modeled nitrogen flux via groundwater of some 300 tonnes annually may potentially be expected, with 25,000 tonnes stored in the groundwater system. A proposed initiative to reduce manageable nitrogen loading from the catchment by 20% would be a useful mitigation measure.

Keywords: groundwater, diffuse contamination, dating, lag, nitrogen, model, Lake Taupo.

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1 Introduction

Lake Taupo is a large (622 km^2) oligotrophic lake in the central North Island, New Zealand, which occupies a volcanic collapse caldera, fig. 1. Although water quality in the lake is very high, there is evidence of decline (Gibbs [1]). Nutrient loads from inflowing streams have also increased (Vant and Smith [2]). Historically the lake has extremely low levels of nitrogen which has limited the growth of aquatic plants in its waters. Much of the traditional scrubland around the lake has, however, been converted for agricultural use over about the last 40 years. This has led to increases in nitrogen being leached and transported to the lake. The northern and western catchments are the primary focus of study given their greater potential for land-use intensification (Ministry of Agriculture [3]).

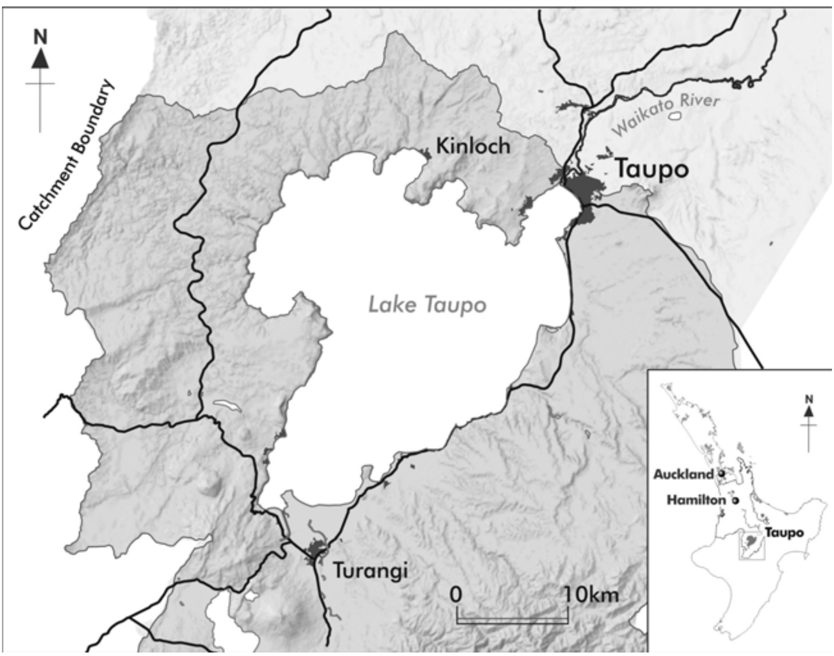

Figure 1: Lake Taupo study area.

Groundwater is the primary link for transport of nutrients derived from land-use activities to the lake. Streams in the study area are baseflow dominated (Schouten et al. [4]). It is important to predict nitrogen mass loading from groundwater to the lake and assess the lag in full land-use impact for lake management. There is strong community support for protection of the lake, which is a national icon. Maintaining ecological values for tourism is of greater economic benefit than risking degradation through further pastoral development. There is a current proposal to reduce nitrogen loading to the lake by 20%.

2 Scope and approach

Hydrogeologic study of the northern and western catchments included investigation of the geologic setting, aquifer hydraulic characteristics, water chemistry and stream-groundwater interaction. Of particular focus in this paper is information derived from groundwater age dating and numerical modeling of the northern catchment.

Numerous shallow monitoring wells were constructed to augment available wells in the sparsely developed area. Samples were taken for water chemistry and age dating, and pumping tests were carried out. The interaction between groundwater and streams was investigated by flow gauging, water quality sampling and tritium dating. Tracer injection tests were also undertaken as an initial study of denitrification. Direct groundwater seepage to the lake was studied at two locations, both at the shoreline and subsurface. The information derived was used to calibrate model predictions of mass transport and lag.

3 Results

3.1 Hydrogeology

The local geology which comprises young (<0.4 Ma) locally derived rhyolitic pyroclastic formations can be broadly divided into three main groupings. The Whakamaru Group ignimbrites toward the west are poorly to moderately welded with low fracture density but flow is nevertheless generally fracture controlled. To their east is a more complex grouping considered loosely as 'rhyolitic pyroclastics'. These are both overlain by unwelded Oruanui Ignimbrite, which varies in thickness up to some 30 m Hadfield et al. [5].

Hydraulic conductivity for these simplified groups was estimated from pumping and slug testing, table 1. Leakage estimates from pumping tests (which vary from 0.02 to $1x10^{-4}$ d^{-1}), measured vertical head differences (3 to 80 m) and groundwater dating results, indicate variable connection between hydraulic units. Aquitards, such as paleosols, can however be locally important. Localised Whakaroa Rhyolite in the Kinloch area exhibits highly variable fracture dependent permeability ranging up to some 120 m d^{-1}.

Table 1: Summary of aquifer hydraulic conductivity estimates (m d^{-1}).

Formation	Median	Mean	Std. deviation	Minimum	Maximum
Whakamaru	0.01	0.26	0.66	0.007	2.02
Rhyolitic	0.93	2.15	4.03	0.008	17.3
Oruanui Ignimbrite	0.28	2.92	4.79	0.03	13.9

Groundwater flow in the study area is consistent with topography although more subdued, fig. 2. The catchment divide is critical for management implications. There is no discrepancy evident between surface and groundwater catchments. A recharge regime exists generally throughout the area with a highly significant relationship between depth to static water level and well intake depth. The lake acts as a sink for groundwater, which is recharged from rainfall in the catchment.

3.2 Streams and catchments

The vast majority of stream reaches gain flow from groundwater. Although streams capture most groundwater before entering the lake, direct seepage makes significant contribution in some northern sub-catchments e.g. Mapara and Whangamata (about 80%) (Piper [6]).

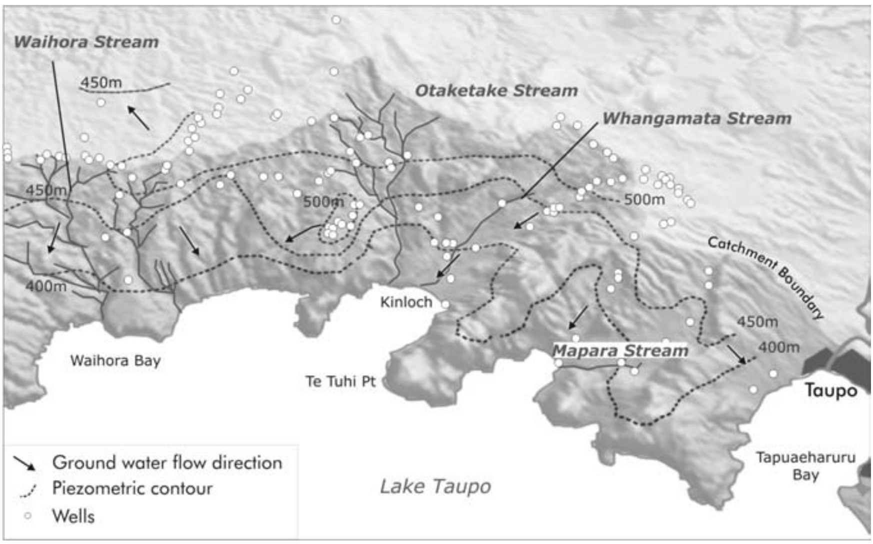

Figure 2: Lake Taupo’s northern catchment and groundwater flow.

A substantial component of direct groundwater discharge to the lake from the Whangamata catchment was not accounted for in shallow beach seepage measurement (Hector [7]). Subsequent deeper investigation by divers found seepage zones in the lake to depths of about six metres accounted for this discrepancy. Algal mats associated with these seepage zones are inferred to take up much of the nitrogen from these inflows (Gibbs et al. [8]).

3.3 Water chemistry

Groundwater from 44 wells sampled for analysis of nutrients and major ion chemistry in 2000, showed relatively uniform sodium bicarbonate dominated groundwater typical of rhyolitic formation. There was, however, clear evidence of land-use impacts with nitrate-N commonly elevated above ambient condition, considered to be less than 1 g m^{-3}. Very high nitrate-N (~36 g m^{-3}) at one well was found to result from point source contamination from a nearby woolshed.

Anaerobic or poorly aerobic conditions are indicated at about 15% of sites by the nitrate ammonia couple, presence of dissolved iron and manganese and Eh pH measurement. Investigation of denitrification by injecting nitrate and bromide into five available shallow piezometers, using the method of Trudell et al [9], inferred active denitrification may be rare due to a lack of available carbon. One site where denitrification was apparent is considered exceptional given the rare presence of peat (Environment Waikato unpublished results).

3.4 Dating

The age of groundwater in the northern and western Lake Taupo catchments was estimated by measuring the cosmosgenic isotope tritium (half life 12.3 years) in samples, as well as atmospheric trace gases, CFCs and SF_6), which can resolve ambiguity. Age interpretation of groundwater depends on mixing processes underground. Groundwater and stream samples comprise a mixture, rather than a discrete age, reflecting variable flowpaths Maloszewski and Zuber [10]. It is useful to estimate not only the mean residence time (MRT) but also the fraction of water recharged since farming was established in the Taupo area in the 1960s (% young fraction (YF)). An indication of potential future nitrogen concentration may be obtained by considering an increase inversely proportional to the % YF (Morgenstern et al [11]).

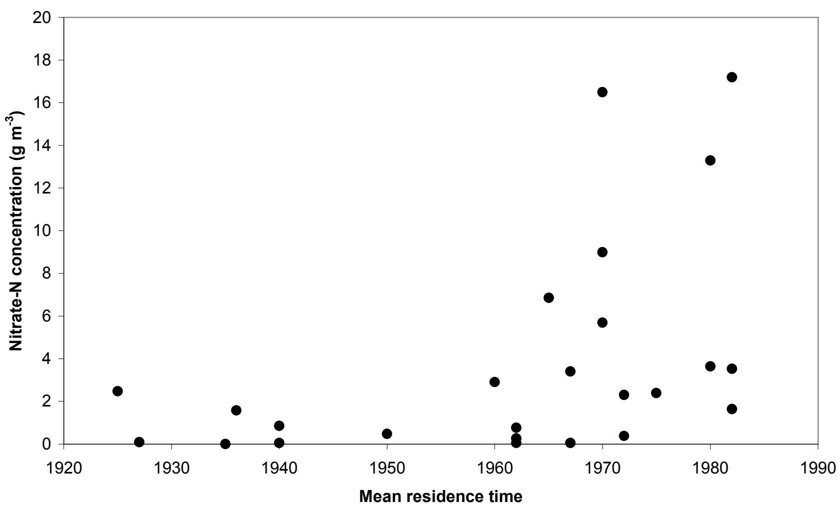

Figure 3: Nitrate-N concentration versus mean residence time (expressed as mean recharge date).

A total of 25 groundwater samples were analysed for tritium, CFCs and some for SF_6 tracer concentrations. Mean residence times ranged from about 20 to

75 years. Higher nitrate-nitrogen concentrations were shown to occur in younger groundwater, fig. 3. Not all of the young waters, however, have high nitrogen concentrations as the farming influence is not uniform.

Tritium samples were also collected from 11 streams mouths in the northern and western sub-catchments in 2001/02. MRT ranged from about 30 to 80 years with the oldest (and lowest% YF) occurring in the 'rhyolitic pyroclastic', north-eastern catchments e.g. Whangamata and Mapara. Streams in the western, Whakamaru Group, sub-catchments, which have steeper relief and higher rainfall were found to have younger MRTs. Potential future nitrogen loads from these streams are approximated on the basis of the percentage of farming input to come Vant and Smith [2]. Tritium sampled at multiple sites along streams within nine selected sub-catchments showed little age variation within catchments, inferring uniform drainage patterns (Piper [6]).

3.5 Modeling

A numerical groundwater model of the northern catchment was constructed using Modflow to simulate flow and MT3D for contaminant transport. Steady state flow and transient transport simulation were used to model future land-use impacts and consider management options

A three-layer model was used to enable vertical head distribution and vertical migration of contaminants to be explored. Model boundaries included a no flow catchment boundary, constant head lake boundary and river leakage to the Mapara, Whangamata, Otaketake and Waihora Streams. Water use was considered relatively insignificant and not included.

Inputs to the model included effective rainfall and solute (nitrogen) leaching from the ground surface. Recharge is estimated from available rainfall data, which shows a strong orographic influence. A uniform portion of evapotranspiration loss is assumed across the domain with resulting recharge ranging from 450 to 800 mm y^{-1}. Contaminant is added as spatially variable diffuse loading. This is based on characteristic nitrogen loading from mapped land-use with an assumed inverse linear relationship with effective rainfall. Work by Green and Clothier [12] indicates that a linear approximation may be reasonable.

Steady state flow conditions were calibrated with groundwater levels in 54 observation wells using the inverse method with a normalized root mean square of <8% achieved. Flow was also matched in the four streams. Calibration of the flow model was most sensitive to changes in recharge, horizontal conductivity of deeper formation (layer 3), vertical hydraulic in the middle layer and horizontal conductivity in shallow formation (layer 1) respectively.

Model simulated groundwater velocities ranged from about 0.02 to 0.3 m d^{-1} and some groundwater travel times to the lake are in excess of 100 years (fig. 4). Horizontal hydraulic conductivity inputs range from 0.01 m d^{-1} to 100 m d^{-1} in the Kinloch area, consistent with field estimates.

Nitrate nitrogen was modeled as a conservative solute primarily because of the lack of observed denitrification in groundwater in the northern catchment.

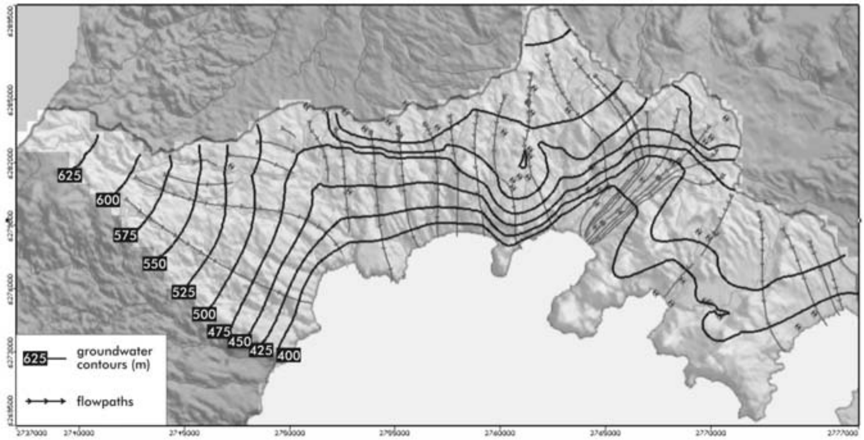

Figure 4: Modeled piezometric contours and flowpaths (10 year time steps).

Some denitrification is, however, inferred by water chemistry at some wells and at some wetland margins.

Ambient conditions were simulated by first running the model for 800 years with a loading rate of 2 kg N ha^{-1} y^{-1} estimated for native vegetation. Ambient recharge concentration ranged from 0.25 g m^{-3} to 0.44 g m^{-3}. Typical loading rates used for drystock and dairy land-use were 12-14 kg N ha^{-1} y^{-1} and 40-50 kg N ha^{-1} y^{-1} respectively Ledgard [13]. This translates into recharge concentrations ranging up to 7.3 g m^{-3}.

Modeled concentrations after 35 years of simulated current land-use were compared with observations from 31 wells and historic water quality trends in the Mapara and Whangamata Streams. Highest nitrate concentrations of about 7 g m^{-3} were predicted in an area of dairy land use just west of Kinloch. Calibration using simplified land use data produced a normalized root mean square of <16%. Increasing dairy farm loading in the Kinloch area improved the fit to <14% and removing four wells with groundwater chemistry indicating likely anaerobic condition improved the fit further to <12%. Although further improvement in calibration is achieved by manipulation of recharge concentration, emphasis was given to scenario and sensitivity testing using simplified land-use loading. The transport model was most sensitive in order of importance to changes in recharge concentration, conductivity, total porosity, recharge rate and dispersivity.

On the basis of the above assumptions, modelled ambient conditions indicate some 66 tonnes of nitrogen discharging from northern catchment groundwater annually with about 5,000 tonnes stored in aquifers. After 35 years of current farming about 150 tonnes of nitrogen would discharge annually with about 11,500 tonnes being stored in aquifers, fig. 6. Modeled continuation of land-use at the current intensity suggests nitrate-N transport from groundwater would equilibrate after about 250 years. Assuming conservative transport, some

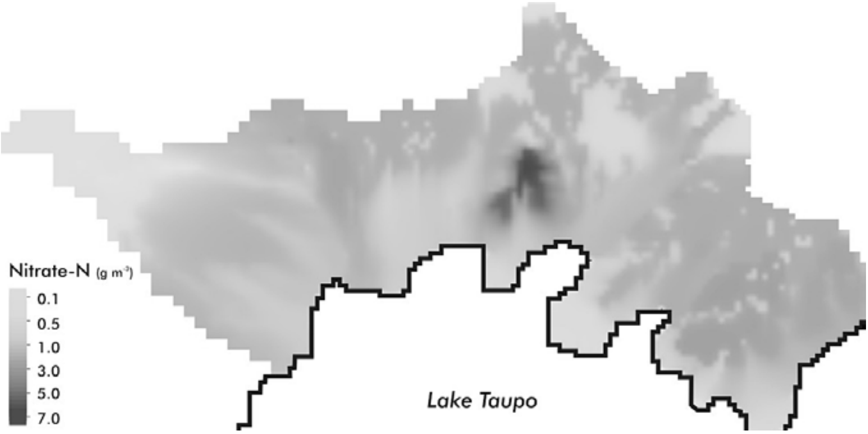

Figure 5: Nitrate-N concentration (g m^{-3}) in shallow groundwater based on 250 years of current land-use.

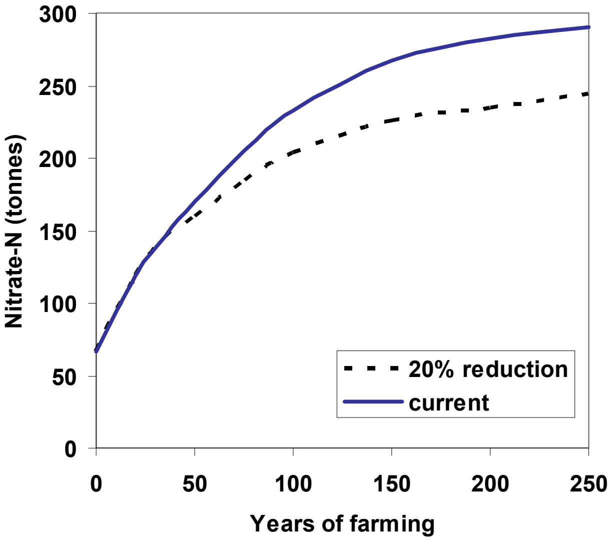

Figure 6: Mass flux of nitrate-N from groundwater.

300 tonnes of nitrate-N would migrate annually and 25,000 tonnes would be present in groundwater within the model domain.

The proposed policy initiative to reduce the manageable nitrogen load by 20% was also modeled by instantly reducing current land-use loading after 35 years. The resulting predicted curves show mitigated increases in nitrate-N loading from the northern catchment (fig. 6). Temporal concentration trends at individual wells are more variable, dependent on the extent to which concentrations had already stabilised.

Stream nitrate-N concentrations were calculated from flow budgeting and mass flux of solute. Predicted concentrations for Whangamata Stream are of similar magnitude to those observed. The Mapara Stream concentration, by contrast, is over-predicted, which may be the influence of wetland denitrification in the head of that stream.

Modeling predicts a small immediate response in nitrate-N concentration to a 20% reduction in manageable nitrogen load to the Whangamata Stream but a longer-term increase. Nitrogen solute added to the model before and after the 20% reduction was differentiated to explain how an instantaneous response can occur in a stream with a mean residence time of about 80 years. This indicated that more than 10% of the nitrogen in the stream after one year is from new addition with there being about 45% after 45 years. The latter is about 20% higher than interpreted from tritium measurement.

4 Discussion

Models are inevitable simplifications of reality. Assumptions such as instantaneous farming development would accelerate effects, whereas exclusion of some streams from the model conversely increases average flowpaths and travel times. Although land-use loading was simplified for model input, a more detailed approach by Brown et al. [14] totaled within 10%. Stream uptake of nitrogen is an important attenuation factor not addressed in this modeling but may be up to 50% Elliot and Stroud [15]. The combined application of dating and modeling is, however, effective in revealing the approximate extent of, and substantial delays in, land-use impacts. Modeling is particularly helpful in demonstrating the continuing slow build up of nitrogen mass in, as well as flux from, the groundwater system.

It has been estimated that total annual mass flux of nitrogen to Lake Taupo is about 1,200 tonnes. About 56% of this is from unmodified inputs such as rainfall and undeveloped land. A further 7% is from water imported to the catchment for hydroelectric power generation. The approximately 470 tonnes (37%) remaining is the manageable nitrogen load. The majority of this (about 31% or 58 tonnes) is from non-dairy pasture; about 5% from dairy farming and 2% from urban wastewater. Nitrogen load to the lake from current farming is expected to increase by at least 20% and up to 80% before equilibrium would be established according to Vant and Smith [2].

A proposed 20% decrease in manageable nitrogen load is proposed by regional government. A fund has been established to buy farmland to convert from pastoral to forestry or other low nitrogen leaching use. Landowners in the area would have their current nitrogen output capped and would only be able to increase this by offsetting elsewhere through a nitrogen trading system. Even before this initiative was promulgated, active conversion of farms to dairying was effectively stopped by market concerns about future land-use constraints.

5 Conclusions

Water quality in Lake Taupo is starting to decline, essentially as a result of farming in its catchment. The lake, which is nitrogen limited, acts as a sink for groundwater migrating via baseflow dominated streams, as well as direct seepage. Elevated nitrate concentrations are part of evidence of land-use impacts on groundwater.

Tritium, CFC and SF_6 dating of groundwater show higher nitrogen concentrations generally occur in groundwater receiving more recent recharge from farming, which developed in the area some 40 years ago. Mean residence times of groundwater samples analysed range from about 20 to 75 years.

Numerical modeling shows there is a considerable time lag (>100 years) between land-use change and maximum nitrogen flux from groundwater. This reflects the time required to replace old pristine groundwater with nitrogen enriched water from farming. Annual nitrogen discharge from groundwater in the northern catchment is estimated from the modeling to be some 66 tonnes under ambient conditions, nearly 150 tonnes under current land-use and potentially some 300 tonnes if present farming activities continue. The amount of nitrogen stored in the groundwater system would also increase from about 5,000 tonnes under ambient conditions to some 11,000 tonnes under current land-use and potentially 25,000 tonnes once the full effects of current farming are realised.

Land-use effects in the Lake Taupo catchment, although considerable, are insidious, due to substantial lags and subtle changes. Groundwater modeling and dating are useful in revealing long term trends and impacts. Modeling indicates that a proposed initiative to reduce the manageable nitrogen load from the lake catchment by 20% would effectively mitigate the extent of increase otherwise expected in the northern catchment.

References

[1] Gibbs, M.M., 2006: Lake Taupo long-term monitoring programme 2004-2005, Environment Waikato Technical Report 2006/30, Environment Waikato, Hamilton.

[2] Vant, B. and Smith, P. 2004: Nutrient concentrations and water ages in 11 streams flowing into Lake Taupo. Environment Waikato, Hamilton. Technical Report, 2002/18R. 20 p.

[3] Ministry of Agriculture, 1997: Impacts of dairy conversion, Lake Taupo District. MAF Policy Paper 1997/9. Ministry of Agriculture, Wellington

[4] Schouten, C.J., Terzaghi, W., Gordon, Y., 1981: Summaries of water quality and mass transport data for the Lake Taupo catchment, New Zealand. Water and Soil Miscellaneous Publication 24, Ministry of Works, and Development, Wellington.

[5] Hadfield, J.C., Nicole, D.A., Rosen, M.R., Wilson, C.J.N. and Morgenstern, U., 2000: Hydrogeology of Lake Taupo Catchment – Phase 1. Environment Waikato, Hamilton

www.witpress.com, ISSN 1743-3541 (on-line)

[6] Piper, J.J., 2004: Surface water / groundwater interaction and catchment influence on waters entering Lake Taupo, New Zealand, Unpublished MSc thesis, Victoria University of Wellington.
[7] Hector, R.P., 2004: Investigation of direct groundwater and nutrient seepage to Lake Taupo. Unpublished MSc thesis, University of Waikato.
[8] Gibbs, M.M, Clayton, J. and Wells , R., 2005: Further investigation of direct groundwater seepage to Lake Taupo. NIWA report to Environment Waikato. 27 pp.
[9] Trudell, M.R., Gillham, R.W. and Cherry, J.A., 1986: An in-situ study of the occurrence and rate of denitrification in a shallow unconfined sand aquifer. Journal of Hydrology 83, 251-268.
[10] Maloszewski P, Zuber A, 1982: Determining the turnover time of groundwater systems with the aid of environmental tracers: I.: Models and their applicability, Journal of Hydrology., 57
[11] Morgenstern U, Reeves R, Daughney C, and Cameron S, 2004: Groundwater age, time trends in water chemistry, and future nutrient load in the Lakes Rotorua and Okareka area, GNS client report 2004/17.
[12] Green, S., and Clothier B., 2002: Modeling the impact of dairy farming on nitrate leaching in the Lake Taupo catchment. HortResearch report to Environment Waikato.
[13] Ledgard, S., 2000: Sheep and beef farming systems in the Lake Taupo catchment: Estimates of the effect of different management practices on nitrate leaching. Report for Environment Waikato. AgResearch.
[14] Brown, L., Hill, R.B., Singleton, P.L., (2002) Modelling surface water nitrogen. Proceedings of the New Zealand Soil Science Society Conference, Wellington December 2002.
[15] Elliot, A.H., and Stroud, M.J., 2001: Prediction of nutrient loads to Lake Taupo under various landuse scenarios. National Institute of Water and Atmospheric Research Ltd, 52 p.

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Effect of preozonation and prechlorination on total organic carbon removal in surface water treatment in Iran

A. Torabian[1], A. A. Ghadimkhani[1], A. Rashidi Mehrabadi[2], M. Shokouhi[1] & R. Heidarzadeh[1]
[1]*University of Tehran, Iran*
[2]*Power and Water University of Technology, Tehran, Iran*

Abstract

In drinking water treatment, prechlorination and preozonation are often applied in order to control micro-organisms and taste and odour causing materials, which may influence organics removal by pre-oxidation and adsorption.

Using commercial and natural water humid substances, the positive effect of prechlorination and preozonation as an aid to coagulation-flocculation of these compounds were confirmed by the removal of total organic carbon (TOC). These experiments were conducted at bench-scale through a series of jar tests using different pH, coagulant dosage and TOC concentration of approx. 4, 8 and 12 mg/l. In addition to TOC removal, the existence of an optimum preozonation dose and an optimum prechlorination dose were also confirmed. Experiments show that prechlorination and preozonation can improve coagulation and flocculation depended on TOC concentration of influent surface raw water. The results demonstrate a differential effect between prechlorination and preozonation on removal of TOC. Preozonation showed a positive effect on a system with low influent TOC and very low molecular weight (noncolloidal) humic substances.
Keywords: *prechlorination, preozonation, TOC, surface raw water, preoxidation.*

1 Introduction

In the early 1900s, the United States' drinking water industry drastically reduced the number of fatal waterborne disease outbreaks when it began chlorinating

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drinking water. Some ninety years later, the United States Environmental Protection Agency imposed stringent regulations governing chlorination of drinking water supplies because this same chemical, which had saved so many lives, produced suspected carcinogens in the presence of naturally occurring organic matter (Letterman [11]).

Two groups of these potential carcinogens are trihalomethanes (THMs) and haloacetic acids (HAAs). Both form when chlorine reacts with natural organic matter in raw water. According to the 1995 Community Water Systems Survey conducted by the US Environmental Protection Agency (USEPA), 14.2% of surface water treatment systems servicing a population of 50,001-100,000 are using ClO2 as a predisinfectant compared to 47.5% using chlorine, 15.5% using chloramines, and 5.4% using ozone (USEPA [8]).

Although ozone can destroy phenolic compounds structure effectively, it is not economical to reduce the TOC to an acceptable level using single ozonation process due to high ozonation cost [10].

Unfortunately, the anticipated maximum contaminant levels for haloacetic acids (HAAs) and trihalomethanes (THMs) of certain source water may not be using enhanced coagulation (Crozes et al. [5]). Thus, the conventional water treatment process with coagulation/sedimentation and filtration is unable to remove a significant amount of natural organic matter (NOM) [14].

Because of the formation of DBP, more advanced technologies including oxidation, adsorption, and membrane filtration were introduced to remove NOM prior to disinfection process. Ozone has been used as a strong oxidant for years, and several researches (Amy et al. [1], Jacangelo et al. [7]) have shown that ozonation prior to chlorination can lower formation potential of THM and HAA [15].

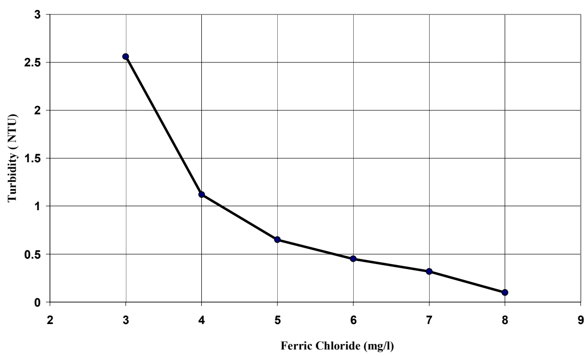

Figure 1: Coagulant only jar test result for turbidity removal of humic acid synthetic water.

It has been reported that ozonation could convert NOM from humic substances to non-humic fractions and from higher- to lower-MW fractions (Amy et al. [1] Owen et al. [13]). In the preozonation process, ozone is added to source water prior to coagulation. The role of ozone acts as an oxidant sometimes as a coagulant-aid. The dosage of preozonation to achieve the best coagulation ranges from 0.4 to 0.8 mg O3/mg carbon [4].

A study by Edwards and Benjamin examined the effect of preozonation on between coagulant and natural organic matter (NOM) and found that increasing the ozone dosage led to increase in metal residuals for both Iron and aluminium-based coagulants. As the zeta potential increased (became less negative) with increasing coagulant dosage, less metal residual was detected [6].

As the ozonation dosages increased, however, the flocs became more highly charged (more negative), and thus the critical coagulants (CCC) was increased for increasing ozone dosage. TOC removal was sometimes increased and sometimes decreased, depending on ozonation dosage and coagulant dosage. However, the amount of TOC removal by coagulation decreased with increasing ozone dosage because of transformation of NOM into more hydrophilic, less absorbable molecules. Overall TOC removal slightly increased with increasing ozone dosage at very high and low coagulant dosages but decreased at moderate coagulant dosages [6].

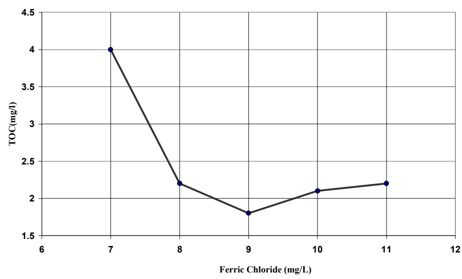

Figure 2: Coagulant only jar test result for TOC removal of humic acid synthetic water.

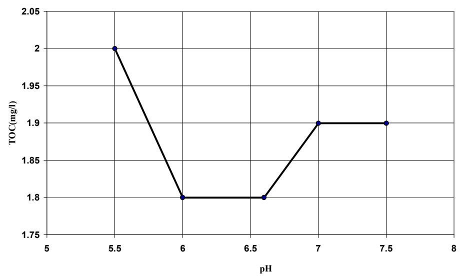

Figure 3: Coagulant only jar test result for turbidity removal of humic acid synthetic water.

Edwards and Benjamin concluded that at constant pH, the dosage of metal salt coagulants required for optimal particulate removal in the presence of NOM increased with increasing ozone dosage. For organic polymer coagulants, however, the opposite proved true [6].

Becker found that oxalate (a common ozonation by-product) had an adverse effect on coagulation and filtration of turbidity and TOC when alum was used as the only coagulant [3].

Schneider and Tobiason concluded that when alum is used as a coagulant, preozonation leads to increases in settled water turbidity, TOC removal, and DOC at the conditions tested (0.5 to 0.8 mg/l ozone at bench scale). Also, they concluded that when cationic polymer is used as a coagulant, preozonation leads to increases removal of turbidity, TOC, and DOC. When PACl is used as a coagulant, preozonation leads either to increases or decreases in turbidity NOM removal [12].

The objectives of this research work are intended to: (1) prechlorination effect on TOC removal, (2) preozonation effect on TOC removal. (3) Comparing between prechlorination and preozonation effect on TOC removal. (4) Varying contact time effect of preozonation and prechlorination effect on TOC removal.

2 Materials and methods

2.1 Waters samples and materials

Clay, kaolin, humic acid in powder form used for simulation of raw water with desired turbidity and TOC. Commercial 60% chlorine used for prechlorination, which is used in the Tehranpars Water Treatment Plants. The humic acid was purchased in powder form of U. S. Acros Company. The ozonator model COG of Arda Company used for preozonation.

2.2 Analysis methods

Experiments were carried out at room temperature in a batch mode. Bench scale preozonation and prechlorination was performed. First, the raw water with turbidity of 20 NTU simulated with clay and kaolin. Then, desired TOC concentration (4, 8, and 12 mg/l) simulated with adding of humic acid in powder form. Commercial 40% ferric Chloride (that is used in the Tehranpars Water Treatment Plants) used as a coagulant. Standard jar test procedures used to evaluate ferric Chloride requirements and the primary water quality parameters.

The model water rapidly mixed for 2 min. During this period, the pH of the water adjusted to the desired range by addition of 0.02 N sulphuric acid or 0.1 N sodium hydroxide. The beakers transferred to a six-place jar-test apparatus, and the water mixed at 35 rpm for 30 min. The beakers removed from the jar-test apparatus, and contents allowed settling quiescently for 30 min. The 500-ml aliquots treated with different coagulant dosage at the same pH.

According to Primary Drinking water Standards of USEPA (1997), Turbidity must be less than 0.5 NTU in 95% of samples per month. Thus, the ferric chloride optimum dosage for removal of turbidity in compliance with standard obtained for TOC of 4 mg/l. According to Primary Secondary Water Standards of USEPA (1997), TOC of filtered water should be less than 2 mg/l (or less than 2.0 SUVA) [9].

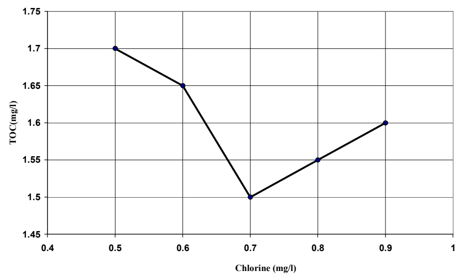

Figure 4: Chlorine effect on TOC removal.

Thus, the ferric chloride optimum dosage for removal of TOC in compliance with standard obtained for TOC of 4 mg/l. Then optimum pH identified. In alternative 1 the optimum coagulant dosage and optimum pH determined 9 mg/l and 6.0, respectively. In alternative 2, TOC removal investigated with prechlorination. Thus, the chlorine optimum dosage for maximum removal of TOC determined. In alternative 3, TOC removal investigated with preozonation and ozone optimum dosage for maximum removal of TOC determined.

Finally, 3 alternatives compared with another. TOC analyzed using a TOC analyzer according to standard method [2].

3 Results and discussion

3.1 Results

3.1.1 No preoxidation, prechlorinatiom and preozonation effects on TOC removal of 4 mg/l

Results of ferric chloride jar-test experiments for turbidity removal are shown in figure 1. As illustrated in figure 1, the optimum dosage for turbidity removal is 6 mg/l. Figure 2 demonstrates the optimum dosage for TOC removal. In general, removal of both turbidity and TOC in the Jar test results was good. The optimum dosage for turbidity removal is 9 mg/l.

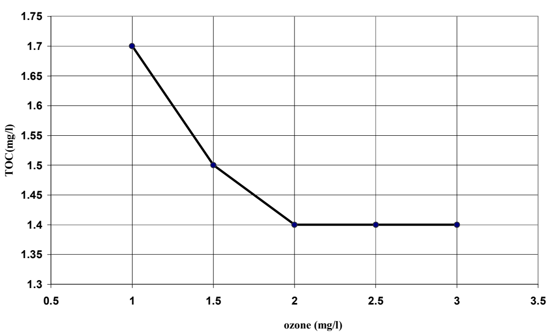

Figure 5: Ozone effect on TOC removal.

Figure 3 demonstrates that the optimum range of pH for TOC removal is 6-6.5. The addition of chlorine before coagulation with ferric chloride increased removal of organic matter. Figure 4 shows results for Jar test experiments conducted at pH 6. Finally, the addition of ozone before coagulation with ferric chloride increased TOC removal. Figure 5 shows results for Jar test results.

As illustrated in figures 4 and 5, the optimum dosage of Prechlorinatiom and Preozonation are 0.6 and 2 mg/l, respectively.

3.1.2 No preoxidation, prechlorinatiom and preozonation effects on TOC removal of 8 and 12 mg/l

The results are as same as 3.1.1. Thus, the results listed at tables 1 and 2. Also, figure 7 illustrated the results.

3.2 Discussion

3.2.1 No preoxidation

Data taken from the entire jar test results in which ferric chloride used as a coagulant that is shown that required coagulant dosage for TOC removal increases with TOC increment. Figure 8 demonstrates relation between required optimum dosage of ferric Chloride and TOC removal.

Lowering pH to range of 6-6.5 causes to increase removal efficiency of TOC.

3.2.2 Prechlorination, coagulation

In general, when chlorine added to humic acid synthetic waters at pH 6.5, removal of turbidity and organic matter slightly increased. The results of analysed data are presented in figure 4.

Lowering pH during enhanced coagulation increases the disinfection efficiency of chlorine. The US Environmental Protection Agency (USEPA) contact timetables for chlorine reflect this change in disinfection efficiency relate to pH.

3.2.3 Preozonation, coagulation

The addition of ozone before coagulation with ferric chloride increased removal of turbidity and organic matter. These effects appeared after only 5 min of ozone contact time before coagulation addition. No definitive trend was observed with ozone contact time.

A comparison of ozonation and no ozonation shows that the addition of ozone had a beneficial effect on TOC removal.

A comparison of no ozone and applied ozone dosage of 2 mg/l dosage shows that the 2 mg/l dosage had a beneficial effect on turbidity removal and TOC removal. TOC removal was the reduction in negative surface charge following ozonation. The charge-neutralizing effect of preozonation may act in concert with the cationic polymer to decrease the ferric chloride to turbidity removal, thus providing “excess” ferric chloride to aid in the removal of TOC that would normally not be removed. The removal of the low–molecular-weight hydrophilic organic species is likely attributed to increased specific interactions between the organic materials and ferric chloride, followed by hydrophobic expulsion of

NOM-ferric chloride. The fact that the change in EPM seemed to occur on approximately the same time scale as the changes observed in the coagulation experiments supports this argument.

As illustrated in table 2, specific ozone dosage of 0.5 mg per mg TOC applied in the removal of TOC.

The reduction of TOC will decrease the demand and decay rate use result of the decrease in reactive material. Both of these reductions result in a lower ozone dosage required to achieve equivalent disinfection efficiency. Ozonation, when applied before enhanced coagulation, can sufficiently change the remove low molecular weight, non humic fraction.

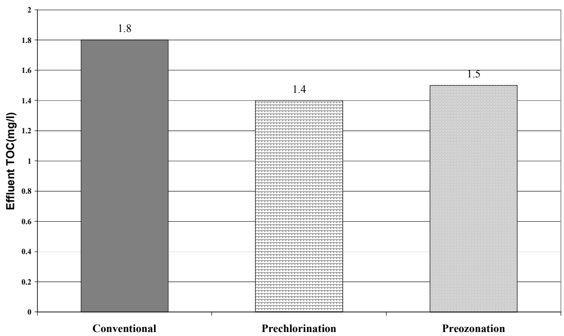

Figure 6: A comparison of conventional (no preoxidation), preozonation and prechlorination.

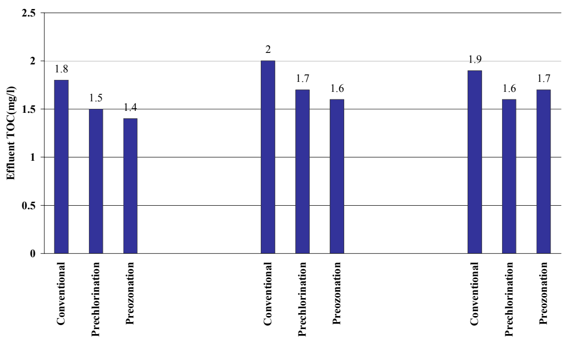

Figure 7: A comparison of conventional (No Preoxidation), Preozonation and Prechlorination at TOC of 4, 8, and 12 mg/l (left to right).

3.2.4 Comparison of settled water TOC at no preoxidation, prechlorination and preozonation

The authors observed detrimental effects of non oxidation, prechlorination, and preozonation on TOC removal when ferric chloride used as a coagulant.

As illustrated in figure 6, a comparison of settled water TOC at no preoxidation, prechlorination, and preozonation shows that addition of ozone had a beneficial effect on both turbidity and TOC removal. A comparison of ozone and chlorine shows that ozone had a beneficial effect on both turbidity and TOC removal in low TOC concentration of influent water. Adversely, in high TOC concentration, chlorine had a beneficial effect on both turbidity and TOC removal. But, the effect of prechlorination and preozonation on coagulant-particle-NOM interactions are subtle and complex. Thus, use of prechlorination and preozonation as a coagulant aid will not likely be its primary application.

Table 1: Jar test results for various TOC concentrations.

Optimum Dosage for turbidity removal	6	30	60
Optimum Dosage for TOC removal	9	40	70
Optimum pH	6.5	6.5	6.5

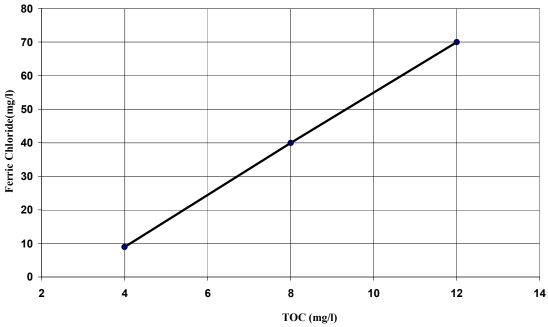

Figure 8: Required optimum ferric Chloride dose for TOC removal.

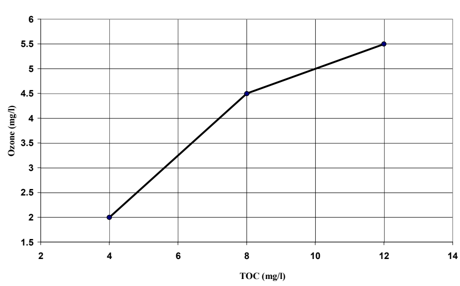

Figure 9: Required optimum ozone dose for TOC removal.

Table 2: Maximum TOC removal with No Preoxidation, Prechlorination, and Preozonation.

Alternative	TOC of 4 mg/l	TOC of 8 mg/l	TOC of 12 mg/l
No Preoxidation	1.8	2	1.9
Prechlorination	1.4	1.6	1.7
Preozonation	1.5	1.7	1.6

4 Conclusions

Based on the results of the described experiments and condition; the authors drew the following conclusions.

- When ferric chloride used as a coagulant, prechlorination and preozonation lead to decrease in settled water turbidity and TOC at the conditions tested.
- When ferric chloride used as a coagulant, required dosage increased with influent TOC increment.
- When ferric chloride used as a coagulant, preozonation leads to increase removal of turbidity and TOC. These increases are apparent after only a few minutes of contact time.
- Specific ozone dosage of about 0.5 mg per mg TOC (applied ozone dosage of approximately 2, 4.5, and 5.5 mg/l) aids in the removal of TOC.
- Ozone had a beneficial effect on both turbidity and TOC removal in low TOC concentration of influent water. But, in high TOC concentration, chlorine had a beneficial effect.
- Varying the preozone contact time from 5 to 30 min has little effect on settled water turbidity and TOC for the conditions tested.

The effect of prechlorination and preozonation on coagulant-particle-NOM interactions are subtle and complex. Thus, use of prechlorination and preozonation as a coagulant aid will not likely be its primary application.

Acknowledgements

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References

[1] Amy, G.L., Sierka, R.A., Bedessem, J., Price, D., Tan, L., 1992. Molecular size distributions of dissolved organic matter. J. Am. Water Work Assoc. 84 (6), 67-75.

[2] APHA. Standard Methods for the Examination of Water and Wastewater. 1995. AWWA, WEF.

[3] Becker, W.C. Impact of Ozonation on coagulation: Model and Natural water Studies. Doctoral dissertation, The John Hopkins University, Baltimore (1995).

[4] Chiang P.C., E.E. Chang, C.H. Liang. 2002. NOM characteristics and treatabilities of ozonation processes. Chemosphere 46 (2002) 929-936. www.elsevier.com/locate/chemosphere.

[5] Crozes, G., White, P., Marshall, M., 1995. Enhanced coagulation: its effect on Nom Removal and chemical costs. J. Am. Water Work Assoc. 87 (1), 78-89.

[6] Edwards, M., Benjamin, M.M. 2002. Effect of Preozonation on Coagulant-NOM-Interactions. Jour. AWWA, 84:63.

[7] Jacangelo, J.G., N.L., Reagua, K.M. Aieta, E.M. Kranser S.W., McGuire, M.J., 1989. Ozonation assessing its role in the formation and control of disinfection by-products. J. Am. Water Work Assoc. 81 (8), 74-84.

[8] Hoehn H., Co-Chair Andrea Dietrich, Co-Chair and Daniel Gallagher. July 27, 2001. July 27, 2001. The effect of predisinfection with chlorine dioxide on the formation of haloacetic acids and trihalomethanes in a drinking water supply. Master of Science in Environmental Engineering. The Virginia Polytechnic Institute and State University.

[9] Kawamura s., 2000. Integrated Design and Operation on Water treatment Facilities. John Wiley and Sons Press. CA.

[10] Ken C., Steve V., Bill B., and Mark C. 2000.secendory effects of enhanced coagulation and softening. Jour. AWWA. Vol. 92. No. 6. pp. 63-75.

[11] Letterman, Raymond, ed. 1999. Water Quality and treatment: A Handbook of Community Water Supplies. New York, NY: McGraw-Hill.

[12] Orren D. Schneider and John E. Tobiason. 2000. Preozonation effects on coagulation. JOURNAL AWWA .Vol. 92, No. 10 pp. 74–87.

[13] Owen, D.M., Amy, G.L., Chowdhury, Z.K., Paode, R., Mccoy, G., Viscosil, K., 1995. NOM characterization and treatability. J. Am. Water Work Assoc. 87 (1), 46-63.

[14] Tae-Wook Ha, Kwang-Ho Choo, and Sang-June Choi. Effect of chlorine on adsorption/ultrafiltration treatment for removing natural organic matter in drinking water. 2004. Journal of Colloid and Interface Science 274 (2004) 587–593 www.elsevier.com/locate/jcis.

[15] W. Liua, H. Wua, Z. Wang, S.L. Ong, J.Y. Hu, W.J. Ng. Investigation of assimilable organic carbon (AOC) and bacterial regrowth in drinking water distribution system. 2001. Water Research 36 (2002) 891–898.

Section 4
Groundwater flow problems and remediation

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Spatial and temporal distribution of nitrate contents in the Mancha Oriental Hydrogeological System, SE Spain: 1998–2003

A. Moratalla[1], J. J. Gómez-Alday[2] & J. de las Heras[1]
[1]Centro Regional de Estudios del Agua (CREA), Universidad de Castilla-La Mancha (UCLM), Albacete, Spain
[2]Grupo de Teledetección y SIG. Instituto de Desarrollo Regional (IDR), Universidad de Castilla-La Mancha (UCLM), Albacete, Spain

Abstract

This project determines the nitrate content as well as spatial and temporal distribution in the water supply wells which extract groundwater from the Mancha Oriental System (MOS). The underground resources of the system are used in maintaining approximately 80,000 hectares of irrigation and are the water supply for a total population of 275,000 inhabitants. The average nitrate contents show a heterogeneous spatial distribution, varying between 0.1 mg l^{-1} and 125 mg l^{-1}. The highest levels are associated with areas with large areas of irrigated crops. However, there are also points that are not found to be spatially linked to this type of farming in which nitrate has been detected in significant quantities. The presence of nitrate in these areas can be explained considering other sources of pollution or transport of the pollutant from contaminated areas through groundwater flow. In general, average nitrate values show a growing tendency during the period 1998 and 2003.
Keywords: nitrate, irrigated crops, groundwater, Mancha Oriental System, SE Spain.

1 Introduction

The natural quality of groundwater can be observed as altered by the presence of nitrate proceeding from farming and livestock activities. In fact, the excessive application of fertilizers has been admitted as the main cause of nitrate and other

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pollutants appearing in groundwater, [1–3]. In the European Union, Directives 91/676/CEE, 2000/60/CE, and 2006/118/CE consider polluted groundwater with nitrate content higher than 50 mg l^{-1}.

Over the last 30 years, farming activities have grown significantly due to the extraction of water from the MOS: Irrigated crops have risen from 20,000 ha in 1982 to nearly 80,000 ha in the year 2003, according to Calera and Martín [4]. The representative crops grown in the area are corn, barley, wheat, sunflowers and alfalfa. These crops require high quantities of fertilizer and water to maintain elevated agricultural yield. In addition, groundwater within this system also represents the only resource supplying a total of 275,000 inhabitants spread throughout the provinces of Albacete and Cuenca.

The regional dependence on groundwater resources justifies undertaking studies on the origin, behavior and evolution of nitrate content found. In the MOS, De las Heras *et al* [5], study nitrate pollution in groundwater within the MOS during the years 1998 and 1999 and indicate a high presence of nitrate in irrigation water and the public water supply. The study concludes that certain irrigation practices linked to high output crops could be responsible for the high nitrate content found in several areas of the MOS.

In this study, groundwater used for the urban water supply has been sampled and the nitrate content has been determined along with spatial and temporal distribution for the years 1998, 1999, 2001 and 2003. Although, in general, the highest nitrogen content is associated with large irrigated areas, there are other points where the nitrate levels could be related to other sources of contamination. In addition, the nitrate present could have been transported from production areas through groundwater flow.

2 Study area

The MOS is located in the south east of the Region of Castilla-La Mancha, within the hydrographic catchment of the Júcar River Basin. This hydrogeological system comprises an area of 7,260 km^2, Sanz *et al* [6].

Considering hydrogeology, the MOS can be considered a multiple-layered aquifer formed by the superposition of nine hydrogeological units (HU). The layers are of a diverse nature, belonging to the Triassic, Jurassic, Cretaceous, and Miocene period. The MOS is divided into six hydrogeological domains: Septentrional Domain (DS), Central Domain (DC), Salobral-Los Llanos Domain (DSL), Moro-Nevazos Domain (DMN), Pozo Cañada Domain (DPC), and Montearagón-Carcelén Domain (DMC), see Sanz *et al* [6] for further details.

The groundwater type within the study area is Calcium Bicarbonate (Table 1). This composition is related to the dissolution of the carbonate lithologies that are predominant in the aquifer HU.

The area climate is continental, semi-arid, with extreme temperatures in summer as well as winter. Average annual temperatures vary between 13ºC and 14.5ºC. Precipitation is between 300 mm in the south and 550 mm in the northern MOS.

Table 1: General characteristics of groundwater (782 samples, years 1998, 1999, 2001 and 2003).

Variable	Average	Min.	Max.	S.D.
NO^{3-} (mg l^{-1})	24.1	2.6	116.2	18.7
HCO^{3-}(mg l^{-1})	321.4	130.9	603.6	75.7
Cl-(mg l^{-1})	39.1	5.2	171.4	30.4
SO^{4-}(mg l^{-1})	127.5	4.3	634.1	121.4
Ca^{2+}(mg l^{-1})	96.5	24.0	218.0	32.7
Mg^{2+}(mg l^{-1})	47.3	4.0	145.0	25.9
Na^{+}(mg l^{-1})	15.4	1.8	122.5	16.0
K^{+}(mg l^{-1})	1.7	0.1	15.1	2.4

The largest river crossing the hydrogeological area is the Júcar River. The Valdemembra, Arroyo Ledaña and Cabriel Rivers are affluents of the Júcar on the left side.

3 Methodology

A total of 782 groundwater samples were collected in 57 sampling events from urban water supplies located throughout the three hydrogeological domains of the MOS (Fig. 1). Each point has been sampled every trimester during the years 1998, 1999, 2001 and 2003. Data is not available in the DS for 1998. In this domain only one sample point was analyzed in 1999. The nitrate content in each sample was determined using the ionic chromatography technique APHA [7].

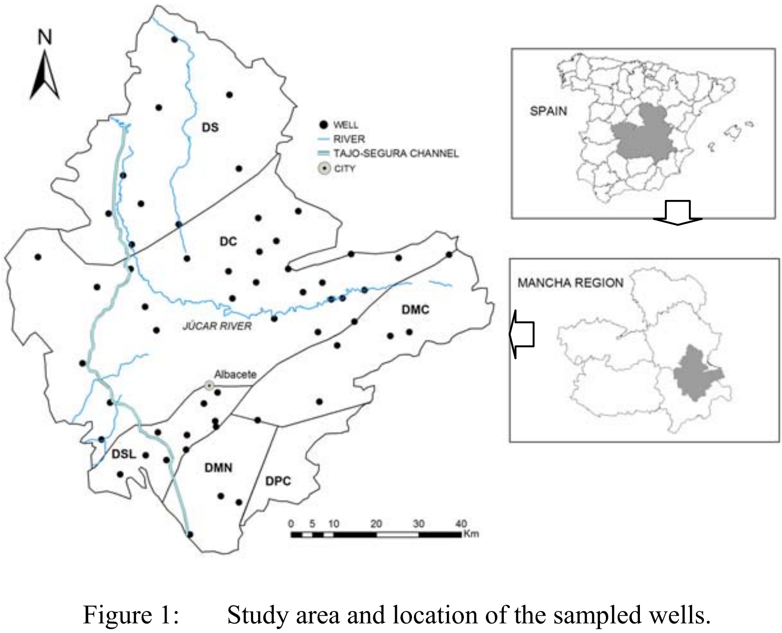

Figure 1: Study area and location of the sampled wells.

The maps of nitrate distribution and irrigated surfaces were obtained using the ArcMap 8.2 software (ESRI). The data on irrigated areas come from the GIS and Teledetection Group from the IDR at UCLM. In order to obtain the spatial distribution of nitrate content (Fig. 2), the method “inverse distance weighted” (IDW) was used.

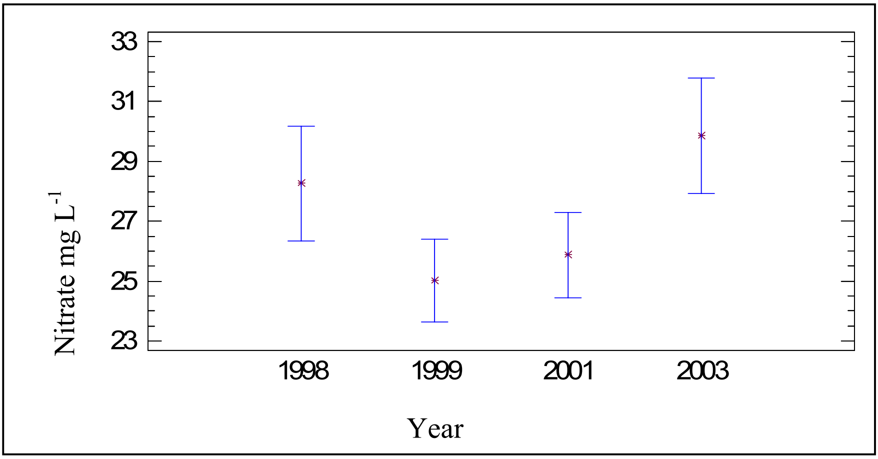

Figure 2: Nitrate content means and 95% intervals LCD.

4 Nitrate contents in groundwater

Analytical results indicate significant differences in average nitrate concentration depending on the domain considered, although none exceed the admissible maximum quantity (50 mg l^{-1}). The highest average content is found in DC and DSL. In DC, nitrate concentrations are between 0.1 mg l^{-1} and 125 mg l^{-1} (Mean value: 29.7 mg l^{-1}) (Fig. 3; Table 2). In DSL, nitrate concentration values vary between 11.7 mg l^{-1} and 61.4 mg l^{-1} (Mean value: 26.3 mg l^{-1}). The domains DS (Mean value: 15.2 mg l^{-1}), DPC (Mean value: 17.8 mg l^{-1}), DMN (Mean value: 18.6 mg l^{-1}), and DMC (Mean value: 22.3 mg l^{-1}) have lower values present (Fig. 3; Table 2).

The analysis of the temporal evolution of values yields results indicating a decrease between 1998 and 1999 in average nitrate concentration in all domains considered. This tendency was also found in the study by De las Heras *et al* [5]. However, after 1999 a progressive increase in nitrate presence was found in groundwater.

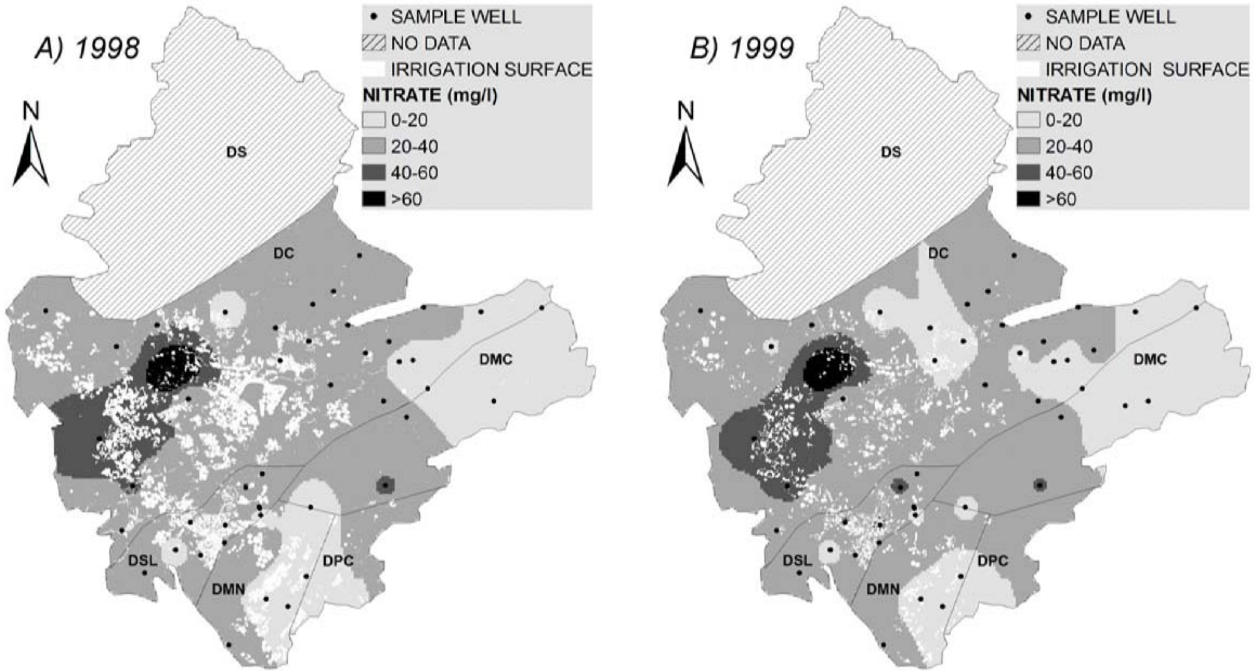

Figure 3: Distribution of the mean annual nitrate content in groundwater. A) 1998, B) 1999, C) 2001, and D) 2003.

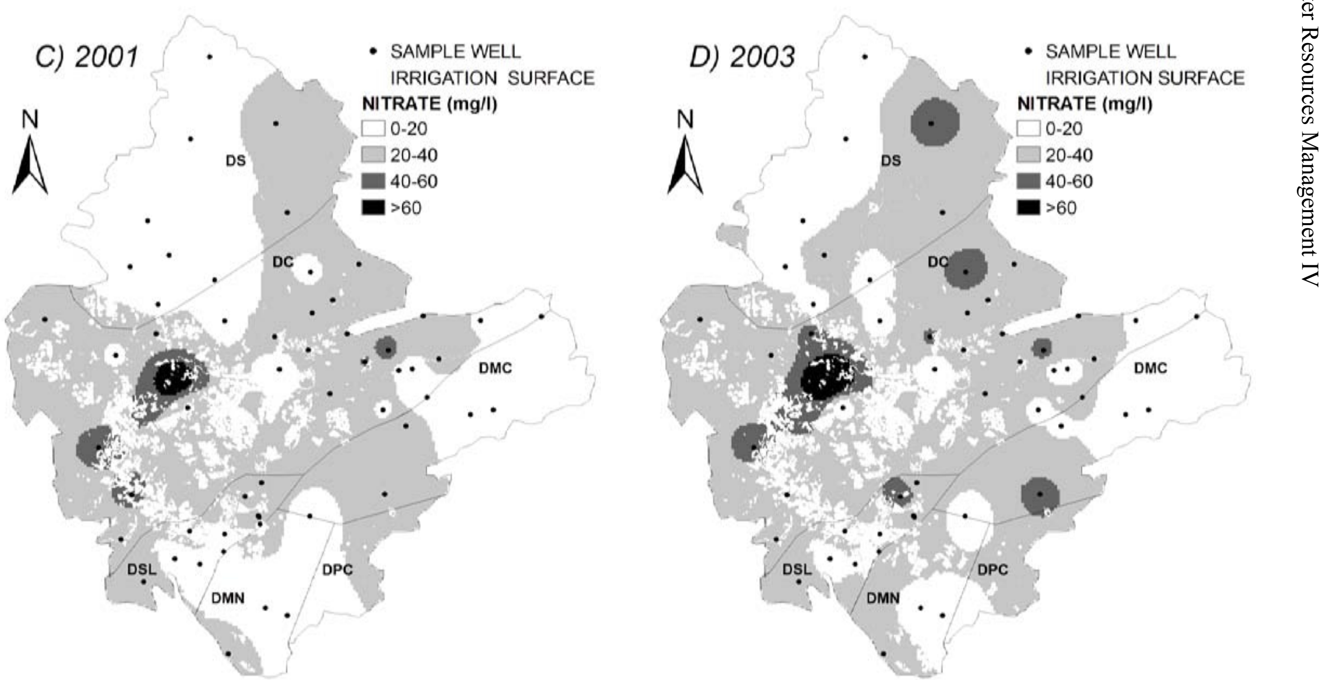

Figure 3: (cont.)

Table 2: Basic statistics of the nitrate contents (n=number of samples).

	n	Mean	Minimum	Maximum	S.D.
Septentrional Domain					
2001	16	13.2	9.1	29.4	5.7
2003	30	16.3	6.8	61.9	12.6
Total	47	15.2			
Central Domain					
1998	74	31.1	1.5	125.0	19.3
1999	133	26.9	0.9	122.0	16.6
2001	168	28.4	0.1	107.1	17.4
2003	73	36.8	5.6	124.1	21.6
Total	448	29.7			
Montearagón-Carcelén Domain					
1998	5	30.7	5.0	42.0	15.3
1999	16	19.8	4.3	43.2	14.1
2001	22	21.5	0.4	56.2	17.7
2003	11	28.2	2.6	52.4	18.9
Total	54	22.3			
Pozo Cañada Domain					
1998	5	19.8	16.4	22.9	3.1
1999	9	16.8	14.4	20.4	2.2
Total	14	17.8			
Moro-Nevazos Domain					
1998	8	20.6	14.1	26.4	3.9
1999	18	14.7	10.3	24.9	3.9
2001	18	20.6	9.9	34.1	8.6
2003	6	22.00	10.4	33.1	10.7
Total	50	18.6			
Salobral-Los Llanos Domain					
1998	45	26.8	14.0	49.4	8.9
1999	88	26.3	12.4	48.1	9.4
2001	21	24.7	12.8	41.5	9.1
2003	15	27.4	11.7	61.4	11.9
Total	169	26.3			

5 Spatial and temporal distribution of nitrate contents in the MOS

Nitrate content in groundwater is distributed in a heterogeneous fashion throughout the MOS (Fig. 3). The areas of highest annual means of nitrate concentrations are associated with the DC and DSL domains, which both sustain large areas of irrigated cultivation. Agricultural activities have been admitted to be potential sources of nitrate pollution due to extensive application of inorganic fertilizers [8–11].

Nevertheless, some samples in the DS and DMC domains with nitrate concentrations higher than the permitted levels are not spatially associated with irrigated crops (Fig. 3). This fact suggests that the origin of nitrate could be associated with sources other than farming, such as urban or industrial waste, or waste from livestock activities. Another possible explanation would be that groundwater can transport nitrate from point sources of pollution to other areas of the MOS where potential polluting activities are nonexistent. In order to collect data supporting this hypothesis, additional studies are required to further investigate the spatial and temporal evolution of nitrate concentrations and the relationship with the flow regime of the hydrogeological system (e.g. aquifer-river relation).

6 Conclusions

The MOS groundwater presents elevated nitrate content, which can reach maximum values of 125 mg l^{-1}. Significant differences have been found within the different domains considered. The average maximum values were detected in DC (29.7 mg l^{-1}) and DSL (26.3 mg l^{-1}). The minimum values were found in DS (15.2 mg l^{-1}). Nitrate presence in the MOS groundwater decreases within the period 1998–1999 and increases within the period 1999 and 2003.

The maximum values correspond to the most extensive irrigated farmed areas (DC and DSL). The source of nitrate in these domains can be associated with the use of inorganic fertilizers applied to irrigated crops. Nevertheless, there are areas where the irrigated area is practically zero, as in DS and DMC, where average nitrate content was also relatively high. In these areas, the nitrate present could be related to other inputs or may have been transported from the source of pollution by groundwater flow.

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References

[1] Follet, R. F., *Nitrogen Management and Ground Water Protection,* Elsevier Science Publishers B.V: Amsterdam, The Netherlands, 1989.

[2] Kladivko, E.J., Van Scoyoc, G.E., Monke, E.J., Oades, K.M. & Pask, W., Pesticida and nutrient movement into subsurface tile drains on a silt loam soil in Indiana. *Journal of Environmental Quality*, **20**, pp. 264-270, 1991.

[3] Keeney, D.R. & De Luca, T.H., Des Moines River nitrate in relation to watershed agricultural practices: 1945 versus 1980's. *Journal of Environmental Quality,* **22**, pp. 267-272, 1993.

[4] Calera, A. & Martín F., Uso de la teledetección en el seguimiento de los cultivos de regadío (Chapter XIV). Agua y Agronomía, ed. Mundi Prensa: Madrid, Barcelona, México, pp. 523-583, 2005.

[5] De Las Heras, J., Castro, E., Mañas, P., Sánchez-Vizcaíno, J., Sánchez, J.C. & Mejías, M., Groundwater Quality and Nitrate Pollution of 08-29 Hydrogeological Unit Mancha Oriental, Spain. *Journal of Balkan Ecology*, **4**, pp. 434-444. 2001.
[6] Sanz, D., Martínez-Alfaro, P.E., Castaño, S. & Gómez-Alday, J.J., Caracterización de los dominios hidrogeológicos individualizados en el Sistema Mancha Oriental. SE Español. *Geogaceta*, **38**, pp. 251-254, 2005.
[7] APHA, Standard methods for the examination of water and wastewater. American Public Health Association, Washington D.C., 1989.
[8] González, J.C., Grande, J.A., Barragán, F.J., Ocaña, J.A. & De La Torre, M.L., Nitrate Accumulation and Other Components of the Groundwater in Relation to Cropping System in an Aquifer in Southwestern Spain. *Water Resources Management* **19**, pp. 1-22, 2005.
[9] Helena B., Vega M., Barrado E., Pardo R. & Fernández L., A case of hydrochemical characterization of an alluvial aquifer influenced by human activities. *Water, Air and Soil Pollution* **112**, pp. 365-387, 1998
[10] Schepers, J.S., Varvel, G.E. & Watts, D.G., Nitrogen and water management strategies to protect groundwater quality. *Trans. World Cong. Soil Sci.* **5**, pp. 192-204, 1994.
[11] Richards, R., Baker, D., Creamer, N., Kramer, J., Ewing, D., Merryfield, B. & Wallrabenstein, L., , 'Well water quality, well vulnerability, and agricultural contamination in the Midwestern United States', *Journal of Environmental Quality,* **25**, pp. 389-402, 1996.

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Mathematical model and technology to provide new resources of groundwater for irrigations

E. Nițescu[1], I. Moruz[1], C. Nițescu[2], E. Chiorescu[3] & Șt. Popescu[3]
[1]*Department of Cadastre, Technical University "Gh. Asachi", Iassy, Romania*
[2]*Department of Mathematics, Technical University "Gh. Asachi", Iassy, Romania*
[3]*Department of Hydroamelioration and Environment Protection, Technical University "Gh. Asachi", Iassy, Romania*

Abstract

The opening and use of local sources of groundwater for irrigation offers important economic and social advantages. However, an extended use of groundwater is limited by the currently utilized technology for the tapping of these waters, which is very expensive. This paper shows a mathematical model and technological solution that allows the creation of local underground water accumulations at affordable prices for agricultural farms. Specifically for Romania is the fact that there is used mainly surface water (ca. 93% of the total consumption). With a few exceptions, the groundwater is used only for the water supply of some urban areas. Today in Romania the reasons that make it difficult to introduce irrigation in agricultural farms or mini-farms are: a) the lack of water (in farms located outside irrigated areas); b) a prohibitive price for water delivered to farms situated at the boundaries of large irrigation systems; c) costs generated by the huge transport distances of the water, by re-pumping and by water losses that sometimes exceed 40% of the volume carried via channel network. These considerations show evidently that, in order to deliver water to also farms located far away from surface-water sources, the only economic solution is the use of local groundwater resources. The primary data from field and/or lab studies have to be systematized and analysed on various categories: topographic, climatic, hydrologic, hydro-geologic, pedological, geological and chemical. Moreover, there have been set the basic work assumptions, all these leading to the construction of the conceptual and the mathematical models for underground reservoirs' design. Taking into account the fact that in Romania such irrigation systems have not been built and the prices for conventional technologies are prohibitive, at least today, we propose a new technology, based on a Patent Ro 94257 Nițescu (*Installation for the Execution of Vertical, Nonpermeable Screens*, 1987). This technology has to be mainly focused on a reducing of the specific investment costs, because the lowering of costs becomes a crucial condition for a sustained development of agriculture.

Keywords: groundwater, underground water accumulations, technological solution, conceptual and mathematical models, irrigation.

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1 Introduction

Romania is characterized by the fact that there is used mainly surface water (ca. 93% of the total consumption). The groundwater, with a few exceptions, is used only for the water supply of some urban areas. In developed countries the system is better balanced, in France, for example Nițescu et al [3], groundwater represents 60% of the consumption, agriculture using 68% of the total water, whilst industries use only 5% (due to the general implementation of water recycling).

Today in Romania the reasons that make it difficult to introduce irrigations in agricultural farms or mini-farms are: a) the lack of water (in farms located outside irrigated areas); b) a prohibitive price for water delivered to farms situated at the boundaries of large irrigation systems; c) costs generated by the huge transport distances of the water, by re-pumping and by water losses that exceed sometimes 40% of the volumes carried via channel network.

These considerations show evidently that, in order to deliver water also to farms located far away from surface-water sources, the only economic solution is a more extensive use of local groundwater resources.

However, in many agricultural areas of Romania, the groundwater and deeper aquifers do not provide flows complying with the real needs. Hence, it is mandatory to create local reservoirs of groundwater (at relatively small depths), action which can be justified by the following arguments: a) the use of groundwater for irrigations provides better conditions in terms of seasonal, annual and multi-annual adjustment of irrigation systems; b) for agricultural surfaces on that prevails a salt-ascension risk, the use of groundwater eliminates the need of desalination draining systems; c) in agricultural areas with a high groundwater level, the tapping of groundwater lowers this level, fact that is benefit for cultures; d) the water losses generated by evaporation are eliminated.

Therefore a rational use of the upper aquifer does not harm the environment through negative hydro-geological alterations. Furthermore it helps to keep the balance of water within irrigated areas, by decreasing the risk of soil deterioration and of secondary salt marshes. Along the same line, in India there is a law that obliges farmers to use local groundwater for irrigations, providing that in the agricultural areas there is an adequate groundwater level. In terms of environment (soil's) protection some objections will arise about the groundwater's mineral content, which frequently is higher than that of surface waters. The answer to provide for this issue is that in the case of an underground reservoir, the water which is continuously consumed and replaced will progressively decrease its load of minerals. In these situations a solution is represented by performing a dilution with non-mineralized water until the load of minerals gets to admissible limits, or to provide irrigation only on light soils [3].

2 Agricultural areas fitted for the creation of local underground reservoirs from upper-aquifer waters

The areas in that can be conceived underground water reservoirs are:

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a) Areas located at the base of terraces, if within these exists a permanent water flow.

b) Wetland areas; studies proved that the groundwater afflux in rivers represent 10–40% of the river's total flow. Mainly, the feeding of the river from aquifers takes place during the low water period. Considering that the water deficit for irrigation appears during a low water period, it results that the local underground reservoir has to be designed in order to avoid water losses towards the river bed in the area of the aquifer tapping. In these conditions, the feeding of the reservoir is not made also from river, but only from infiltrated rain waters and from the drainage of the underground hydrographical basin of the designed reservoir. Exceptions are the greater rivers with slight level fluctuations, the feeding of the aquifer from the river being quasi-permanent (e.g. the Danube River's wetland).

c) Irrigated areas where the groundwater level has considerably increased due to water losses in the distribution system.

d) Areas in that the aquifer can be artificially loaded.

In order to tap water from the underground reservoir, can be employed one of the following usual technical solutions: 1° tapping shafts, if the groundwater layer is mighty and assures a sufficient afflux; 2° shafts with radial drain pipes, if the groundwater has a small flow rate; 3° horizontal catching galleries, solution that can be applied only when the aquifer is at a small depth (max.10–15 m).

In Romania farms needing irrigations are located in plateau or plain areas, where the groundwater assures in most cases only small rates of flow. This situation would require solutions of the type 2° or 3°, both implying prohibitive costs for a farm or even for farm associations.

The conclusion is that, depending on hydrological and hydro-geological conditions, the local underground reservoirs can supply irrigation systems. The water of the reservoir can be used alternatively to complement a surface source during peak consumption periods, or to provide the whole irrigation water volumes at pre-defined flow rates. Outside irrigation periods the available groundwater can be used for other activities, which will contribute to a faster investment amortization. A complex use of water stored in underground reservoirs allows also a good control of groundwater layers in order to avoid an excessive humidity within soils around the reservoir.

3 Establishing the parameters and the computing model for a local underground reservoir

The function-simulation for a local underground reservoir comprises the following steps.

3.1 Conceptual model. Functioning scenarios

In order to establish the conceptual model concerning the flow issues for a local underground reservoir, have been identified the characteristics and the main problems for the considered groundwater layer, as well as for the irrigation system. There also have been established and structured the data needed for the modelling performed in the pre-processing stage.

3.1.1 Data from the preliminary studies

The primary data from field and/or lab studies, what must be considered for design of local sources for irrigation, mostly collected by specialized authorities, have to be systematized and analyzed on various categories: topographic, climatic, hydrologic, hydro-geologic, pedological, geological and chemical. Digital data can be efficiently be processed in EXCEL or/and MATLAB. Climatic, hydrologic and hydro-geologic data are to be statistically prepared (e.g. multi-annual averages).

3.1.1.1 Topographic data. This can be extracted (if there is no digital map available) from a physical support (maps, etc) on layers, by means of scanning and digitization with a special software, e.g. DIDGER 3. So, topographic data are to be prepared in a pre-processing format compatible with the digital processing software (e.g. FEFLOW v5.1, AQUA3D).

With an object-oriented approach, topographic data have been structured as objects in various classes, hence resulting a series of layers: 1° the delimitation of the catching surface of the underground reservoir, S_{UB}, and of the culturable irrigable surface, S_{IS}; 2° level curves; 3° hydrographical network; 4° distribution of pressures (observation drillings); 5° shafts and exploitation drains; 6° road network and towns; 7° soils and textures classes and using classes. The outlines of the modelling domain have been established in order to create boundary conditions very similar to the natural feeding of the aquifer and reservoir's exploitation.

To analyse the possibility of an artificial recharge of the exploited aquifer, there must be known the relative position of the surfaces S_{IS} and S_{UB}. To this purpose will be determined the surface $S_{IS\_UB}=S_{IS} \cap S_{UB}$. Thereupon must be analysed the typical possible situations: I – the surfaces S_{IS} and S_{UB} are disjunctive if $S_{IS\_UB}=\Phi$; II – in the case $S_{IS\_UB}\neq\Phi$, the surface S_{IS} is included (or includes) partially/totally in surface S_{UB}, if $S_{IS\_UB} \subseteq S_{UB}$ (or $S_{IS\_UB} \subseteq S_{IS}$).

3.1.1.2 Climate data. This must be collected from several meteorological stations that influence the aquifer's hydrographic basin; these data being: 1° monthly precipitations; 2° monthly temperatures; 3° potential and/or real evapo-transpiration (if these data are not available, the PET, the potential evapo-transpiration, can be computed on basis of recorded temperatures, being preferably to use the Penman formula. The RET, the real evapo-transpiration can be taken as equal of the PET).

3.1.1.3 Hydrological data. These are to be collected when levels in the aquifer are significantly influenced by levels within one or two rivers (when the upper aquifer is located in rivers' inferior terraces). These data refer to the status of monthly levels and are to be collected from 1...2 hydrometric stations, preferably one in the upstream area and the other one in the downstream area.

3.1.1.4 Hydro-geological data. These are to be collected from hydro-geological stations or from the hydroelectric plant's observation drillings and are comprising: 1° the geometrical and geotechnical parameters (depth of basic impermeable layer, rocks' texture); 2° the pressure levels in the observation drillings (measurements to be performed according to the frequency instructed by the station); 3° pumping tests (useful for the assessment of conductivity/ transmissivity); 4º the specific underground flow discharge.

3.1.1.5 Chemical data. These are needed in order to establish the water's quality parameters. Furthermore, these data will serve to define the type of uses for the water and to appreciate the risks of pollution.

3.1.2 The basic work assumptions to the construction of the conceptual model

Considering the fact that the aquifer is or of the alluvial type, or that it shows only a little thickness, the multi-layer structure has been assimilated to an one-layer aquifer, for which have been adopted equivalent values in vertical direction.

In such an underground medium the velocities of flow are relatively small, and the vertical distribution of pressures gets closer to the one imposed by hydrostatic law. This means that the vertical component of pressure gradient is small compared to the horizontal one. Besides, in case of non-isotropic aquifers, the vertical hydro-conductivity is much lesser than the horizontal one.

Therefore the modelling domain, Ω, allows a two-dimensional (2D) representation, which is situated in the horizontal plane (xOy). Within the xOy plane the isotropy conditions are fulfilled. On the other hand the homogeneity conditions are fulfilled only in some zones, the domain being generally non-homogeny.

So the values of the parameters (aquifer's characteristics: porosity, hydro-conductivity, infiltrations, levels, flows) have imposed punctually or in certain zones (in characteristic points, respectively on elements of model's domain).

In the issue, for the considered model there have been adopted the following working assumptions: 1° the aquifer is considered as a isotropic and saturated porous medium; 2° homogeneity and isotropy conditions are fulfilled for a representative elementary volume (REV) for a given study scale (Darcy law's applicability conditions); 3º the aquifer storages a free water table; 4° the flow process is assumed to take place in horizontal direction (Dupuit assumption); 5º the aquifer's feeding is made by means of infiltrations in the modelling domain and by filtration flows on the domain's outlines (coming from aquifers located outside the modelling domain, having a higher hydrostatic level, or from a river).

Next to the data processing will be identified the main variables (hydraulic load) and the secondary variables (velocity, flows rates), concerning the global flowing within the aquifer.

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In the conceptual model must be considered the type of the adequate zone for the arrangement of the underground reservoir, as well as the technical solutions for the water catching and for the accomplishment and the operation of the irrigation system. So, for example, at the base of a terrace the groundwater will be retained by the implantation of a vertical screen that is constrained in the impermeable base of the aquifer, and for the water catching will be realized a pumping shaft with two horizontal drains, which are laid along the screen. The ground water provice (domain) of so accomplished underground reservoir is bounded as follows: 1º at upper part - by the ground water line; 2º at bottom - by the vertical impermeable screen equipped with drains and 3º sidewise – by the greatest slope lines; beside these, it was adopted continuous flow irrigation but only in growing season.

The design of the underground reservoir was performed by soil water balance method for a water management year and this consists in two distinct stages: a) the dimensioning and b) the verification.

In the verification stage, the functioning scenarios for an underground reservoir differentiate between two distinctive work stages: 1° the stage of reservoir filling (when no water is tapped from the collecting - tapping shaft); 2° the stage of reservoir draining (when water is tapped from the collecting tapping shaft). So, starting from a known initial potential of the aquatic layer and taking into account both scenarios, which alternate in time, will be determined the evolution of the aquifer's potential (hydraulic head) for the analyzed period (for example a water management year).

The mathematical models for the two designing stages are following presented.

3.2 The mathematical model for the dimensioning stage

In this stage must determine the drain's lengths, L_1 and L_2, using the equation:

$$\beta_{\mathrm{UR}} q_{\mathrm{UF}} \left(L_1 \cos\alpha_1 + L_2 \cos\alpha_2 \right) T_{\mathrm{WMY}} = \bar{M} A_{\mathrm{IS}}, \qquad (1)$$

where:

β_{UR} = storage coefficient of underground reservoir, [1];
q_{UF} = specific underground flow discharge, $[\mathrm{L}^2\mathrm{T}^{-1}]$;
α_1, α_2 = angles between underground flow's direction and normal to the drains' axis, [1];
T_{WMY} = time span of the water management year, [T];
A_{IS} = culturable irrigable area, $[\mathrm{L}^2]$;
$\bar{M}$ = the weighted mean irrigation rate, corresponding to the agricultural cultures' (cultures#agr's) composition, [L].

The rate $\bar{M}$ was evaluated as follows.

$$\bar{M} = \sum_{(j)} \chi_j M_j = \sum_{(j)} \frac{A_j}{A_{\mathrm{IS}}} M_j, \text{ with } \sum_{(j)} \chi_j = 1 \text{ and } \sum_{(j)} A_j = A_{\mathrm{IS}}, \qquad (2)$$

where:

χ_j = the weight of the culture#agr j in the considered composition;

A_j = occupied area of the culture#agr j in the considered composition;

M_j = the irrigation rate of the culture#agr j.

3.3 The mathematical model for the verification stage

In this stage the mathematical model is constituted by the equations which govern the flowing, as well as the initial and the boundary conditions – for essentially horizontal 2D flow through unconfined (phreatic) aquifer (Diersch [1]). These equations and conditions (vertically averaged/depth-integrated) were particularized in keeping with the working assumptions of the conceptual model.

3.3.1 The governing equations

These following: vertically averaged mass conservation (of the liquid phase) and depth-integrated Darcy flux:

$$S\frac{\partial h}{\partial t}+\frac{\partial \overline{q}_x}{\partial x}+\frac{\partial \overline{q}_y}{\partial y}=\overline{Q}_p\,, \tag{3}$$

$$\overline{q}_x=-\left(h-z^{\text{bottom}}\right)K\frac{\partial h}{\partial x}\ \text{ and }\ \overline{q}_y=-\left(h-z^{\text{bottom}}\right)K\frac{\partial h}{\partial y}\,, \tag{4}$$

where:

S = storage coefficient, [1];

h = hydraulic head, [L];

$\overline{q}_x, \overline{q}_y$ = depth-integrated Darcy velocity along the coordinate axis, [L^2T^{-1}];

$\overline{Q}_p$ = depth-integrated specific sink/source rate of liquid, [LT^{-1}];

K = hydraulic conductivity, [LT^{-1}];

z^{bottom} = bottom geometry (height) of aquifer, [L].

Specific rate $\overline{Q}_p$ is represented by the infiltration water losses' rate, what has been computed on basis of following balance equation:

$$\overline{Q}_p=\begin{cases}\dfrac{1}{T_{\text{GS}}}\left[\left(1-\sigma\right)\text{PP}_{\text{GS}}-\text{RET}_{\text{GS}}+\overline{M}\,\dfrac{A_{\text{IS\_UB}}}{A_{\text{UB}}}\right], \text{ for } t\in T_{\text{GS}};\\[2ex] \dfrac{1}{T_{\text{OGS}}}\left[\left(1-\sigma\right)\text{PP}_{\text{OGS}}-\text{RET}_{\text{OGS}}\right], \text{ for } t\in T_{\text{OGS}},\end{cases} \tag{5}$$

where:

T_{GS} = time span of growing season, [T];

T_{OGS} = time span out of growing season, $T_{\text{OGS}} = T_{\text{WMY}} - T_{\text{GS}}$, [T];

σ = flow coefficient, assessed in function of the terrain's slope and texture, and also the type of use;

$\mathrm{PP}_{\mathrm{GS}}, \mathrm{RET}_{\mathrm{GS}}$ = precipitations, RET respectively, since growing season, [L];

$\mathrm{PP}_{\mathrm{OGS}}, \mathrm{RET}_{\mathrm{OGS}}$ = precipitations, RET respectively, since out growing season, [L];

3.3.2 The initial conditions

These conditions are:

$$h(x, y, 0) = h_I(x, y) \text{ with } (x, y) \in \Omega \tag{6}$$

where h_I is a known function, defined on all analysed domain, Ω, which reflects the initial spatial distribution of the status variable *h*.

3.3.3 The boundary conditions

The conditions on the domain's outside, Γ, are only of integral Neumann type:

$$K\left(\frac{\partial h}{\partial x} n_x + \frac{\partial h}{\partial y} n_y\right) = -\overline{q}_h^R(t) \text{ with } (x, y) \in \Gamma, \tag{7}$$

where:

n_x, n_y = components of the normal unit vector (positive outward), [1];

$\overline{q}_h^R(t)$ =depth-integrated normal boundary fluid flux (positive outward), $[\mathrm{L}^2\mathrm{T}^{-1}]$.

In the stage of reservoir drainage, only on the bottom domain's outside, Γ_2, we have:

$$\overline{q}_h^R(t) = \frac{\overline{M}A_{\mathrm{IS}}}{T_{\mathrm{GS}}\left(L_1 \cos\alpha_1 + L_2 \cos\alpha_2\right)} > 0; \tag{8}$$

but in the stage of reservoir filling, on all outside Γ, the conditions (7) became of natural type, id est $\overline{q}_h^R(t) \equiv 0$.

4 Technology for the construction of a underground reservoir

If we take into discussion (in order to diminish costs) only the tapping of upper aquifers, we have to keep in mind that the flow rate to such a catching (small depth) decreases in summertime when the feeding infiltrations become minimal, whilst the needs for irrigation are maximal. From this considerations results that the water supply of farms, which do not have access to surface waters, or for which the access is limited by huge pumping and water transport costs, is an issue which could be solved by means of: a) designing and building local reservoirs using less deep groundwater layers, such as the upper aquifer; b) conceiving a technology for the construction of water reservoirs at affordable cost.

Taking into account the fact that in Romania such irrigation systems have not yet been built and the prices for conventional technologies are prohibitive, at least today, we propose a new technology, based on a Patent [2], the invention being awarded with gold at World Fair Eureka-2000 in Belgium.

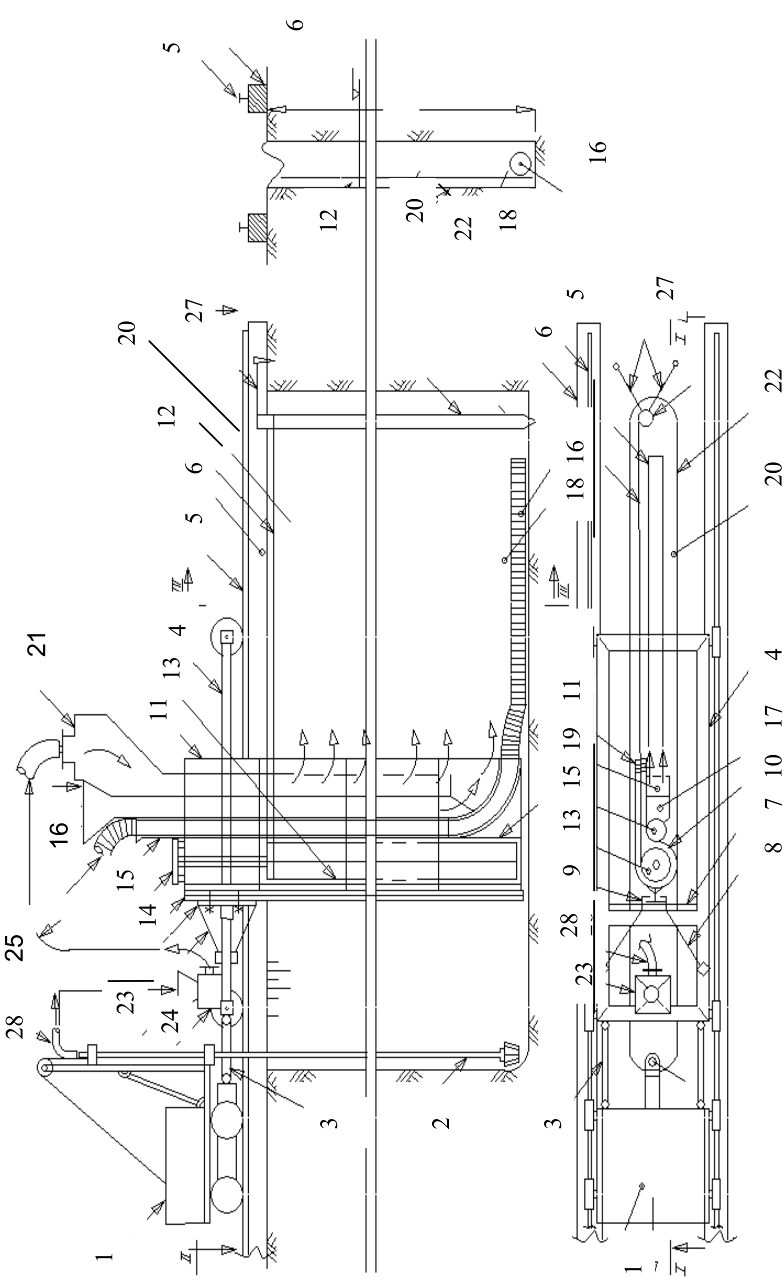

Figure 1: Scheme of an installation for earth mounting of vertical flexible screen.

The installation, fig.1, allows the barring of groundwater flow with a vertical membrane-like impermeable screen (20), located in plane, and shaped as a “V”, in order to intercept the current lines of the flowing water. The installation comprises a digger (1), the resulted residues being pumped towards a bunker (21) fitted with two discharge gutters (19) via several slots, back towards the trench, on each side of the membrane-screen, un-winded from a drum (13), which is vertically introduced in caisson (15) equipped at the posterior part with an exit slot of the membrane-screen (20) in the trench. The patent describes also how several screens can be connected, thus their length (that is the underground dam) being not limited.

The implementation of the technology means the following stages: a) when the screen is installed, a column for the pumping shaft will be mounted, the shaft being located in the angled peak of the screen; b) after finishing the screen, the installation has to be completed with a draining system along its wings. The draining lines, which are connected with the pumping shaft, are mandatory if the groundwater gets accumulated in rocks with low porosity; c) a depth drain is necessary to be executed, a drain able to be mounted from surface down to depths of 10–20 meters.

It is to be mentioned that the realization costs of the membrane-like flexible screen are ten times lower than that of conventional technologies (like ready-parts screens; monolith concrete screens, earth and cement screens).

5 Conclusions

1º The critical global climate changes that also have an impact on Romania, imposed the extended use of groundwater in order to complete and balance the surface-waters, also providing a better protection of the environment.

2º The technology presented in this paper for the realization of underground reservoirs has to be mainly focused on the lowering of investment costs. On the other hand this becomes a crucial condition for a sustained development of agriculture.

3º The presented conceptual and mathematical models contribute to scientific substantiation of the underground reservoirs’ for irrigation design.

References

[1] Diersch, H.J.G., *FEFLOW finite element subsurface flow & transport simulation system*), Reference Manual, WASY Institute, Berlin, 2002.

[2] Nițescu E., *Installation for the Execution of Vertical, Nonpermeable Screens*, Romanian Patent RO- 94275, 1987.

[3] Nițescu E., Popescu Șt., Toma D., Chiorescu Esmeralda a.o., *Technological Development Concerning Creation and Tapping of Ground Water for Irrigation*, Grant tip A – CNCSIS, nr. 536/2005.

Groundwater resources, development and management in the largest tectonic sedimentary basin, Japan

T. Tanaka
Graduate School of Life and Environmental Sciences,
University of Tsukuba, Ibaraki, Japan

Abstract

The purpose of this paper is to show a typical example of excessive extraction of groundwater resources and their resulting problems suffered in the economic growth period in Japan from 1955 to 1972. In the case of the study basin, this problem has been remedied by the implementation of regulations over the past 50 years and the relatively high recharge rate due to the natural hydrologic conditions of the area. Environmental consequences according to the excessive groundwater development and strategies for groundwater resource management are discussed, based on the Japanese experiences during the past 50 years.
Keywords: tectonic sedimentary basin, groundwater resources, excessive extraction, land subsidence, countermeasures, regulation law, monitoring system.

1 Introduction

The Kanto Plain is the largest tectonic sedimentary basin in Japan as shown in fig. 1. The plain is underlain by Pliocene and Pleistocene sediments of the Kanto Tectonic Basin, consisting of unconsolidated layers of silt, sand and gravel which extend to a depth of more than 3,000 m. Figure 2 shows the east-west geologic cross section across the Tokyo Metropolitan area to a depth of around 500 m below the sea level. These sediments form the main confined groundwater aquifers in the Kanto Plain.

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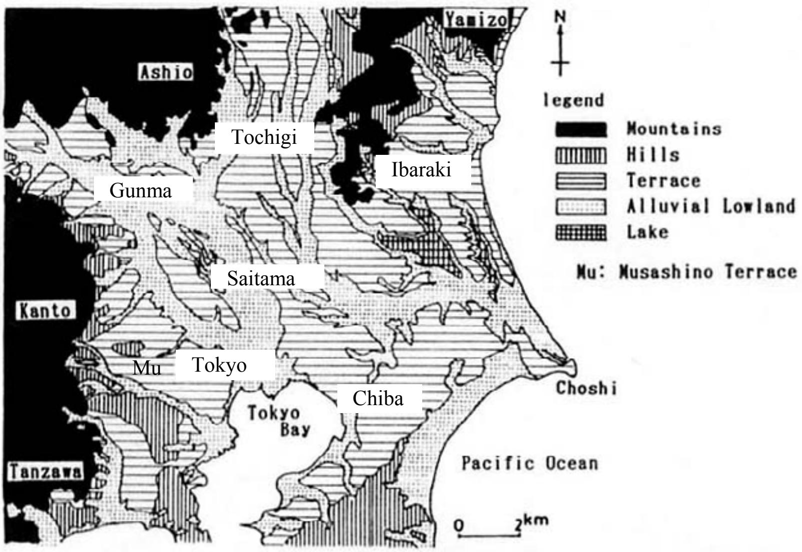

Figure 1: Topographic map of the Kanto Plain (Endo [1]).

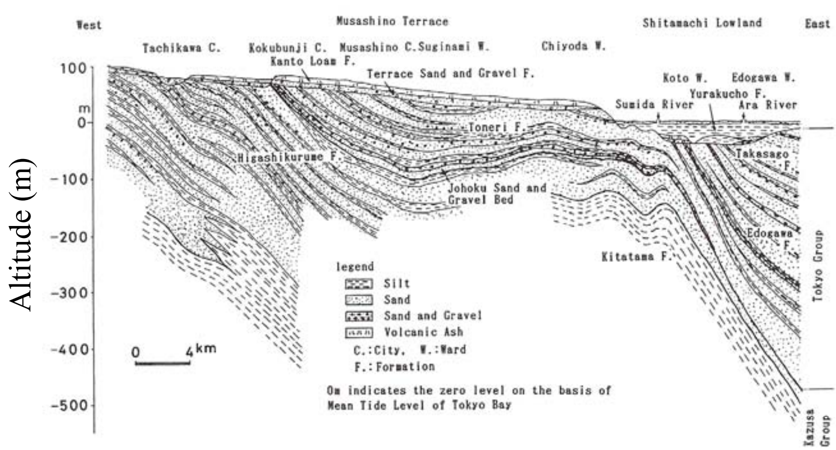

Figure 2: East-west geologic cross section across the Tokyo Metropolitan area (Endo [1]).

The area for groundwater development in the Kanto Plain is about 13,300 km^2 and estimated total storage volume of groundwater resources reaches 500×10^9 m^3. These confined groundwater resources have been mainly exploited for household water supplies and industrial, air-conditioning uses and irrigation and drainage in the Kanto region for nearly eight decades. Total amounts of groundwater use in the Kanto Plain in 1999 were estimated as about 1.2×10^9 m^3/year. Heavy utilization of confined groundwater in the past decades, especially the period of high economic growth in Japan of 1960s, serious problems related groundwater development such as land subsidence, groundwater salinization, oxygen-deficient air accident have appeared over the alluvial low lands and a part of terraces within the plain. Figure 3 shows the Japanese experience of environmental consequences according to the excessive groundwater development.

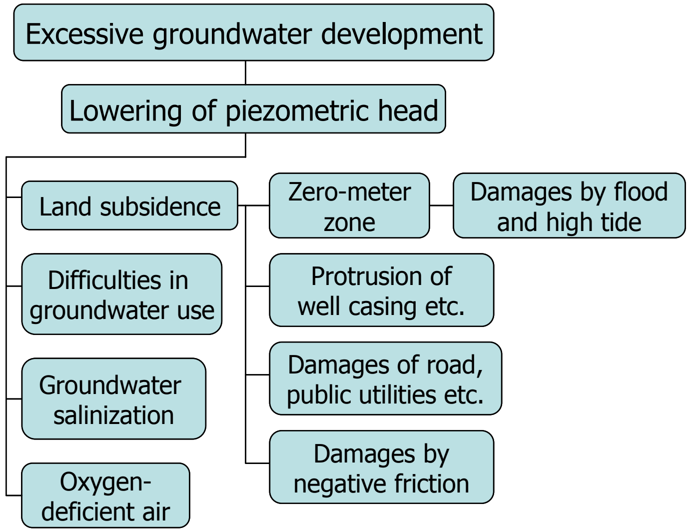

Figure 3: Japanese experience of environmental consequences according to the excessive groundwater development (Kayane [2]).

Figure 4 shows the cumulative land subsidence at main benchmarks in the Tokyo area. Land subsidence has resumed in the middle of 1950s when industries began pumping up large quantities of groundwater to support increased production activity. The cumulative land subsidence at the benchmark of No.9832 amounted to over 4.5 m from 1918 to 2000. Land subsidence in the Tokyo area has been rapidly reduced since about 1973 due to the restrictions for extraction of groundwater by means of the laws and the ordinance mentioned below.

Because of the serious problems related to the excessive groundwater use, the National Government has restricted extraction of groundwater for industrial use by the Industrial Water Law since 1956, and for air-conditioning use by the Law on Regulating the Extraction of Groundwater for Use in Buildings since 1962. The areas regulated by the laws are surround Tokyo Bay, the eastern part of the Tokyo Metropolis, the southern part of Saitama Prefecture, and the western part of Chiba Prefecture, respectively. In addition, local governments such as Tokyo's and each prefecture within the plain have also restricted the drilling of

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new wells in the area not covered by the national laws applying the each prefecture's ordinance. By the recognition of the local governments that these control regulations are highly effective, the area subjected the implementation has been gradually increased.

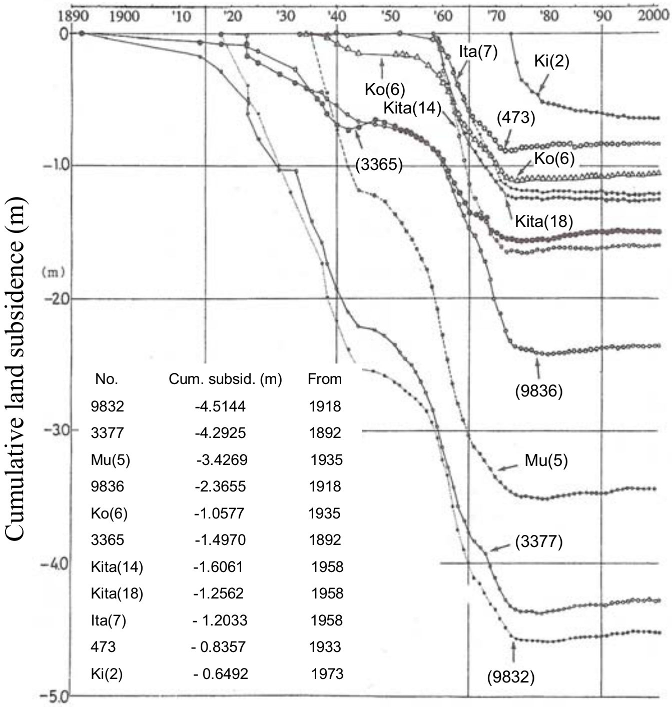

Figure 4: Cumulative land subsidence at main benchmarks in the Tokyo Metropolitan area (Imai [3]).

2 Change of piezometric heads of confined aquifers due to the groundwater development in the Kanto Plain

Figure 5 shows change of the piezometric heads of confined aquifers in the Tokyo area from 1954 to 1999. History of groundwater development in the Kanto Plain is typically shown in this figure. Water demand in the Tokyo and its surrounds had increased after the World War II. The extra demand was fuelled by Japan's high economic growth resulting in over-extraction of the groundwater resource.

In the early 1970s, piezometric heads in several observation wells had declined to more than 40 m below the ground surface.

The groundwater potential distribution of the confined aquifer in the Kanto Plain in 1975 is shown in fig. 6. This figure clearly shows that the large declined piezometric heads centred surround the Tokyo area in the year.

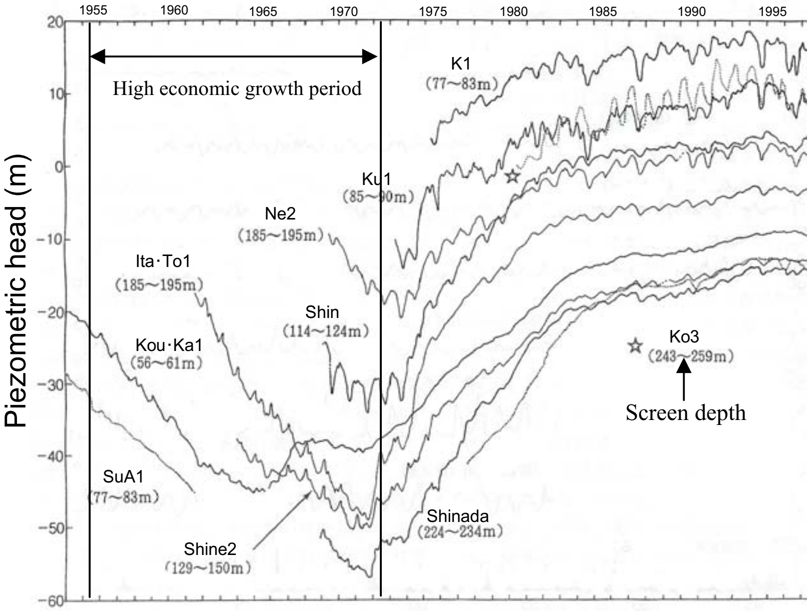

Figure 5: Changes of piezometric heads of confined aquifers in the Tokyo Metropolitan area (corrected Kawashima [4]).

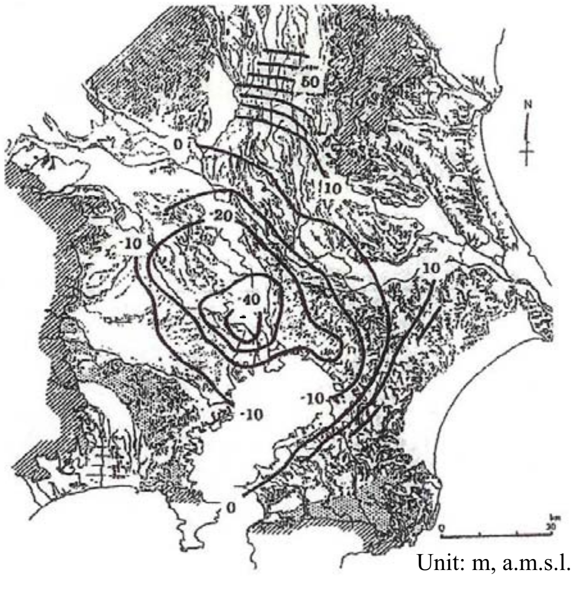

Figure 6: Groundwater potential distribution of the confined aquifer in the Kanto Plain in 1975 (Prefectures Governor's Association of the Kanto Region [5]).

After implementing regulations mentioned above groundwater potentials have recovered so quickly as shown in fig. 5 for better than was expected. Annual precipitation of the Kanto Plain is about 1,300 to 1,600 mm, while the potential evapotranspiration is estimated as 700 to 800 mm/year. Therefore, the residual 500 to 900 mm/year of water can be expected as the potential recharge rate in this region. These values correspond to the daily recharge rates of 1.4 to 2.5 mm. It is believed that because of this high potential recharge rate, the piezometric heads of the confined aquifers have recovered so quickly.

3 Countermeasures to prevent the land subsidence

Two groundwater laws are effective in whole Japan, but practical application of laws to a specific area is decided by the local government. The Tokyo Metropolitan Government has succeeded in reduce the rate of land subsidence by converting water resources from groundwater to surface water and by making legislative guidance in order to save groundwater resources in factories and buildings. However, bordering prefectures such as Saitama, Chiba and Ibaraki in the same Kanto Plain are still suffering from the land subsidence as shown in fig. 7.

This figure shows cumulative land subsidence during the 10-year period from 1988 to 1997 in the Kanto Plain. The center of the land subsidence in the period exists in the northern part of the plain. In 1950s, the land subsidence in the Kanto Plain was most severe in the southern part of Saitama Prefecture, and gradually spread over for areas such as the northern part in Saitama Prefecture, the western part in Ibaraki Prefecture, the north-western part in Chiba Prefecture and the southern part in both Gunma and Tochigi Prefectures. In 2004, a total area of about 26 km^2 subsided 2 cm or more/year in the northern part of the Kanto Plain. The maximum land subsidence recorded in 2004 was 4.7 cm in the southern part of the area (Ministry of Environment [7]).

According to the situation of the land subsidence in the northern part of the Kanto Plain, the National Government has decided to formulate the Guideline for Measures to Prevent Land Subsidence in the Northern Part of the Kanto Plain in November 1991. The purpose of the Guideline is to prevent the land subsidence through comprehensive promotion of policies related to prevent the land subsidence in that area. In the Guideline, the target extraction quantity of groundwater in the prevention area is formulated as 0.48×10^9 m^3/year. The total extraction of groundwater in the area in 2002 was $0.51 \times 10^9 m^3$/year (Ministry of Land, Infrastructure and Transport [8]), and exceeded the quantity than that of the target volume. The target year of the present Guideline was set in the fiscal year of 2000. The National Government has been setting up in the work to improve the contents of the Guideline.

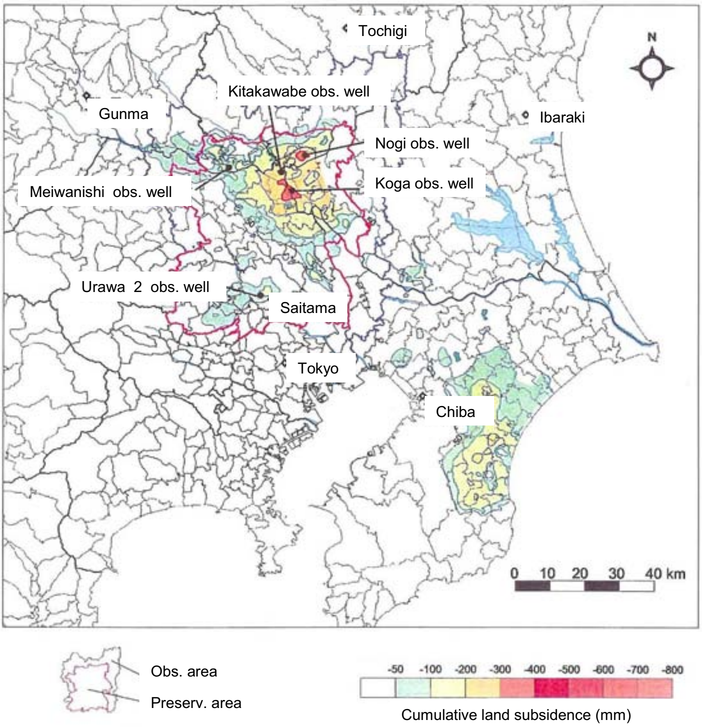

Figure 7: Areal distribution of cumulative land subsidence in the Kanto Plain during the 10-year period from 1988 to 1997 (Ministry of Land, Infrastructure and Transport [6]).

4 Concluding remarks

This paper showed a typical example of excessive extraction of groundwater resources and their resulting problems suffered in the Kanto Plain, the largest tectonic sedimentary basin in Japan. In the case of the Kanto Plain, this problem has been remedied by the implementation of regulations over past 50 years and the relatively high recharge rate due to the natural hydrologic conditions of the area.

For sound groundwater management, the information on pumping amounts in time and space, hydrologic parameters, aquifer characteristics, quantities of irrigation and drainage and so on should be unified so as to make the groundwater resource management effectively. Appropriate legislative and administrative measures are essential for providing background for acting the proper groundwater management. It requires the knowledge of hydrologic balance between the quantity of recharge rate and the extraction of groundwater for the sustainable management of groundwater resources. A balanced budget is the goal for all groundwater managers.

Furthermore, to maintain the continual social and economic growth of the area and to minimize the enlargement of the environmental problems related to the groundwater development, it should establish the continuous monitoring systems such as groundwater levels, quantities of extraction, and the land subsidence as well as monitoring the groundwater pollution.

References

[1] Endo, T., Confined groundwater system in Tokyo. *Environ. Geol. Water Sci.*, **20**, pp. 21-34, 1992.
[2] Kayane, I., Personal communication, 1989.
[3] Imai, T., New policy for groundwater preservation in Tokyo Metropolitan area. *Groundwater Technology*, **44**(3), pp. 14-21, 2002. (in Japanese)
[4] Kawashima, S., Groundwater environment in Tokyo. *Ground-water Technology*, **43**(3), pp. 6-19, 2001. (in Japanese)
[5] Prefectural Governor's Association of the Kanto Region, *Research Report of the Distribution of Groundwater Level in the Kanto Groundwater Basin*. pp. 1-65, 1991. (in Japanese)
[6] Ministry of Land, Infrastructure and Transport, *Evaluation of the Guideline for Measures to Prevent Land Subsidence in the Northern Part of the Kanto Plain*. pp. 1-36, 2002. (in Japanese)
[7] Ministry of Environment, *Outline of Land Subsidence Area in Japan in the FY 2005*, pp. 1-30, 2006. (in Japanese)
[8] Ministry of Land, Infrastructure and Transport, *On the Measures to Prevent Land Subsidence in the Northern Part of the Kanto Plain*. pp. 1-148, 2005. (in Japanese)

Use of geophysical methods in investigating PRBs employing non-conductive reactive materials

Y. S. Kim, J. J. Kim, G. H. Lee & J. B. Park
School of Civil, Urban and Geosystem Engineering,
Seoul National University, Korea

Abstract

Resistivity survey and ground penetrating radar (GPR) were used for investigating a full-scale permeable reactive barrier (PRB) installed nearby abandoned mines of the Go-sung area, located on the south coast of the Korean peninsula. The aims of the testing program included evaluating the applicability of geophysical methods in: 1) locating PRB installations, 2) investigating the movement of groundwater through the PRB and 3) qualitatively identifying the extent of contaminant removal on the PRB. The results indicated that both resistivity and GPR surveys were applicable in giving exact locations of PRB installation based on given information on the reactive material properties, including particle size, composition and electrical characteristics. The resistivity survey was also successful in evaluating the movement of groundwater at different locations nearby the PRB installation. On this basis, results and discussion of the testing program provides evidence of the reliability of geophysical surveys to be used as in-destructive methods for investigating PRBs in operation.
Keywords: permeable reactive barriers, abandoned mines, heavy metals, resistivity survey, ground penetrating radar.

1 Introduction

Permeable reactive barrier (PRB) has gained wide acceptance as an effective technology to remediate a variety of contaminants present in groundwater. The technology has many advantages over traditional methods of remediation, in that it is passive and a large amount of contaminants can be treated in a cost-effective

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manner [6]. In Korea, several PRBs are known to have recently been installed nearby abandoned mines as a means of intercepting possible release of contaminants from mine tailing dumps. While the technology has proven to be an efficient approach to remediate a wide range of contaminants worldwide, a limited number of reports have been made to investigate the performance of PRBs after installation. In the long term, PRBs may experience reduction in efficiency of contaminant removal, reduction in permeability from build up of mineral precipitates, and build-up of microbial biomass [5, 6]. In addition, PRBs can be damaged or may not operate as designed if not properly managed during its years of operation after installation.

A full-scale permeable reactive barrier (PRB) installed nearby abandoned mines of the Go-sung area, located on the south coast of the Korean peninsula was the study site for performance monitoring. This research reports the results of geophysical surveys conducted prior to intensive groundwater and reactive material sampling. While more than 50% of full-scale PRB installations worldwide use granular iron as the reactive media [3], non-conductive reactive media such as steel slag and zeolite were employed at the study site. The aims of the testing program included evaluating the applicability of geophysical methods in: 1) locating PRB installations, 2) investigating the movement of groundwater through the PRB and 3) qualitatively identifying the extent of contaminant removal on the PRB.

The greatest advantage of using geophysical method is that it provides in-destructive means of understanding the subsurface conditions nearby PRB installation. In addition, high resolution images can be obtained through rapid data acquisition. Without such prior investigation, efforts made in order to monitor PRBs involving drilling compliance wells may become laborious and cause costly damage. Interpretations made in this study were to aid in understanding the subsurface characteristics and selecting locations for groundwater and soil sampling.

2 Site description

The study site of PRB installation is located nearby an abandoned mine which was in operation during the early 1950s to the mid 1960s. After the copper mine was abandoned, use of contaminated surface water for household and agricultural supply continued until there were doubts of possible outbreak of a number of diseases from heavy metal poisoning. The needs for taking restoration measures were recognized by the public more recently. The Mine Reclamation Corporation installed vertical walls around the mine tailing dump and a group of pile-type PRBs at an open end of the vertical walls in order to prevent any movement of contaminants out from the mine tailing dump.

A total of four non-conductive reactive materials were used in the pile-type PRB under study, which are zeolite, iron coated sand, steel slag and activated carbon. These reactive materials were selected for remediating groundwater contaminated from acid mine drainage, consisting heavy metals including cadmium and arsenic. Five rows of pile-type PRBs with a vertical length of 6 m and a diameter of 0.25 m were aligned in a zigzag pattern to create a group of pile-type PRBs with a total width of 2.25 m. Reports made prior to installation revealed that the nearby soils and groundwater were primarily contaminated by copper and cadmium, respectively. In addition, mine tailing dump was estimated to contain approximately 1,000 kilograms of arsenic and 5,500 kilograms of copper. Although only 2 years have passed since its installation, the exact location of pile-type PRBs was questionable. This was primarily because the group of pile-type PRBs was installed at a privately owned property which made management difficult. Even with the aid of ground positioning system (GPS) and design data, locating the pile-type PRBs was challenging due to the limited level of precision.

3 Description of geophysical survey

Resistivity survey and ground penetrating radar (GPR) were used for investigating the study site. Prior to conducting geophysical surveys, GPS was employed to aid in marking exact locations of the grid lines. The GPR survey was conducted in an effort to locate the pile-type PRB installations. Therefore a total of 3 grid lines were directed perpendicular to the estimated direction of the PRB installation, as shown in figure 1. The survey was conducted using a 100 MHz Pulse EKKO transmitter and receiver antennae pair. Although the depth of penetration may dependent on the dielectric properties of the underlying soil, it was considered to be sufficient since the pile-type PRBs were designed to be buried at a depth of 1 to 2 m from the surface.

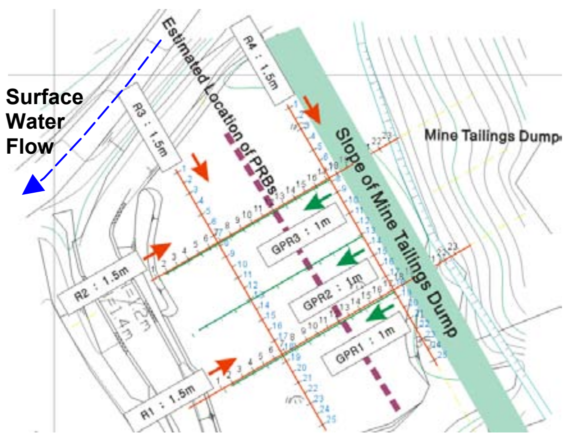

Figure 1: Schematic diagram showing grid lines of geophysical surveys conducted at the study site.

A total of 4 grid lines were selected for resistivity survey. Since the resistivity survey was conducted in an effort to investigate the subsurface conditions and movement of groundwater in addition to locating PRB installation and identifying the extent of contaminant removal, grid lines were directed both parallel and perpendicular to the pile-type PRBs (refer to figure 1). Wenner arrays were employed by placing electrodes 1.5 m apart, which gives a penetration depth of 7.5 m (≒5 times the electrode spacing). Since the surface of the study site contained a significant amount of large gravels and rocks, salt water was discharged in order to increase the contact resistance between the subsurface and stainless steel electrodes.

4 Results

4.1 GPR survey

Results of the GPR images obtained from lines perpendicular to the pile-type PRBs are shown in figure 2. Since 100 MHz antennas were used, the maximum penetration of the radar signals was approximately 3 m, and images obtained at deeper levels were considered to be dominated by noise (thus considered unreliable). Generally, interpretation of GPR results is focused on searching for anomalies such as hyperbolic reflections, irregularities in largely uniform reflection patterns, and changes in frequency of the signals [4]. Hyperbolic reflections are caused by point reflectors in the ground, while irregularities in largely uniform reflection patterns are usually caused by disturbances to the natural sedimentation of soils as a result of construction. Changes in the frequency of radar signals are dominated by the dielectric properties of the subsurface medium, which is primarily effected by the volumetric moisture content.

As shown by the dotted circles of figure 2, the GPR image contains significant number of anomalies, which are predominantly irregularities in largely uniform reflection patterns. These anomalies are believed to be caused by disturbances in soil medium from the construction work performed to remediate and immobilize heavy metal contaminants (including pile-type PRBs and vertical walls surrounding the mine tailings dump). Note that natural sedimentation can not be anticipated at the study site, since original soil was excavated and covered by nearby soils prior to installation of pile-type PRBs. A clear hyperbolic reflection was observed at a low depth in GPR3 image which was later visually confirmed to be a buried steel pipe through excavation. Such hyperbolic pattern is created as the reflected signal appears to rise towards the surface as the transducer approaches and passes over it.

Anomalies seen at (horizontal) locations corresponding to 11-15 m of GPR2 image and 10-14 m of GPR1 image are believed to be pile-type PRBs. It appears at a depth between 1.5 to 2 m. Although the anomalies do not necessarily bear resemblance to the shape or size of the target causing the reflected signal, pile-type PRBs were found to create large trapezoidal shaped anomalies. Differences in the particle size, particle size distribution, and water content of reactive materials from surrounding soils may be the principal reasons which cause such anomalies. However, pile-type PRBs were not observed in the GPR3 image. Such results suggest possible damage in pile-type PRBs at locations corresponding to the GPR3 line.

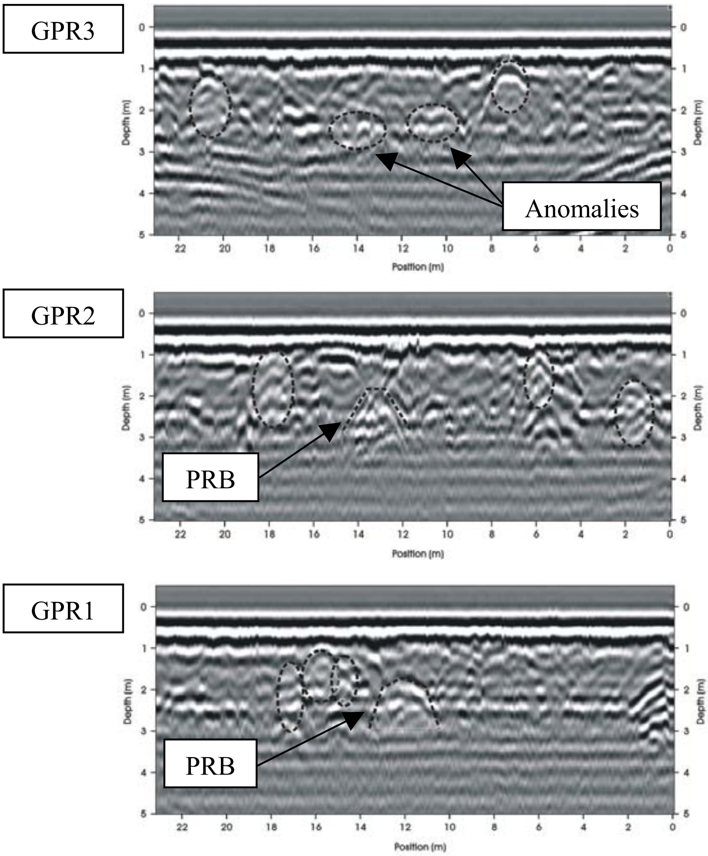

Figure 2: Radar images along grid lines GPR1-3, which are perpendicular to the direction of pile-type PRBs.

4.2 Resistivity survey

Results of the resistivity survey obtained from grid lines perpendicular to the pile-type PRBs are shown in figure 3. Since reactive materials used for pile-type PRB construction had a narrow particle size distribution and large particle size (greater than 5 mm), the electrical resistivity at the location of pile-type PRB was anticipated to be significantly lower than surrounding soils (especially densely compacted original soil). Greater porosity or volumetric water content of reactive materials brings about a decrease in electrical resistivity. Employing an electrode spacing of 1.5 m, resolution of the resistivity image was expected to be sufficient to represent pile-type PRBs as low resistivity areas with widths of 2 to 4 m. Considering that the pile-type PRBs were installed to depths reaching 7 to 8 m where the original soil was intact, its approximate location was found as indicated by the black dotted rectangles in figure 3. The original soil was estimated to lie at a depth of 3 to 4.5 m, under the assumption that it showed resistivity values greater than 84Ω·m. Note that the estimated location of the pile-type PRBs based on 2-D resistivity images are in accord to that from interpretation of GPR images. In addition, buried steel pipe reacted to develop an area of low resistivity in grid line R2.

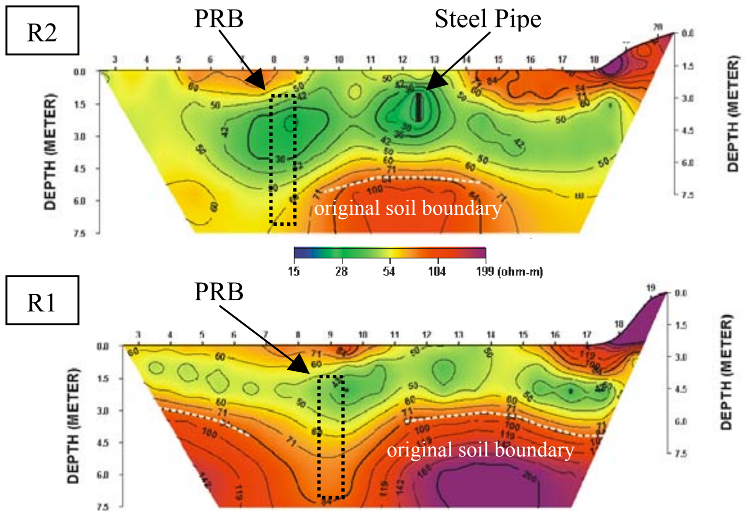

Figure 3: 2-D Resistivity images along grid lines R1 and R2 which are perpendicular to the direction of pile-type PRBs.

Assuming that the pile-type PRBs are located at the estimated locations shown in figure 3, no significant contrast in electrical resistivity was observed at the areas following its two opposite faces. Therefore, it was interpreted that either the contrast in resistivity was not sufficient enough to map any adsorption of heavy metals within the pile-type PRBs, or the pile-type PRBs are not experiencing any inflow of groundwater contaminated by heavy metals from the mine tailing dump. Laboratory analysis on groundwater samples is required for a clear verification. Based on the resistivity survey results, the pile-type PRB was found to be installed at adequate depths to intercept possible release of contaminants from the mine tailing dump.

Results of the resistivity survey obtained from grid lines parallel to the pile-type PRBs are shown in figure 4. Similar to the results of figure 3, the reclaimed soil above original soil at the study site generally showed low resistivity values. High values of resistivity observed at depths above 1.5 m in gridline R4 can be attributed to the presence of the nearby mine tailing dump which is dominantly composed of large gravels and rocks.

Since bulk resistivity of soils can be estimated to be directly proportional to the porosity (which is volumetric water content at saturated state) and the electrical resistivity of the pore water based on the Archie's law [1], areas of low resistivity shown in image obtained from gridline R4 are believed to be areas that act as passageways for groundwater from the mine tailing dump. With resistivity values up to 21 Ω•m, these areas may contain various dissolved ions and possibly heavy metals released from the mine tailing dump. Therefore, they were selected as locations of high priority for groundwater and soil sampling to be performed.

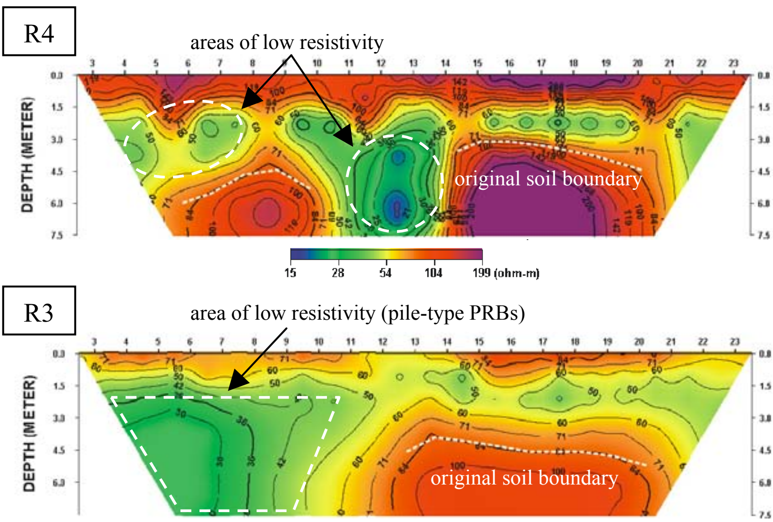

Figure 4: 2-D Resistivity images along grid lines R3 and R4 which are parallel to the direction of pile-type PRBs.

Area of low resistivity shown in image obtained from grid line R3 is believed to be a part of the pile-type PRBs installation. Although the gridline R3 was initially planned to be placed sufficiently away from the pile-type PRBs, the estimated location of the pile-type PRBs from GPR images and 2-D resistivity images R1 and R2 were found to be farther away than expected from the mine tailing dump. In addition, the estimated direction of the pile-type PRBs were not parallel with the mine tailing dump (refer to figure 5). As a result, grid line R3 is believed to partially overlap the location of pile-type PRBs. Another possible cause for the area of low resistivity may be inflow of surface water with higher electrical conductivity than water. A small stream runs perpendicular to the grid line R3, as shown in figure 1. Analysis on groundwater and surface water samples from the study site is required for a clear verification.

5 Discussion

Excavation to depths of 1 to 2 m were performed above anomalies observed in GPR images and regions of low resistivity observed in 2-D resistivity images, in order to verify the exact location of the pile-type PRBs. It was confirmed that the interpretation of the results of geophysical surveys were highly reliable. As shown in figure 5, the pile-type PRB was approximately 12 to 15 m away from, and not completely parallel to the slope of mine tailings dump. Use of geophysical surveys to locate PRBs employing non-conductive reactive materials is highly effective, since it can substitute or minimize site excavation (based on design plan), which is a destructive method and may cause costly damage. Figure 5 is a schematic diagram giving a summary of interpretations made based on geophysical surveys.

Interpretations of the 2-D resistivity images were successful in giving approximate locations of the original soil and reclaimed soil boundaries. Such geological information provides knowledge on possible spatial variations in the amount of groundwater flow nearby PRB installations. In addition, areas of low resistivity which can be interpreted as areas of high porosity (thus high water content at saturated state) or high concentration of dissolved ions can be selected as locations adequate for groundwater sampling. Such interpretations made from resistivity images become effective especially when the site subject to investigation lies over a large area.

Figure 5: Schematic diagram showing the confirmed location of the PRBs from excavation. Dotted circles represent areas of low resistivity and black x symbols indicate of anomalies in GPR images.

It was difficult to investigate possible migration of heavy metal contaminants in the groundwater by interpreting the 2-D resistivity images obtained perpendicular to the PRBs installation and comparing resistivity images obtained from front and back ends of the PRBs installation (for example, images shown in figure 4). The primary reason was due to soil excavation and reclamation conducted at the study site prior to pile-type PRBs installation which caused spatial variations in soil properties such as soil type and bulk density. In addition, the pile-type PRBs at the study site may not be experiencing sufficient inflow of conductive heavy metal contaminants. Assuming that there are no lithological variations, Benson and Turner [2] reported that at least a 5-10% electrical contrast between contaminated and uncontaminated soil is required to successfully map a contaminant plume. However, it is suggested that repeating resistivity surveys over specific time intervals can be effective in investigating and monitoring the performance of PRB installations. By repeating surveys using on identical grid lines, comparison of resistivity images may be done quantitatively to give more promising results.

6 Conclusion

Geophysical surveys were performed on full scale pile-type PRB installation in order to investigate their applicability in providing information on the location of PRBs, nearby subsurface conditions including the movement of groundwater and qualitatively identifying the performance of PRBs. Main conclusions derived from the interpretation of the field study can be summarized as follows.

1) Use of geophysical survey to locate PRBs employing non-conductive reactive materials is highly effective, since it is a non-destructive method which can substitute or minimize site excavation based on the design plan.

2) Interpretations of the 2-D resistivity images were successful in giving approximate locations of the original soil and reclaimed soil boundaries. In addition, groundwater movement was estimated based by interpreting areas of low resistivity which may be viewed primarily as areas of high volumetric water content or areas that contain high total dissolved solids.

3) It was difficult to investigate possible migration of heavy metal contaminants in the groundwater by interpreting the 2-D resistivity images obtained perpendicular to the PRBs installation. However, it is suggested that repeating resistivity surveys over specific time intervals can be effective in investigating and monitoring the performance of PRB installations.

References

[1] Archie, G. E., The electrical resistivity log as an aid in determining some reservoir characteristics. *Trans. AIME*, 146, pp.54-62, 1942.

[2] Benson, R.C., Turner, M.S., Volgelsong, W.D. & Turner P.P., Correlation between field geophysical measurements and laboratory water sample analysis. *Proc. Surface and Borehole Geophysical Methods in Groundwater Investigations,* pp. 178-197, 1985.

[3] Field Applications of In Situ Remediation Technologies: Permeable Reactive Barriers, U.S. Environmental Protection Agency, Office of Solid Waste and Emergency Response Technology Innovation Office, 2002.

[4] Hunaidi, O. & Giamou, P., Ground-penetrating radar for detection of leaks in buried plastic water distribution pipes. *Seventh International Conference on Ground Penetrating Radar,* pp. 783-786, 1998.

[5] Liang, L., Korte, N., Gu, B., Puls, R. & Reeter, C., Geochemical and microbiological reactions affecting the long-term performance of in situ barriers. *Advances in Environmental Research*, 4, pp. 273-286, 2000.
[6] Wilkins, R. & Puls R., Capstone report on the application, monitoring, and performance of permeable reactive barriers for ground-water remediation: Volume 1 – Performance evaluations at two sites, U.S. Environmental Protection Agency, Ground Water and Ecosystems Restoration Division, National Risk Management Research Laboratory, 2003.

Modelling an aquifer's response to a remedial action in Wadi Suq, Oman

O. Abdalla[1], O. Satti[2], K. Al-Rawahy[2], A. Kacimov[3] & H. Al-Zarie[2]

[1]*Department of Earth Sciences, College of Science, Sultan Qaboos University, Oman, UAE*

[2]*Department of Petroleum and Chemical Engineering, College of Engineering*

[3]*Department of Soils and Water, College of Agriculture and Marine Sciences, Sultan Qaboos University, Oman, UAE*

Abstract

The present study simulates a predictive alluvium-aquifer's response to a proposed remedial action in the Wadi Suq area, Oman. Contamination due to copper mining and smelting, which was incepted in 1982 and ceased in 1993, has severely impacted the groundwater quality down the gradient. The major concern is the increase of salinity as abundant seawater has been used in mining. Saline plume of an average 35000 mg/l TDS has formed down the gradient from the dam site and spread towards the coast that represents a hydrogeologic boundary. The contaminated aquifer, in which groundwater flows for a path of 34 km to the coast, is composed of unconsolidated alluvium, predominantly gravels and sands in its upper part and of weathered ophiolite in its lower part. The massive fresh ophiolite represents the lower hydrogeologic boundary. Though lithologically different, the aquifer units have been simulated as one unit since the hydraulic properties are more or less similar.

Remedial efforts that had been carried out so far were deemed ineffective. The present study proposes a remediation measure by injecting freshwater via wells into the aquifers. Head and concentration values for the year 1993 were used as initial heads/concentrations for the steady flow simulation in which the computed heads/concentrations were matched with the observed ones, whereas transient flow runs were conducted to predict aquifer's response with time. Simulated aquifer response to the injection of freshwater indicates the reduction of TDS with time and the movement of the plume to eventually discharge into the Gulf of Oman. Consequently, the proposed remedial action, though financially challenging, is effectual.

Keywords: Oman, mining, Wadi Suq, MODFLOW, groundwater modelling.

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1 Introduction and background

Oman Mining Company (OMC) has launched large copper mining activities in 1980 in Wadi Suq, Sohar area, northern Oman. An unlined tailings dam, upstream of Wadi Suq, was founded in 1982 and followed in 1983 by the construction of a pipeline to transport seawater from the coast to the processing plant [1]. Copper was processed using flotation method and a total of 15000 tons of ore was mined annually. Ore processing and waste disposal of continued until 1994 to generate about 11 million tons of sulphide rich tailings. Since project's inception till the ban of the seawater usage in 1993, about 5 mm^3 of seawater being disposed of in the tailings dam. Consequently, elevated salinity (high total dissolved solids (TDS)) has been observed in the groundwater down the gradient, pH changed and mobility of certain heavy metals increased. Although, the groundwater pollution was detected as early as 1983, mining activities continued and the situation worsened [1]. Several observation wells were installed down the gradient from the dam (> 40 wells) to monitor the pollution. Periodic and extensive sampling and analyses of water collected from the wells continue since the detection of the pollution in 1983 yielding a comprehensive hydrochemical database. Data collected included but not limited to measuring TDS, EC, concentration of major ions and trace elements. The pollution has severely impacted the smaller settlements in the region that were totally dependent on groundwater, which is no longer suitable for drinking or irrigation.

The area was the focal point of several studies carried out after the discovery of the pollution [1–4]. The main objectives of these studies were to assess and identify the extent of the pollution. Application of remediation techniques and evaluation of the currently taken measures were not assessed or studied. Therefore, the present study aims to shed some light on the remediation proposal and pollution cleanup. The present study simulates the aquifer response to a proposed remediation action. Injection of fresh water via wells in the upper catchment of the wadi is proposed to dilute and disperse the contaminants. The study models the groundwater flow and solute transport to predict the future aquifer response and to measure the validity of the proposal.

Wadi Suq is located in north Oman and is on the eastern side of the Hajar Mountains (fig 1A). The area is characterized by arid climate and variable topography. Highest elevation of about 300 m above sea level is reported in the mining site while the plain down towards the Gulf of Oman is a few meters above the sea level. The length of the Wadi is about 34 km, travels towards the Gulf of Oman, at a slope of 0.008 and has a 70 km^2 catchment area. The wadi is filled of Quaternary alluvium of unconsolidated sand and gravels. Highest elevation of about 300 m above sea level is reported in the mining site while the plain down to the coast is a few meters above the sea level.

In continuous remediation efforts and to contain the problem, OMC first constructed a cut off trench #1 in 1983 right after the tailings dam. Groundwater was pumped from the trench and discharged into a lined evaporation pond to prevent further contaminants flow and to reduce the level of salinity. The pumped water is left in the pond for complete evaporation and solid residues are disposed of. Another trench "T2" was constructed in 1992 when the remediation results of T1 were not satisfactory. OMC has lately realized that seepage from tailings impoundment poses a great threat to the groundwater resources. Pollutants leach into groundwater subsequent to precipitation, especially storm events, will severely impact the quality of soil and groundwater. Consequently, the tailings were covered with lining to prevent further leaching and infiltration. To keep the pH at the desired level (ca 7) to prevent mobility of toxic and heavy metals, lime was added to the tailings to reduce the acidity of mine drainage.

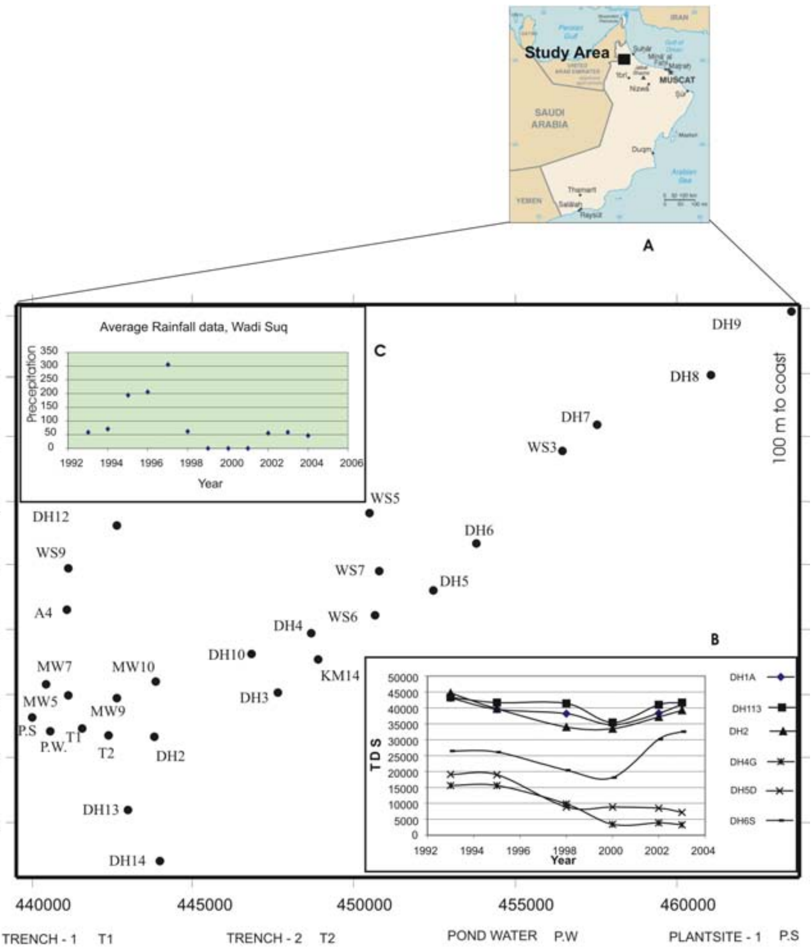

Figure 1: A-location map of representative monitoring wells. Location of the trenches and plant site is also indicated in the map. B- Measured TDS ppm for selected monitoring wells since 1993 to 2003. C- Average precipitation in Wadi Suq since 1993. High values occurred around 1995-1998.

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1.1 Remediation approaches

The remediation efforts that have been carried out thus far in Wadi Suq are ineffective. The approaches used are similar to pump and treat approach but differ in that water is left for evaporation; treated water is not pumped back to the aquifer. Taking into consideration that annual groundwater recharge is minimum; the quality of polluted groundwater if not worsens it will not improve. It is therefore expected that TDS and concentration of several ions remain either unchanged or fluctuating within similar ranges. Fig 1B shows the measure of TDS against time from selected wells in the area. The general trend shows a total decrease in salinity after 1998 and a rebound after 2002. This phenomenon is explained by a comparison with the total precipitation in the area (Fig 1C). The general trend of increasing/decreasing salinity observed in Fig 1B is correlated with the precipitation trend shown in Fig 1C. It can be clearly deduced from the figures that the reduction in the TDS was mainly due to the increase in annual rainfall, whereas the increasing observed salinity in the following years is attributed to the drought during these years. Thus, natural corrective action to the pollution problem has been observed and increasing rainfall will effectively lower the concentration of pollutants. Hence arose the current remediation proposal, which employs dilute and disperse approach to Wadi Suq area. The idea is to inject fresh water into the aquifer using artificial means; if toxic substances are sufficiently diluted they will be rendered harmless. On the other hand artificial recharge of groundwater from injection wells will increase the hydraulic gradient and the flow velocity and therefore enhance the plume movement by advective means. While the concentration of toxic substances is diluted, the plume will be moving towards the ocean (coast is 34 km from the site) and will eventually vanish. The dilute and disperse approach in the Wadi Suq can be proposed as:

- use the existing pipeline previously employed to transport water from the coast;
- use water from desalination plant/s on the coast;
- construct injection wells on the upper catchment;
- inject water of low TDS into the aquifer.

The cost of the above proposal is the minimum since the pipeline is already available and desalination plants are operating. The added cost will be for wells installation and the desalinated water. The average cost of the desalinated cubic meter in Oman ranges between 400-600 Piza ($1.2-$1.6). Currently potable water is transported to the households in the scattered settlements, the cost of which can be integrated with remediation cost.

2 Modelling the hypothesis

The proposed remediation action by injecting freshwater into the aquifer needs to be tested before implementation. The present study simulates aquifer's response to the remediation proposal and checks against the validity of the action. The model will use the USGS MODFLOW 2000 and go through the following steps.

2.1 Build the conceptual model

The model is one layer case with an area of about 550 km^2 subdivided into grids (22 rows and 46 columns). Grids are closely spaced at the site of the dam to enhance the accuracy of head and concentration calculation. Geology of the area is characterized by variable fracture systems running along and across the wadi whereas the subsurface lithology is simple and uniform. The quaternary alluvium that is composed mainly of gravels and sand comprises the main aquifer system. The alluvium layer occupies the upper section and thickens towards the Gulf of Oman. It is overlain by a thin layer of superficial deposits. Tertiary limestones reported at the peripheries of the wadi occasionally extend in form of lenses intermingle with the alluvium. These limestone lenses were described in a few drilling reports of the monitoring wells. The alluvium is unconformably overlying a sequence of ophiolite rocks (Semail Ophiolite) opducted during the Cretaceous. The ophiolite is composed of gabbroic and basaltic rocks, sheeted dykes, dunite and harzburgite. The copper mining is from the ophilitic rocks that are characterized by sulphide mineralization and rich in Fe and Cu in particular and Al, Mn, Zn, Pb, Ni, Cd and Cr in general. The upper part of the ophiolite is highly weathered which enhances permeability and porosity. The hydraulic properties of the weathered ophilite are similar to that of the alluvium and therefore the two units are considered one hydrogeologic unit.

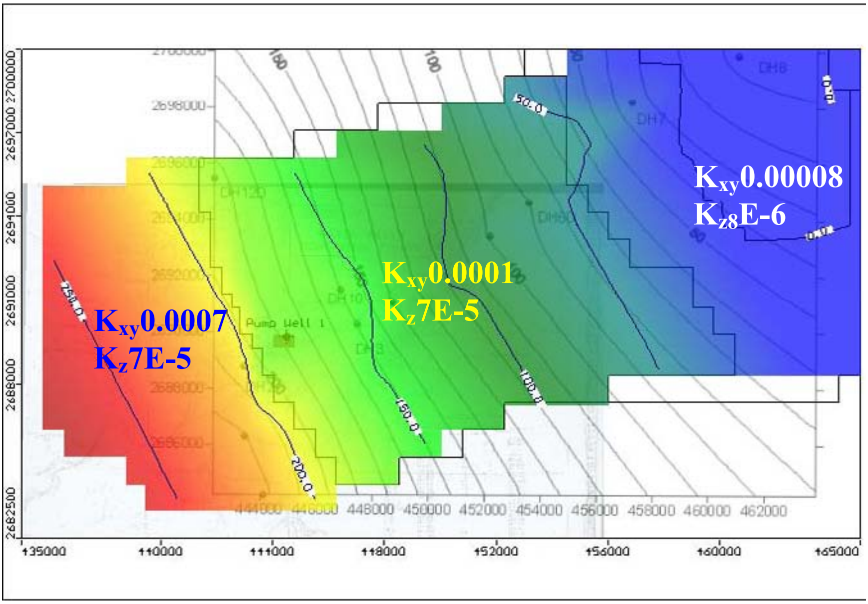

Figure 2: Computed head values after the steady state flow simulation matched with the observed head values in year 1993.

Based on rainfall data and soil cover, 50 mm/year is assigned to the upper catchment. Evapotranspiration 1100 mm/year is assumed throughout the study area with an effective depth of 10 m. The model area is subdivided into 3 zones with reference to hydraulic conductivity (K) value: the upper, middle and lower. The K values (Fig 2) decrease towards the coast as the fines increase. Constant head boundary (≈ 0 m) is assigned to the coastal zone. The aquifer is surrounded from the other 3 sides by crystalline impermeable igneous rocks and thus no flow boundary along these margins was assumed.

2.2 Steady/transient simulation

Head simulation for one day produced initial head values used in the transient run. Computed heads during this stage matched with the observation heads measured in the year 1993 (Fig 2). Simulation of flow and solute transport for the period 1993-2006 utilized these heads together with the initial concentrations of TDS measured in year 1993. Aquifer storage, bulk density and dispersion values were continuously readjusted till the best fit between the computed and observed concentrations/heads were in best match. This best match was achieved with the values: bulk density = 1700 kg/m^3, dispersion = 0.01, specific storage = 0.0001, specific yield = 0.08, effective porosity = 0.08 and total porosity = 0.15.

2.3 Predictive transient simulation

The calibrated values obtained during the previous steady and transient runs are used in this simulation to predict aquifer's response to the injection of freshwater via wells for the period 2006-2016. Four injection wells next to the dam were proposed with time variant discharge (1500-500 m^3/d) to account for well loss and efficiency reduction with time. The injected water has an assumed average TDS of 500 mg/l which is the maximum expected for Oman desalinated water.

3 Results and discussions

The simulation has shown that the TDS plume migrates towards the coast with falling concentration (Fig 3). While the plume is moving with the advective transport at the speed of the groundwater, injected freshwater dilute the polluted groundwater and significantly reduces the TDS with time. The extent of the plume (TDS >15000 ppm) will only be limited to a circle >500m diameter after 10 years of remediation. Only 4 wells were used during the current simulation with total discharge of 1500-500 m^3/d. Increasing the number of the wells and their discharge will exponentially decrease the time required for the cleanup. If more freshwater is added, the hydraulic gradient will increase and consequently speeds up the advective transport of the plume on one hand and dilute the concentration on the other. The mass balance simulation has also shown the significance of the proposed action in reducing the total mass of the solute in the aquifer (Fig 4). The total mass of solutes (sources in) that enter the system is much less than the total mass of the solutes (sinks out) that leave the system. About 10^9 kg of mass will leave the system after 10 years of remediation.

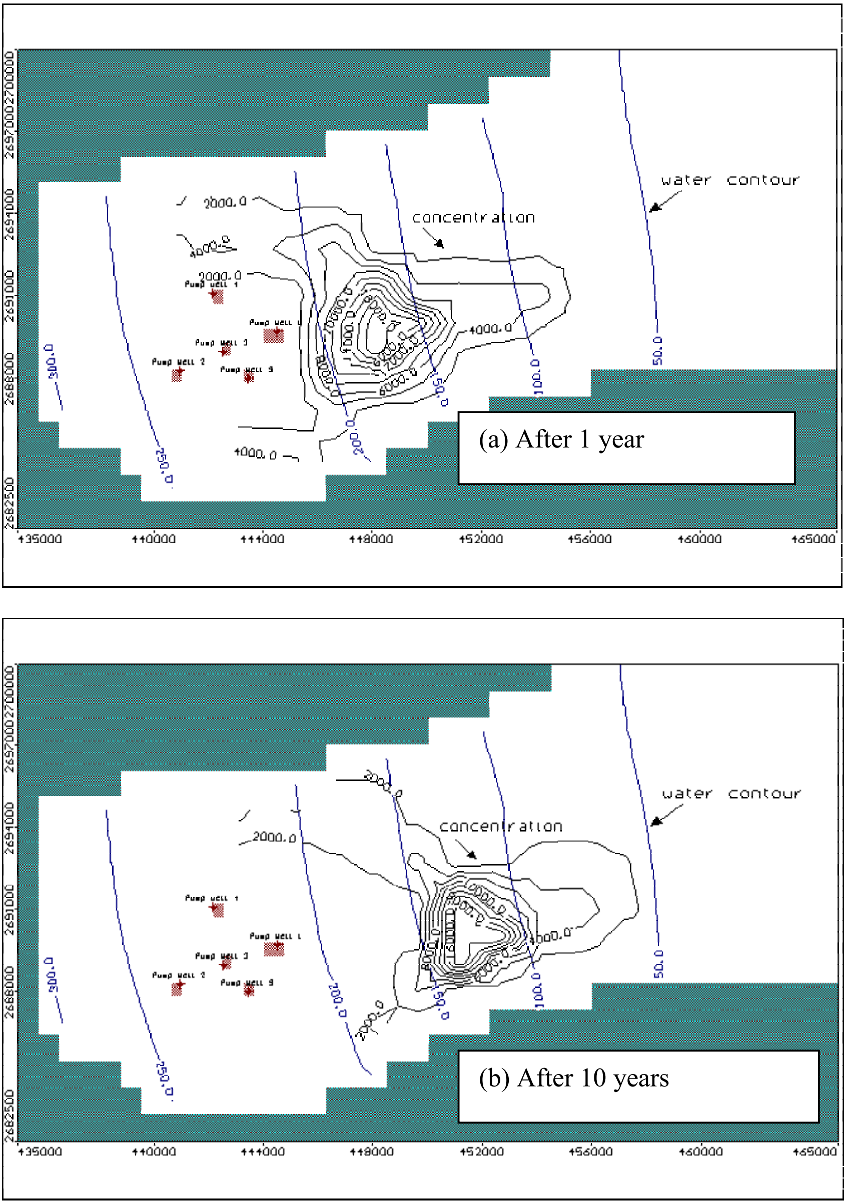

Figure 3: TDS plume moving towards the coast. The figure shows the concentration and plume location after one year of remediation (a) and 10 years of remediation (b).

The above simulation, though applicable and effective, has significant shortcomings that need to be thoroughly considered before any future implementation. The area is characterized by faulting and fracturing systems that enhance the advective transport and also may lead to uncontrolled leakage from the aquifer into the country rocks. This leakage will cause unaccounted for pollution to presently cleaner areas. Conduits in form of faults and fractures should be precisely identified and sealed before the injection of freshwater. Moreover, an estimate of dispersion as a non-measurable parameter adds uncertainty to the computation. The proposed action will on the contrary provide recharge to replenish the ever since depleting groundwater resources, reduce the toxicity in the region, revive the farms and agricultural activities, relief the financial burden of transporting water supplies to locals and before all cleanup the toxic environment.

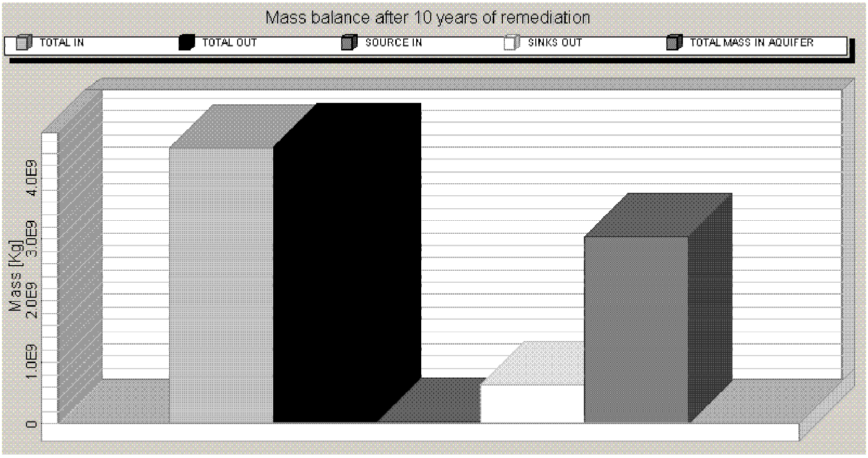

Figure 4: Mass balance after 10 years of remediation.

4 Conclusions

The current simulation has indicated that the total mass of the solutes in the alluvium aquifer of Wadi Suq will reduce significantly with time. The proposed remediation action will provide source of aquifer recharge in addition to the cleanup of the environmental problem. As the infrastructure is already available, injecting water into the aquifer is economically feasible.

Acknowledgements

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References

[1] Ministry of Water Resources, Sultanate of Oman (MWR). 1996. Groundwater Pollution and Remediation in Wadi Suq. Ministry of Water Resources, Muscat, Oman.

[2] Japan International Cooperation Agency (JICA) and Ministry of Commerce and Industry, Oman. 2001. The Feasibility Study on Mine Pollution Control in Sohar Mine Area, Sultanate of Oman (Final Report). Mitsubishi Materials Natural Resource Development Corp, E & E Solutions Incorporation, Japan.

[3] Sharma, R.S and Al-Busaidi, T.S. 2001. Groundwater pollution due to a tailings dam. Engineering Geology 60, 235-244.

[4] Satti, O.; AlRawahi, K.; Kacimov, A.; Abdalla, O. and Al-Zarie H. 2006. Mining related groundwater contamination and its impact on land usability at Wadi Suq, northern Oman. The International Conference on Economic Incentives & Water Demand Management, Sultan Qaboos University, (CAMS), Muscat 18-22 March.

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Hydrochemical evaluation of a heavy contaminated shallow aquifer diluted by Delice River waters, Central Anatolia, Turkey

M. Çelik
Department of Geological Engineering, Engineering Faculty, Ankara University, Turkey

Abstract

The aim of this study is to hydrochemically evaluate the dilution with infiltrating waters from the Delice River into a shallow alluvium aquifer which is heavily contaminated. Whereas the hydrochemical content of the Delice River water has a conductivity of 1,610 μS/cm and total dissolved solids of 938 mg/l, the hydrochemical content of the shallow aquifer groundwater displays spatial variations in conductivity (between 2,340 and 9,360 μS/cm) and total dissolved solids (between 1,669 and 5,957 mg/l).

The hydrochemical parameters (Na^+, SO_4^{2-}, Cl^-, boron, HCO_3^-), which characterise the different components of the groundwater, allowed the determination of the origin of groundwater contamination. The shallow alluvium aquifer groundwater contamination has been partly diluted by the Delice River waters.

The origin of the contamination is Çevirme Formation lithologic units, lying southwest of the study area, underlying the aquifer and consisting of claystone and gypsiferous marls.
Keywords: Delice River, shallow aquifer, surface water/groundwater relation, hydrochemistry, lithologic contamination, dilution.

1 Introduction

The study area, 190 km east of Ankara (Turkey), is located in the Kızılırmak basin (fig. 1). Surface water of the area is represented by the Delice River and Cender creek. The Delice River extends along the shallow alluvium aquifer and has a hydraulic connection with the aquifer.

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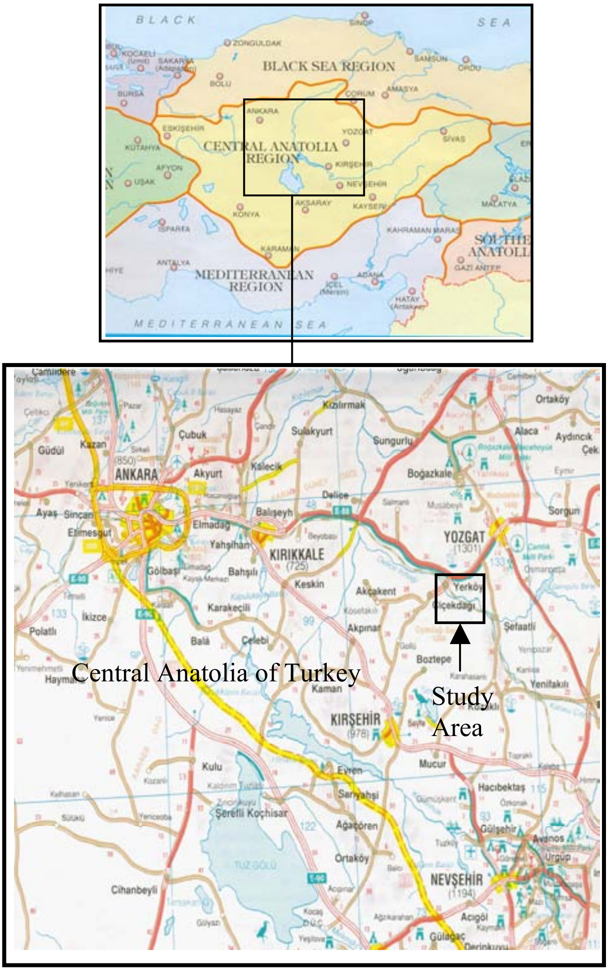


Figure 1: Location maps of the study area.

The aim of this study is to hydrochemically evaluate the dilution with infiltrating waters from the Delice River into the shallow alluvium aquifer which is heavily contaminated. In this regard, a total of 3 thermal and mineral, 3 surfaces and 13 groundwater samples were collected in table 1. Temperature (T), pH, electrical conductivity (EC) and total dissolved solids (TDS) were measured in the field. Salinity problems have been also studied and suggested the best water structure in the other parts of the Kızılırmak basin (Çelik and Yıldırım [1]).

Table 1: Hydrochemical analyses results of the waters (sampling date: 14 November 1998) (Çelik [4]).

Sample No	pH	T (°C)	Ca^{2+}	Mg^{2+}	Na^{+}	K^{+}	Cl^{-}	SO_4^{2-}	HCO_3^{-}	TDS (mg/l)	EC (µS/cm)	Boron (mg/l)
Cender creek												
C1	-	-	200	74.4	635	19.7	450	1,220	256	2,551	4,230	1.76
Delice River												
D1	8.0	-	93.6	29.7	144	6.8	154	179	349	876	1,378	0.30
D2	8.3	-	100	41.3	220	11.2	177	262	345	938	1,610	0.40
Alluvium aquifer												
AB13	8.2	14	118	73.9	195	6.5	93	508	432	1,323	2,340	0.30
YK6	6.5	15.9	177	72.9	635	18.1	510	933	481	1,742	4,770	1.51
IHL	8.7	15.4	94.4	48.1	465	20.0	326	603	400	1,994	3,500	0.97
SE9	7.5	16	128	84.6	294	10.0	246	563	479	1,669	2,760	0.50
DO4	7.6	15.9	124	62.7	393	11.3	336	544	471	1,975	3,420	0.49
HIP12	7.4	15.3	182	33.5	295	8.0	180	565	493	1,993	3,170	0.78
BG3	7.0	13.7	151	35.5	510	21.6	415	775	424	2,469	3,940	1.40
KC11	7.4	15.7	368	112	475	8.9	331	1,784	257	3,360	5,760	0.95
MK7	6.6	15.5	144	63.2	715	17.0	672	1,070	329	3,062	5,220	1.50
OA5	6.5	15.7	334	140	980	26.0	562	2,323	395	4,000	6,600	1.82
MK17	6.8	14.5	506	165	1,080	25.0	615	3,019	248	4,751	8,250	3.13
HK	6.9	16.4	384	86.5	1,310	6.5	580	2,880	657	5,345	8,060	2.85
HK10	6.6	12.4	388	151	1,500	8.7	947	3,237	465	5,957	9,360	2.22
Thermal waters												
B1	6.6	43.7	154	1.5	1,500	115	2,080	363	750	5,324	8,820	3.46
U1	7.8	43	709	31	2,560	70	4,876	377	59	9,286	15,000	4.94
KO1	6.7	36	1,260	12	4,320	120	7,749	305	73	13,762	19,600	7.44

2 Hydrogeology and hydrology

Gündüz [2] explained that the oldest rock in the study area is composed of rhyolite, rhyodasite, and dacite. These units, called the Kötüdağ volcanite member, are fractured and fissured. Canik [3] states that the contact between the volcanite member and granites is suitable for deep groundwater circulation by means of permeability. The Çevirme Formation, consisting of conglomerate, limestone, claystone, and gypsiferous marls, unconformable overlies the Kötüdağ volcanite member. This formation is overlain by the Deliceırmak Formation, which is composed of conglomerate, sandstone and siltstone (fig. 2). The alluvium is composed of gravel, sand and clay sized materials. According to Çelik [4], while the Delice River recharges into the alluvium aquifer about at the centre of the river (near D2), the Delice River is recharged by alluvium aquifer at the lower part of the river (fig. 3).

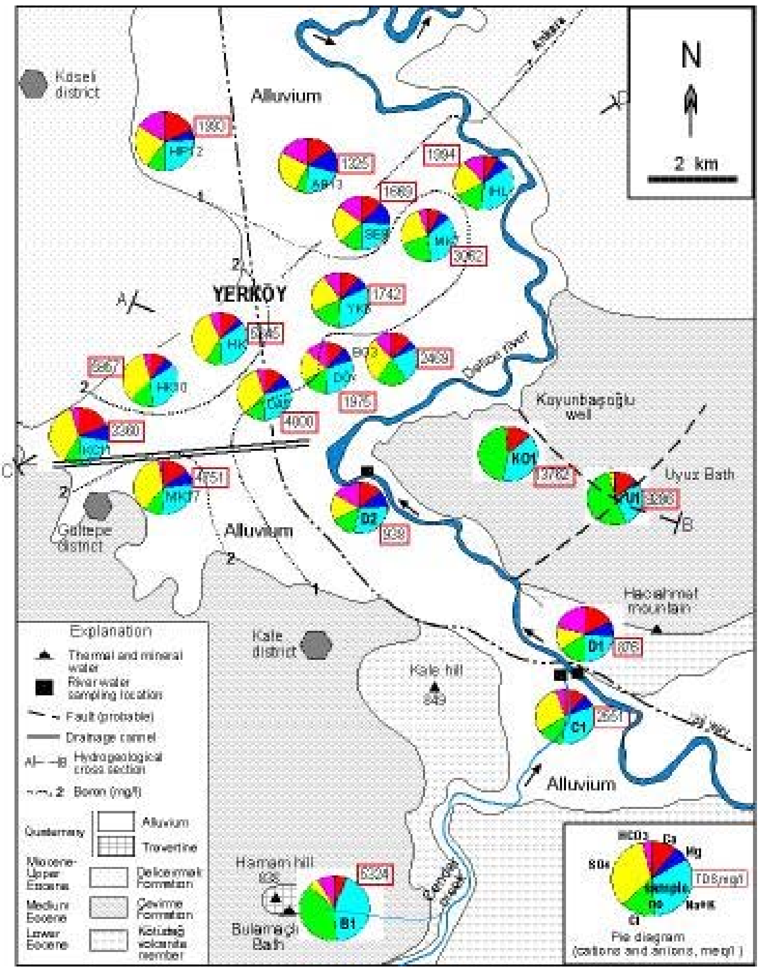


Figure 2: Hydrogeological map of the study area.

There are three types of waters in the study area. Those are surface waters, shallow alluvium aquifer waters and, thermal and mineral waters. The Delice

River and the Cender Creek comprise the surface water network of the study area. The Cender Creek joins the Delice River in the southeast of the area. The Delice River flows through the eastern part of the alluvium aquifer (fig. 2). Discharge of the Cender Creek is estimated as 10-30 l/s. On the basis of measurements conducted at D1 site of the Delice River, discharge was found to be 4.94 and 9.47 m^3/s for November 1998 and May 1999, respectively by Çelik [4]. Wells drilled in the alluvium aquifer are generally shallow (10-15 m). Bulamaçlı bath (B1), Uyuz bath (U1) and Koyunbaşoğlu bath (KO1) are of thermal and mineral waters. According to Canik [3], Bulamaçlı bath spring issues through the fault. Discharge of the spring is 1.45 l/s and its temperature is 44.5°C. According to Gündüz [2], the well for the Uyuz bath is located on two buried faults (fig. 2). The water temperature of Koyunbaşoğlu well was measured as 36°C in November 1998 and 36.9°C in May 1999 (Çelik [4]).

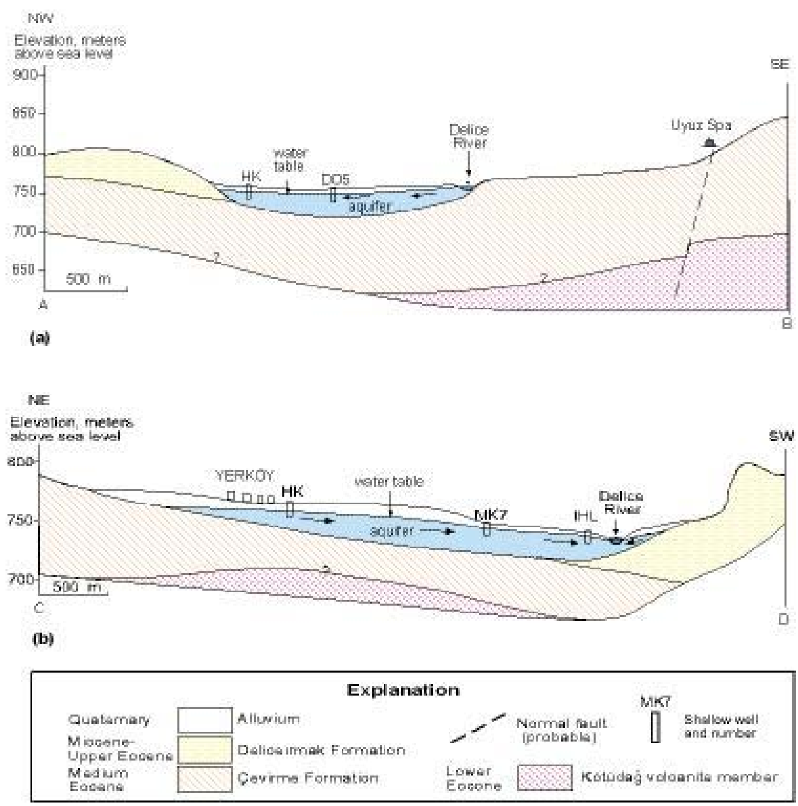


Figure 3: Hydrogeological cross-sections of the study area (modified from Çelik [4]).

3 Hydrochemical evaluations

On the basis of results of the water analyses, Schoeller diagram is drawn (fig. 4). Thermal and mineral waters have Na^+ and Cl^- hydrochemical facies (Na+K>Ca>Mg; $Cl>SO_4>HCO_3$). The Delice River waters have dominant Na^+ cation, but have not any dominant anion (fig. 4). The anions are about same level in the river waters. Shallow alluvium aquifer waters have about same hydrochemical facies (Na^+ and SO_4^{2-}). According to Schoeller diagram, thermal and mineral waters are a different water type from the Delice River and the alluvium aquifer waters.

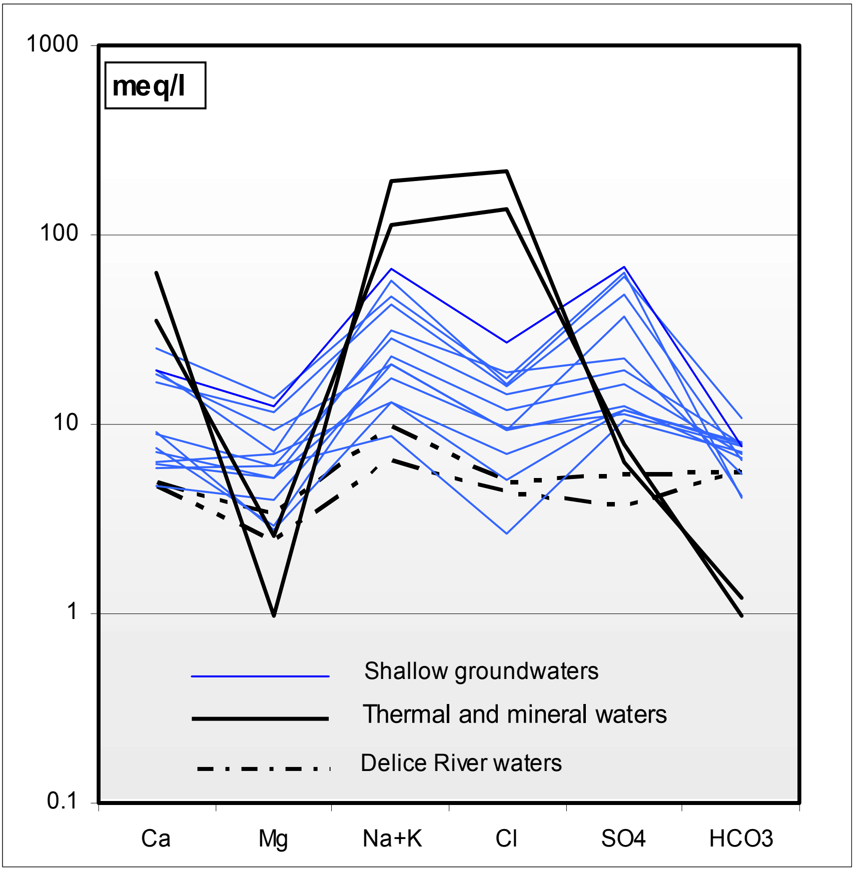


Figure 4: Schoeller diagram of the waters in the study area.

Also, pairs of measured parameters are plotted in x-y diagrams (composition diagrams). The data plot is on straight lines, revealing a positive correlation of Ca, Mg, Na, Cl and SO_4 with TDS (fig. 5). Especially, strong correlations exist between Na-TDS (R^2: 0.88; R: 0.93), SO_4-TDS (R^2: 0.94; R: 0.97). Whereas, ion concentrations of Na and SO_4 are the lowest in the Delice River (D2), the ion concentrations are the highest in the alluvium aquifer well (Well no. HK10) (fig. 6). Therefore the Delice River (D2) and the alluvium aquifer samples (Well no. HK10) may be end members. Mixing rate calculations was made between the Delice River waters and the alluvium aquifer waters (fig. 7). Equation (1) can be written as follows (Mazor [5]).

$$C_s = C_{end\ member1}X + C_{end\ member2}\ (1\text{-}X) \tag{1}$$

C_s : ion concentrations of samples (meq/l)
$C_{end\ member1}$: ion concentration of the first end member (meq/l)
$C_{end\ member2}$: ion concentration of the second end member (meq/l)
X : mixing rate (%).

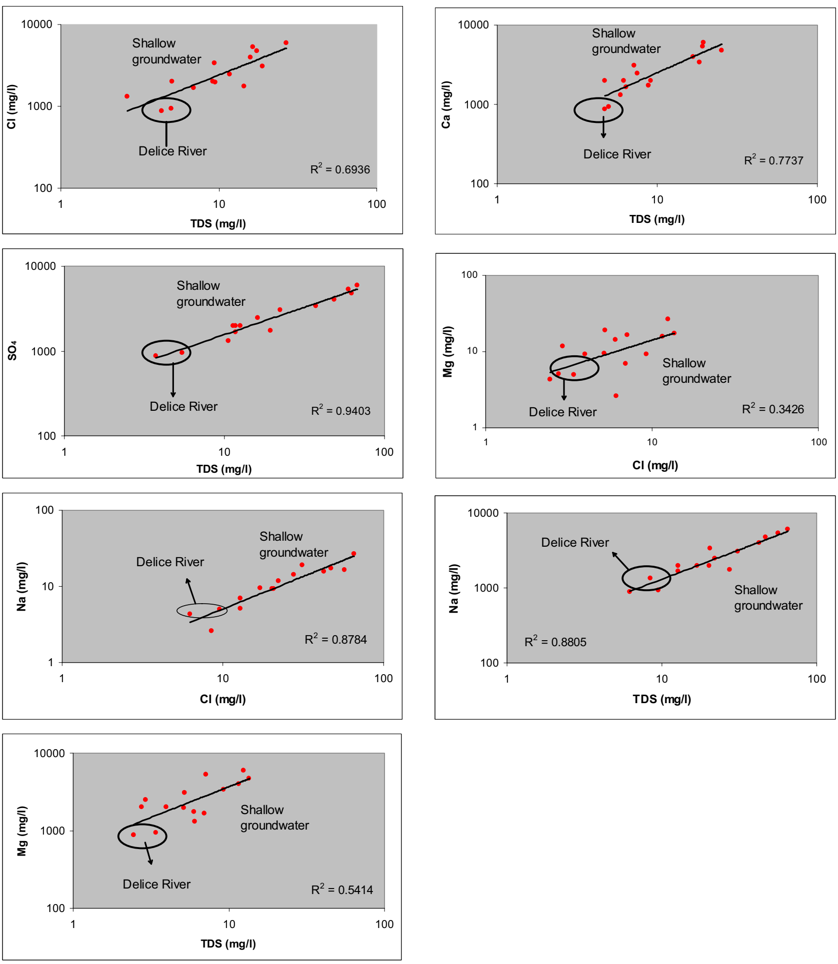


Figure 5: Compositional diagrams of the shallow alluvium aquifer groundwaters and the Delice River samples (Total dissolved solids and the ions are of mg/l).

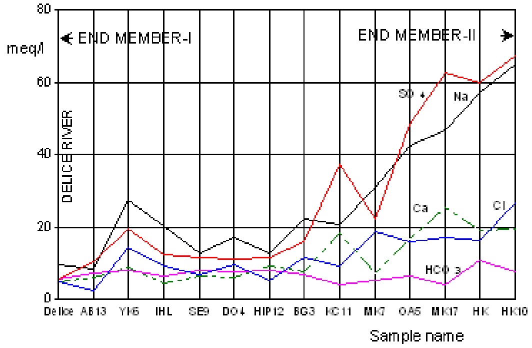
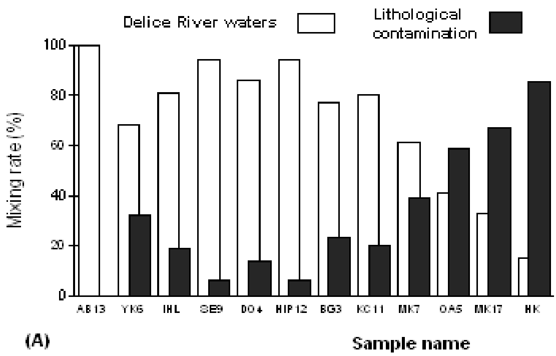


Figure 6: Variation of the ions (anion and cation) between end member-I and end member-II.



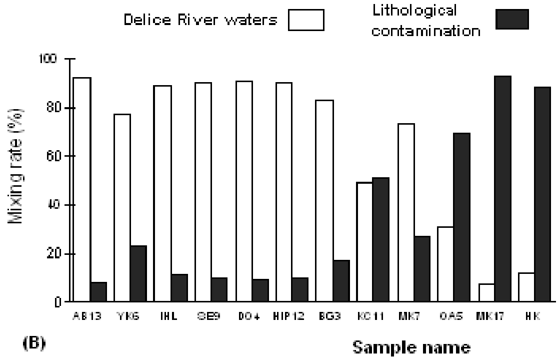


Figure 7: Sodium (A) and sulphate (B) mixing rates into the shallow alluvium aquifer well waters from the Delice River waters.

Since the major ions of Na and SO_4 have been typically increased through the alluvium aquifer, mixing rate calculations are conducted for these ions. According to the mixing calculations, a lot of well waters (AB13, YK6, IHL, SE9, DO4, HIP12 and BG3) have been fairly diluted by the Delice River waters. A few wells have been contaminated by lithology. These are of MK7, KC11, OA5, MK17 and HK wells (fig. 7). According to Mazor (1991) [5], clay and shale rocks often contain salt and gypsum. The rocks have high salinity (900-2,000 mg/l), Cl^- dominant anion, followed by SO_4 and Na major cation. Gypsum has high salinity (2,000-4,000 mg/l), dominant anion is SO_4, dominant cation is Ca, followed by Mg and Na. Salinity (TDS) is between 1,669 and 5,957 mg/l in the study area. As the major ions have been diluted by the Delice River waters, also boron contamination of the alluvium aquifer has been decreased by the same way. A solute in water will move from an area of greater concentration towards an area where it is less concentrated. This process is known as molecular diffusion. The diffusion will occur as long as a concentration gradient exists, even if the fluid is not moving, which can be expressed as Fick's first law (Fetter [6]). The origin of the high salinity and the boron contamination of the alluvium aquifer are Çevirme Formation units which are mainly claystone and gypsiferous marls. The contaminated shallow alluvium aquifer area is about 130 km^2 in the Yerköy plain.

4 Conclusions

Whereas the hydrochemical content of the Delice River water has a conductivity of 1,610 µS/cm and total dissolved solids of 938 mg/l, the hydrochemical content of the shallow aquifer groundwater displays spatial variations in conductivity (between 2,340 and 9,360 µS/cm) and total dissolved solids (between 1,669 and 5,957 mg/l).

The hydrochemical parameters (Na^+, SO_4^{2-}, Cl^-, boron, HCO_3^-), which characterise the different components of the groundwater, allowed the determination of the origin of groundwater contamination. The shallow alluvium aquifer groundwater contamination has been diluted by the Delice River waters. According to mixing calculations, a lot of wells (AB13, YK6, IHL, SE9, DO4, HIP12 and BG3) have been fairly diluted by the Delice River waters. A few wells have been contaminated by lithology. These are of MK7, KC11, OA5, MK17 and HK wells.

The origin of the contamination is Çevirme Formation lithologic units, lying southwest of the study area, underlying the aquifer and consists of claystone and gypsiferous marls.

In order to remediate the waters of the shallow alluvium aquifer, firstly Na, SO_4 and boron ions may be diluted by the Delice River waters. For this, irrigation strategy may be applied in the Yerköy plain with pumping from channels recharged from the Delice River.

References

[1] Çelik, M. & Yıldırım, T., Hydrochemical evaluation of groundwater quality in the Çavuşçayı basin, Sungurlu-Çorum, Turkey, *Environmental Geology*, 50(3), pp. 323-330, 2006.
[2] Gündüz, M., *Hydrogeological investigation of Güven (Uyuz) bath, Yerköy, Yozgat,* (in Turkish), Mineral Research and Exploration Institute Report, No. 9595, Ankara, 1993.
[3] Canik, B., Hydrogeological investigation of Bulamaçlı bath (Çiçekdağı, Kırşehir) (in Turkish), *Mineral Research and Exploration Institute Journal*, 93/94, pp. 118-136, 1982.
[4] Çelik, M., Water quality assessment and the investigation of the relationship between the River Delice and the aquifer systems in the vicinity of Yerköy (Yozgat, Turkey), *Environmental Geology*, 42(6), pp. 690-700, 2002.
[5] Mazor, E., *Applied Chemical and Isotopic Groundwater Hydrology*, Open University Press: Buckingham, pp. 264, 1991.
[6] Fetter, C. W., *Contaminant Hydrogeology*, Macmillan Publishing Company: New York, pp. 458, 1993.

Section 5
Waste water management

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Systems for the sustainable management of agricultural wastewaters

S. Barrington, I. Ali, S. Morin & J. Whalen
Faculty of Agricultural and Environmental Sciences,
Macdonald Campus of McGill University, Canada

Abstract

Agricultural enterprises produce wastewaters in large quantities and from multiple sources. These wastewaters offer relatively low levels of nutrients and conventional land spreading equipment cannot apply these at a sustainable rate of 1000m^3/ha. Two new application technologies were developed to better use the nutrients of these wastewaters in a sustainable fashion, while also using the water applied to the crop and reducing the application costs: a modified surfaced irrigation method and a modified seepage field associated with an organic matter trap and septic tank. The project tested the performance of both systems to obtain the best management practices. The modified surface irrigation system performed with minimal environmental impact when using a plot larger than that required for infiltration and applying the wastewater on dry soils using recommended irrigation rates. The adapted surface irrigation technique reduced the land spreading costs from \$3.50 to \$1.00 Can m^{-3}. The modified seepage field coupled with a septic tank worked well for the disposal of milk house wastewaters when managing the sediments and milk fat. The modified seepage field had limited impact on groundwater quality, but provided crop nutrients and reduced the investment cost of a treatment system for milk house wastewaters \$15 000 to \$6 000 Ca., for a 60 cows dairy herd.
Keywords: agricultural wastewater, sustainable and economical treatment.

1 Introduction

Agricultural enterprises produce large volumes of wastewaters which are generally costly to handle because of their low nutrient content [1]. These wastewaters consist mainly of manure seepage produced from the decomposition

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of manure, from precipitations drained from solid manure piles stored outside, and from wash waters produced when cleaning facilities such as milking equipment and stalls. The operations producing the most wastewaters are dairy herds handling their manures as solids, and located in regions where precipitations exceed evaporation. In the United States and Canada, dairy operations with as many as 200 cows still handle their manures as solids and produce on an annual basis over 2 000 m^3 of wastewaters [2]. While costing over \$7 000 Ca. to spread on land, these wastewaters only offer only \$500 in crop nutrient value. Limited to an application rate of 100 m^3 ha^{-1}, conventional manure spreading equipment is not designed to land spread wastewaters with such a low nutrient content because rates as high as 1 000 m^3 ha^{-1} are required to supply the full crop nutrient requirements [3, 4].

The following are the requirements for the development of more efficient and sustainable systems to land apply such wastewaters: no nutrient accumulation within the recycling system on an annual basis; the valorisation of all components of the wastewater, including the water, and; the affordability of the technique for the continued viability of the farm operation. Nutrient accumulation can be avoided by applying rates equivalent to crop uptake. Cost affordability of the technique is ensured by developing a technique better adapted to land application. The development of the following two techniques was achieved to respect the definition of sustainable wastewater management. The objective of this paper was to describe these techniques and recommend best management practices for minimal environmental impact.

2 Modified surface irrigation system

A surface irrigation technique was developed to more effectively land spread large volumes of wastewater and to meet crop requirements, while still reducing the cost of handling such wastewater (Figure 1). The proposed concept consisted in laying a gated irrigation pipe on the ground, where the soil surface consistently sloped downwards, even at a low rate of 0.1%. After being released by the gated pipe, the wastewater could run down the slope and cover the plot surface while infiltrating the soil. Wastewaters can be fed into this irrigation pipe by means of a flexible non perforated hose and an irrigation pump with a capacity of 3 to 10 m^3 min^{-1}. By collecting manure seepage and milk house wastewaters in a reservoir separate from that of the manure, large clumps of solids are avoided along with the risks of clogging the irrigation system. The nutrient content of the wastewater dictates the area of crop to be irrigated for a complete nutrient uptake, and this area in turn establishes the infiltration capacity and the pumping rate ensuring that the wastewaters will cover the entire plot area. A safety factor of 1.25 should be applied to increase the plot runoff length and prevent wastewater pounding at the foot of the slope.

2.1 Environmental impact evaluation

Once the equipment was found functional, the project consisted in testing its impact on groundwater quality using a control and irrigated silty soil plot each measuring 0.5 ha on the first dairy farm and 0.3 ha on the second farms with respective herds of 44 and 24 cows. All plots were drained using a subsurface system because ground wastewater losses could be measured at the outlet of this system during irrigation sessions. The project consisted in sampling and analyzing the wastewaters found in the storage tank of several farms located in the region South West of Montreal, Canada (table 1). With these results, the amount of wastewater required to supply nutrients to a corn silage crop was calculated (table 2).

Figure 1: The modified surface irrigation system consists of a farm tractor and pump in the background, at the wastewater storage pit. A flexible hose brings the wastewater to the gated irrigation pipe for its release at the soil surface where it flows over the soil surface and spreads by itself.

While respecting the rate associated with the most environmentally limiting nutrient (phosphorous), the test then consisted in applying various rates of wastewater, and measuring the losses at the subsurface drainage outlet (table 3). The water table height was monitored before and after each irrigation session, along with the surface soil moisture content, to be able to recommend best management practices.

Table 1: Characteristics of experimental agricultural wastewaters [2, 3].

Element	Source		
	MH	MS	MH & MS
TS, %	0.25-0.30	0.60-1.55	0.25-1.50
SS, %	0.03-0.06	0.60-1.28	0.22-1.45
DS, %	0.22-0.27	0.03-0.10	0.03-0.25
pH	5.5-7.0	7.0-7.5	6.0-7.5
TKN, mg L^{-1}	40-100	150-1 000	50-1 100
TP, mg L^{-1}	70-130	12-50	15-90
TK, mg L^{-1}	150-300	200-600	200-1000
TC,CFU mL^{-1}	5 000	20 - 2 000	50-3 000
FC, CFU mL^{-1}	2 000	20 - 100	10-2 000
FS, CFU mL^{-1}	3 000	50 – 1 800	5-16 000
n (farm sampled)	2	3	3

Note: CFU – colony forming unit; MH – milk house; MS – manure seepage.

Table 2: Crop nutrient requirements as compared to wastewater nutrient content.

MS& MH	Application to meet crop requirements (m^3 ha^{-1})		
Year*	N	P_2O_5	K_20
2002	2780	1390	125
2003	1040	1360	185
2004	940	860	170
MS			
Year*	N	P_2O_5	K_20
2002	870	1800	285
2003	500	1000	160
2004	170	660	120
Crop requirement*	150	62	120

Note: *corn silage nutrient uptake in kg ha^{-1} for a yield of 30 tons ha^{-1} at 35% dry matter content.
*wastewater application rate based on observed nutrient load for three consecutive years, on two individual dairy farms.

2.2 Results of the environmental impact evaluation

Table 3 summarizes the wastewater losses obtained with the various application rates on the first farm with a herd of 44 cows. Those of the second farm were similar and are therefore not presented.

For both farms, losses of wastewater occurred when applying the wastewater at high rate (exceeding 500 m^3 ha^{-1}), after a rainfall increasing the water table height above that of the subsurface drainage system. To avoid seepage losses, it was therefore recommended to apply the wastewater during dry spells when the water table was below the subsurface drainage system, and to respect irrigation application rates as recommended by Schwab et al. [5].

Table 3: Losses of wastewater by subsurface drainage system.

Irrigation session	Application rate, m^3 ha^{-1}	Rainfall mm*	GW depth, m	Drainage losses, m^3
1st	450	0		
2nd	230	30		
3rd	630	20		1.6
Year 2				
1st	538	6	0h- >1.60 3h->1.60 20hr-1.12	
2nd	552	12	0h->1.60 5h-0.80	
3rd	682	100	0hr-1.37 4hr-0.31 6hr-0.45 72hr-1.1	4.0

*rainfall within 2 days of irrigation session; GW – ground water table; GW depth was observed as of the start of the irrigation session; drainage losses as measured at the subsurface system outlet.

When applied under dry soil conditions, crop yield was increased by 20% and crop protein content was increased while its fibber content was decreased. A time study also conducted during the irrigation sessions indicated that the application procedure required only 2 to 3 hours as compared to 2 to 3 days with conventional equipment. The cost of land spreading the wastewaters was therefore reduced to \$1.00 as compared to \$3.50 Can m^{-3}, excluding the benefits of increasing crop yield and quality.

3 Modified septic system for milk house wastewaters

To develop a method of disposing of milk house wastewaters for dairy farms with at the very most 60 mature cows, a modified seepage system was also developed. For such farms, a conventional septic system is still the least expansive method of disposal for milk house wastewaters, but clogging problems often result and nutrients generally accumulate in the soil surrounding the seepage field. When using a conventional septic system, the heavy milk fat and sediment loads often exceed the septic tank digestion capacity and lead to the washing of organic material and their accumulation within the sewer pipes of the seepage field. The large volume of wastewater applied conventionally over a small surface area generally saturates the soil with wastewaters which in turn losses its permeability and accumulates nutrient often exceeding its adsorption capacity.

To solve these issues with a sustainable solution, a modified seepage system was designed (Figures 2 and 3): a trap was installed before the septic tank to capture sediments and milk fat and manually remove these on a regular basis;

this trap was easier to clean than the septic tank itself, and reduced risks of loading the septic tank above its digestion capacity. The seepage field was designed on the premises that land was not a restraint on dairy farms, as compared to residential lots. Installed in a cropped field of sufficient size, the nutrients accumulating in the soil around the sewer pipes could be removed by the plants.

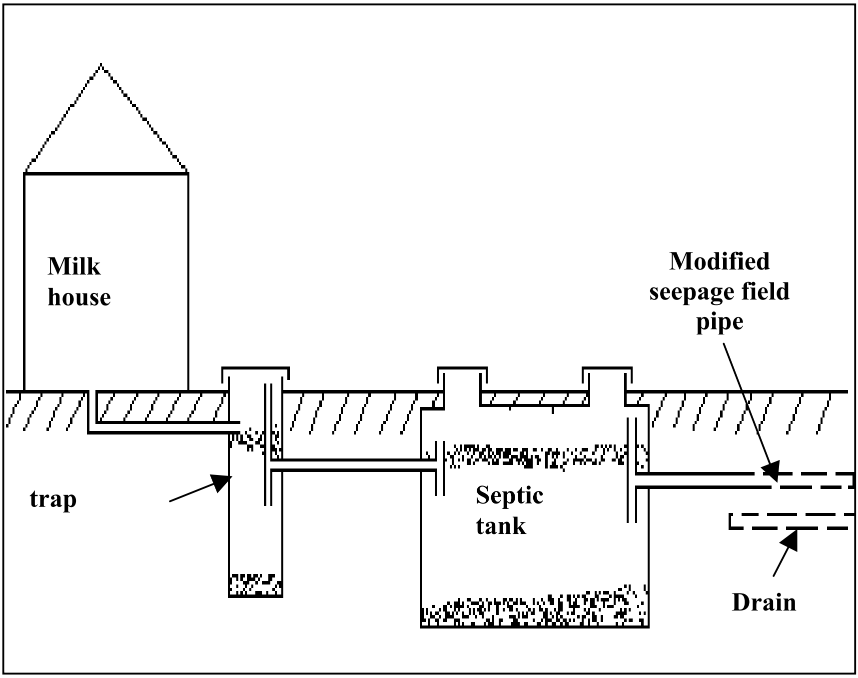

Figure 2: Modified septic system consisting of a grease and sediment trap installed before the septic tank, and a large seepage field of 0.5 ha with a drainage system to insure proper soil filtration.

The wastewaters produced by a herd of 50 dairy cows produced enough nutrients to supply 0.50 ha of cereals (Figure 3). Therefore, the sewer pipes of the seepage field measured 100 m in length and were spaced 15 m apart. The coverage of a large surface of cropped land reduced the risks of soil saturation both by water and nutrients. A filter envelop was installed on the sewer pipes to eliminate the need for a bed of crush stone. The sewer pipes were installed without a slope at a depth of 550 to 650 mm, by allowing one or two 2.4 m sections to drop by 50 to 100 cm. A standard agricultural drain (perforated and corrugated drainage tubing) was installed between each run of sewer pipe, but 150 mm lower, to maintain a water table level below that of the seepage field, and to force nutrients to filter into the ground, especially during the winter, in the absence of a crop. The milk house wastewaters applied on such a large surface do not contribute enough water to promote higher yields.

Because the nutrients supplied through the seepage field corresponded to that of the crop uptake, the system was designed to be sustainable and to prevent the accumulation of nutrients, especially phosphorous. This adaptation is a marked contrast when compared to conventional seepage fields, which are concentrated over a limited land surface and which build up nutrients over time, especially phosphorous. The wastewater nutrient load observed on two dairy farms with a herd of 40 and 50 cows, and over the span of three years is reported in table 1.

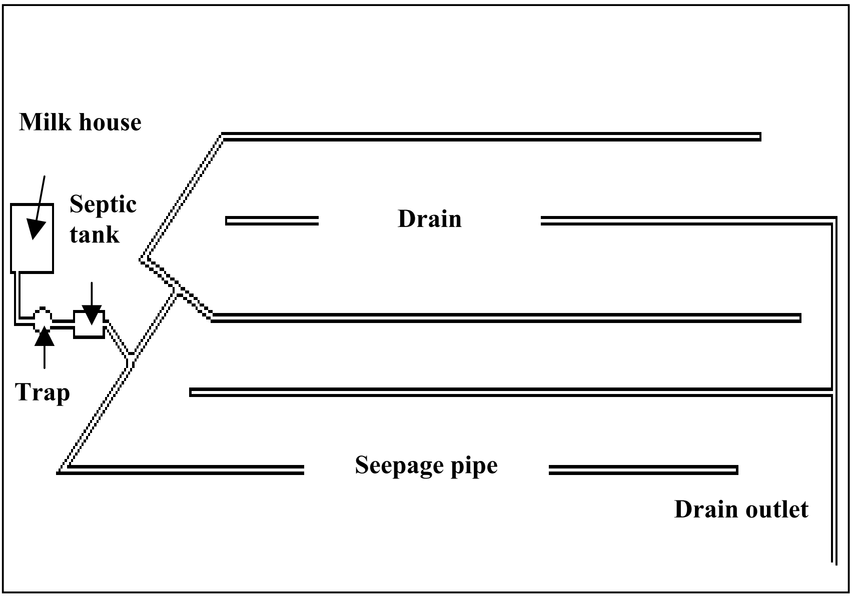

Figure 3: Plan view of one experimental modified septic system. The seepage pipes are spaced at 15 m and the drains are installed 150 mm below and half way between these seepage pipes, to control the level of the groundwater table.

3.1 Environmental impact evaluation

To evaluate the environmental impact of such a system, the amount of wastewater produced was measured by installing a water flow meter in the milk house of the two experimental dairy farms, located in the South West region of Montreal, Canada. After installing the system, the accumulation of milk fat and sediments was monitored in the trap and the quality of the waters drained by the drainage system installed between the seepage field pipes was monitored. The quality of drainage waters from the seepage field was compared to that of a control consisting of a nearby field also drained by a subsurface drainage system.

3.2 Results of the environmental impact evaluation

Both farms were found to produce 13.5 L day^{-1} of milk house wastewater cow^{-1} in the herd. The milk house wastewater characteristics were also found to be able to fertilize 1.0 ha of forage crop (100 dairy $cows)^{-1}$, if applied at a rate of 500 m^3 ha^{-1} yr^{-1} for 50 kg of TP ha^{-1} (table 2).

On one experimental farm as opposed to the other, the trap accumulated as much as 250mm of milk fat over an area of 1.13 m^2, during the first year, because no water softener was used and the pipeline wasted milk was discharged into the septic system. Both farms accumulated sediments. Milk fat and sediment accumulation rates decreased with time over the span of three years of monitoring, likely because an appropriate microbial population was able to establish itself. Milk fat and sediments had to be removed from the trap every season to prevent their flow into the septic tank and sewer pipes of the seepage field.

The quality of the water drained from the seepage field was comparable to that drained from a control subsurface drain (Figure 4 and 5). No significant difference was observed when comparing the drainage waters of the seepage field and that of the control. Therefore the modified seepage field was observed to have a limited impact on the quality of ground waters.

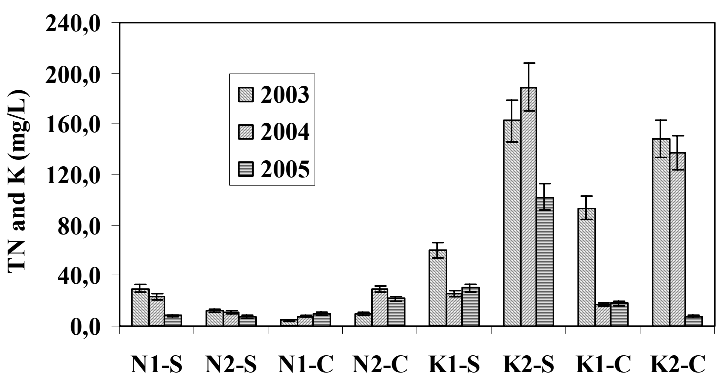

Figure 4: Drainage water quality collected from the modified seepage field and a control drain located in a cropped field receiving no wastewaters. N – Total nitrogen in mg L-1 and K – total potassium in mg L-1. The number besides the element symbol identifies the experimental farm.

At a total cost of $6 000 Can., the modified septic system was found to be quite affordable to build. The 350 m of sewer pipe lines and its subsurface drainage system, cost $3 500 to install while the trap cost $1 000 and the septic tank system cost another $1 500 Can.

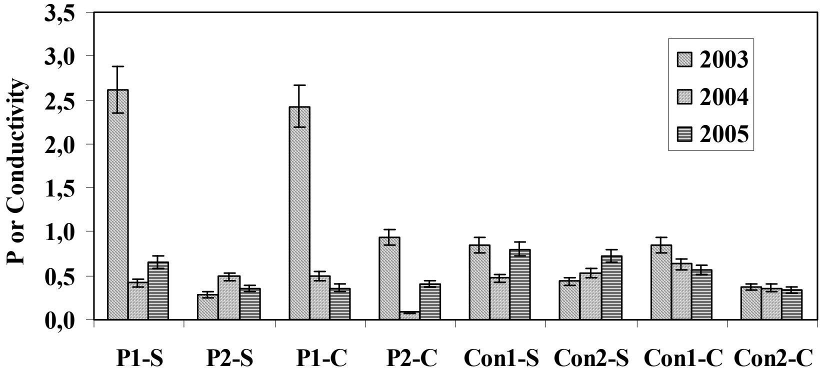

Figure 5: Drainage water quality collected from the modified seepage field and a control drain located in a cropped field receiving no wastewaters. P – Total phosphorous in mg L-1 and Conductivity in mS cm-1. The number besides the element symbol identifies the experimental farm.

Other techniques marketed for the treatment and disposal of milk house wastewaters are available at a cost of at least $15,000 Ca. and do not offer a system eliminating nutrient accumulation, especially phosphorous. The modified septic system is therefore a viable solution for the treatment of milk house wastewaters.

4 Conclusions

Two techniques were developed to dispose off agricultural wastewaters using sustainable concepts. These techniques were sustainable because they led to no nutrient accumulation, to the valorisation of all components of the wastewater including the water and to a lower management cost, thus improving the viability of the farm operation. The techniques consisted in a modified surface irrigation system and septic system, for the respective treatment of both manure and milk house wastewaters and only milk house wastewaters. The best management practices associated with both techniques consisted in applying the wastewaters on a surface large enough for their nutrients to be absorbed by the crop. Furthermore, no groundwater seepage losses were observed when the wastewaters were surface irrigated on a dry soil, at a rate respecting those recommended for irrigation. For the modified septic system, the milk fat and sediment trap had to be cleaned every season, to prevent the overloading of the septic tank and the clogging of the sewer pipes in the seepage field.

References

[1] Loehr, R. 1984. Pollution control for agriculture. Academic Press Inc., New York, USA.
[2] Ribaudo, M.N., Gollehon, N, Aillery, M., Kaplan, J., Joahson, R., Agapoff, J., Christensen, V. 2003. Manure management for water quality: cost to animal feeding operations of applying manure nutrient to land. USDA Economic Research Service, Report AER-824, Washington, DC, USA.
[3] Ali, I., Morin, S. Barrington, S., Whalen, J., Martinez, J. 2006. Surface irrigation of dairy farm effluent. Part I. Nutrient and bacterial load. Journal of Biosystems Engineering, 95 (4), 547-556.
[4] Ali, I., Morin, S., Barrington, S., Whalen, J., Martinez, J. 2006. Surface irrigation of dairy farm effluent. Part II. System design and operation. Journal of Biosystems Engineering. 96 (1), 65-77.
[5] Schwab, G.O., Frevert, R.K., Edminster, T.W., and Barnes, K.K. 1986. Soil and water conservation engineering. John Wiley & Sons, New York, USA.

Fate of anthropogenic micropollutants during wastewater treatment and influence on receiving surface water

A. Bruchet & M. L. Janex-Habibi
CIRSEE, Suez-Environment Le Pecq, France

Abstract

More than 110 volatile and base-neutral compounds or groups of compounds with concentrations exceeding 0.1 µg/L were identified in a wastewater treatment plant, at different stages of treatment. Individual concentrations ranged from 0.1 µg/L to 1 mg/L. The concentrations of many micropollutants were increased after mixing the raw water with recycled waters from the sludge drying process. Six compounds listed among the 33 priority pollutants from the Water Framework Directive, were detected. Due to the high efficiency of the treatment process and to the high dilution ratio at the outlet, this plant showed little influence on the receiving water.
Keywords: base-neutral fraction, glycol ethers, hydrocarbons, nonylphenols, phtalates, priority pollutants, volatile organics, wastewater treatment.

1 Introduction

Various natural and synthetic compounds have been shown to interfere with animal and human endocrine systems among them organochlorine pesticides, plasticizers, detergents, natural and synthetic hormones, organometallics or heavy metals. A relationship is suspected between these compounds and hormonal dependant cancers, in particular breast and testicular cancers. Simultaneously, more and more researchers are also discovering drugs and their metabolites as well as personal care products in drinking water resources. Although the levels of drugs found can be considered as rather small compared to the amounts prescribed to patients, there is concern about their long-term effects on humans as well as on aquatic organisms. One of the questions raised is

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whether drugs such as antibiotics could induce the proliferation of resistant microbes that would in turn damage human health.

Urban wastewaters are one of the primary sources of synthetic organics discharged into environmental waters due to their wide use as household products (detergents, cosmetics and paints), or natural excretion by humans (drugs and metabolites, and synthetic hormones). Most studies usually target a specific group of micropollutants, however wastewaters contain hundreds of micropollutants and, as the number of chemicals being tested for their reproductive or other toxicological effects is growing, many compounds that are not taken into account today might be considered as undesirable in the future. The objective of this study was hence to implement an analytical scheme capable of identifying as many micropollutants as possible in a single waste or natural water sample to create a database for future reference. This paper reports the application of part of this scheme to the analysis of natural and man-made contaminants in a urban wastewater plant sampled at different treatment stages and discusses the fate of relevant compounds in detail. The influence of the plant effluent on the receiving water is also discussed.

2 Analytical methods

The analytical methods implemented were part of a broad screening analytical scheme that was described elsewhere [1] Briefly, this scheme allows the recovery of volatile compounds by purge-and-trap-GC/MS, of a base-neutral and acidic fraction after sequential liquid-liquid extraction with methylene chloride at pH 11 then pH 2. The base-neutral fraction is injected directly in GC/MS then derivatized by silylation to quantitate sterols, while the acidic fraction is injected without derivatization on a polar column and after methylation on a non-polar column to study aliphatic and aromatic carboxylic acids as well as phenols. Solid-phase extraction on a polymeric PLRPS resin followed by diazomethane methylation and GC/MS is used to specifically quantitate Linear alkylbenzene sulfonates (LAS) while chelating agents such as NTA and EDTA are recovered by ion exchange, then derivatized by butylation prior to GC/MS analysis.

3 Description of the wastewater plant

The wastewater plant investigated which was built in 1990 and is located southeast of the Paris area treats the effluents from 250, 000 inhabitants (48, 500 m^3/day). This plant comprises the following treatment steps: screening, mixing of the raw waters with recycled waters from digested sludge dehydration treatment (with belt filters), grease and sand removal, primary settling, low food to mass ratio activated sludge treatment and final clarification , (fig. 1). At the time of sampling the plant raw water showed DOC, COD and BDO_5 values equal to 61 mg/L, 215 mg O_2/L, 80 mg/L O_2/L in the influent, and 16 mg/L, 30 mg O_2/L, <5mg O_2/L in the effluent. The plant was hence removing 75% of the organic matter (estimated by DOC) and at least 95% of biodegradable organic matter (as BOD_5). The treated wastewaters are discharged into the Seine River,

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therefore two samples from this river were collected upstream and downstream to assess any influence of the plant on the river water quality. The plant itself was sampled at five different stages, including the raw water intake, the raw water after mixing with recycled waters from sludge treatment, after sand and grease removal, after primary settling and the treated water after the secondary clarifiers. 24 hour composite samples were collected in glass containers.

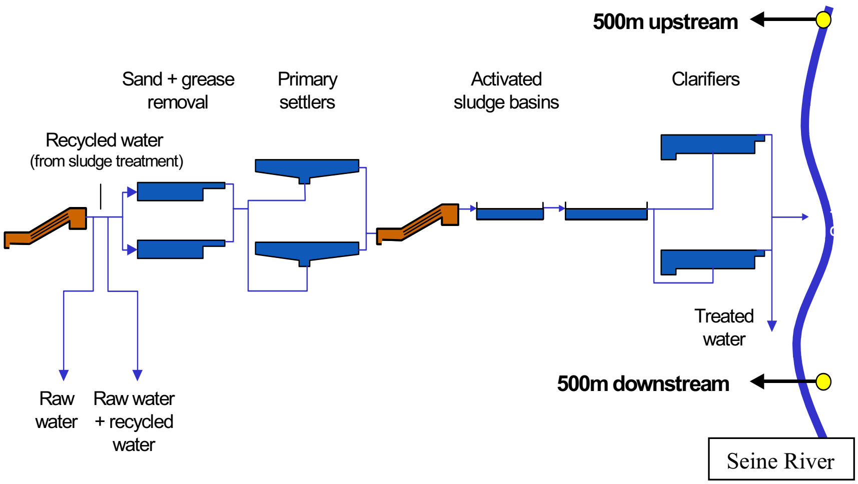

Figure 1: Schematic of the wastewater plant investigated.

4 Results

Because of the rather high dilution in the receiving surface water (Seine river), at least 100 fold during average summer flow conditions, it was not deemed relevant to report compounds present at levels less than 0.1 µg/L. Also, because carrying out a true quantitative work for all compounds detected was considered as impossible, except for specific groups of interest, the concentration of volatile and semi-volatile compounds was estimated relative to internal standards as described by Nguyen *et al* [1]. As the results for select compounds at the present plant were already reported elsewhere [2], the present paper reports in detail for the first time the exhaustive results obtained for volatile compounds and base-neutral compounds.

4.1 Volatile organic compounds

Volatile organic compounds identified in the raw wastewater, in the raw wastewater mixed with recycled waters, in the water after primary settling, and in the treated water after secondary settling, are listed in table 1. More than 50 volatile compounds were detected at levels exceeding 0.1 µg/L. They

Table 1: Volatile compounds at different stages of treatment (µg/L*).

compounds	raw water	raw+recycled water	after primary settling	treated water
Hydrocarbons				
benzene	1.5	10.7	< 0.1	0.5
toluene	3.4	33.3	3.8	0.3
ethylbenzène	0.5	0.7	0.6	0.1
xylenes	6.7	13.7	13.5	0.3
alkylbenzenes ΦC_3	17.7	33.9	111.9	0.1
méthylstyrene	< 0.1	< 0.1	< 0.1	0.3
alkylbenzenes ΦC_4	9.6	19.5	2.5	< 0.1
cymene	< 0.1	4.3	< 0.1	< 0.1
p-ethinyltoluene	< 0.1	11.8	< 0.1	< 0.1
dimethylstyrenes	0.4	0.3	< 0.1	< 0.1
2-methyldecahydro-naphtalene	< 0.1	0.5	0.7	< 0.1
ethylcyclobutane	0.4	< 0.1	< 0.1	< 0.1
alkenes C_9	< 0.1	5.2	6.7	< 0.1
alkane C_8	< 0.1	3.3	< 0.1	< 0.1
alkanes C_9	1.0	3.1	3.0	< 0.1
alkanes C_{10}	33.7	100.9	207.1	< 0.1
alkanes C_{11}	58.1	115.0	123.8	0.6
Halogenated compounds				
1.1-dichloroethane	9.3	36.0	13.9	< 0.1
1.1.1- trichloroethane	1029.0	735.0	559.0	17.2
1.2- dichloroethylene	1.3	13.7	0.9	< 0.1
trichloroethylene	6.8	11.0	4.5	0.5
chloroform	4.7	16.8	3.3	< 0.1
tetrachloroethylene	5.8	6.6	2.9	< 0.1
bromochlorométhane	0.3	2.9	< 0.1	< 0.1
dibromomethane	< 0.1	2.4	< 0.1	< 0.1
dichlorobenzene	< 0.1	16.5	< 0.1	< 0.1
Oxygenated compounds				
1.2-epoxybutane	13.7	27.9	20.5	< 0.1
butanone	< 0.1	23.6	0.2	< 0.1
butyl formate	< 0.1	3.4	0.4	< 0.1
methylisobutylketone	< 0.1	< 0.1	0.5	< 0.1
2-ethyl-4-methyl-1.3-dioxolane	< 0.1	< 0.1	0.4	< 0.1
dimethyl-1.4-dioxolane	< 0.1	< 0.1	1.4	< 0.1

Table 1: (continued).

compounds	raw water	raw+recycled water	after primary settling	treated water
Oxygenated compounds				
dimethyl-1.4-dioxolane	< 0.1	< 0.1	1.4	< 0.1
ketone	0.3	< 0.1	< 0.1	0.1
Alcohols	15.6	23.8	36.7	1.0
alkenol	2.3	< 0.1	< 0.1	< 0.1
Nitrogenous and sulfur compounds				
1-methylthio-1-propene	< 0.1	< 0.1	0.2	< 0.1
dimethyldisulfide	18.6	6.5	0.6	< 0.1
dimethyltrisulfide	2.2	< 0.1	< 0.1	< 0.1
1-butyl-1.2.4-triazole	< 0.1	1.1	0.9	< 0.1
Terpenes				
pinene	2.7	5.3	4.7	< 0.1
limonene	17.2	37.6	22.7	< 0.1
camphene	< 0.1	< 0.1	3.9	< 0.1
menthenol	0.2	< 0.1	0.5	< 0.1

- concentrations estimated relative to 2- Bromo-1-chloropropane internal standard.

primarily comprise aliphatic (with a maximum for C_{10}-C_{11} alkanes) and aromatic hydrocarbons (with a maximum for C_4-alkylbenzenes), chlorinated solvents (with an unusually high level of 1,1,1-trichloroethylene), odorous sulfides, terpenes commonly used in household products, and oxygenated compounds. The unusually high concentration of 1,1,1-trichloroethane reveals an industrial contamination of the raw water as this solvent is commonly used in many industries as a degreasing agent for metal surfaces. The C_1 to C_4-alkylbenzenes indicate a contamination with a light hydrocarbon oil (gasoline) while the heavier alkanes with a maximum in C_{10}-C_{11} indicate contamination with an intermediate oil such as fuel oil or diesel oil. Epoxides such as 1,2-epoxybutane are important intermediates for industrial chemical syntheses. Although present at low concentrations, 2-ethyl-4-methyl-1.3-dioxolane is worth mentioning as this chemical which arises as an impurity during the manufacture of polyester resins, has already induced several taste and odor episodes in drinking waters produced from natural waters contaminated by this compound. Several of the volatile compounds identified are listed among the 33 priority pollutants from the European Framework water directive [3] for which environmental quality standards (EQS) are currently being proposed in Europe [4] or provisional EQS already exist in specific EU countries such as France [5]. These include benzene, trichloroethylene and tetrachloroethylene. At the time of sampling, the plant investigated exhibited an excellent removal efficiency for all volatile compounds and only 1,1,1-trichloroethane was released at a level exceeding 1 μg/L. From a plant management point of view, table 1 clearly indicates that recycling the waters from the sludge dehydration process can drastically increase the concentration of compounds in the water before treatment (see C_{10}-C_{11} alkanes, chloroform, 1,2-epoxybutane, etc) or recycle compounds that are not present in the raw water at the time of sampling (see butanone, butylformate, dichlorobenzene).

4.2 Base-neutral compounds

A wide range of base-neutral compounds were detected after liquid-liquid extraction at basic pH, table 2. Some of the compounds listed in table 1 were also detected in the base-neutral fraction. In this case, the compounds were listed in the fraction where their concentration was the highest, which means when they were measured with the most adapted analytical technique. The base-neutral fraction was dominated by a series of alkanes ranging from C_8 to C_{29}, with maxima for C_{10}, C_{11}, C_{12} (characteristic of diesel oil or fuel oil), and secondary maxima for C_{17}, C_{23}, C_{24}, C_{25} and C_{26}. The hypothesis of a contamination by diesel oil tends to be confirmed by the presence of low levels of paH's and alkyl paH's in this fraction. Some of them, which remain at levels below 0.1 μg/L were listed in table 2 because they were found at higher levels at other stages of treatment during this campaign (one sample was also collected after sand and grease removal and two samples were collected in the Seine river). Fluoranthene and naphthalene are on the list of priority substances from the Water Framework directive, with proposed EQS values of 0.1 and 2.4 μg/L for inland waters. Cycloalkanes are present in minor amounts in oil products. Alkenes from C_{10} to C_{18} may arise as impurities during the manufacture of polyolefin plastics or from the cracking of alkanes in used oil products.

The C_{10} to C_{13}-alkylbenzenes arise as impurities during the manufacture of linear alkylbenzene sulfonate detergents. As a matter of fact LAS were found at levels close to 7 mg/L in the raw wastewater and after mixing with the recycled waters. Therefore these higher molecular weight alkylbenzenes are good indicators of the presence of LAS. In addition to the halogenated compounds detected in the volatile fraction, traces of 2 chlorinated solvents, 1,1,2-trichloroethane and trichloropropene were found in the base-neutral fraction. Glycol ethers such as 2-butoxyethanol or 2-butoxyethoxyethanol are outstanding oxygenated compounds in the base-neutral fraction. The toxicological properties, origins and fate of these water-based solvents were discussed in a previous paper [2]. 2-butoxyethoxy ethanol drastically increases (to 1.27 mg/L) after mixing with the recycled waters from sludge drying. This compound, which shows the highest concentration in the base-neutral fraction, is totally eliminated by the water treatment process. Several weakly estrogenic phtalates (diethyl, and butylbenzylphtalate) were also efficiently eliminated. It is noteworthy that butylisobutylphtalate was less efficiently removed than its linear isomer, butylbenzylphtalate. Like diisooctylphtalate, this compound exceeds 10 μg/L in the treated water. Nonylphenols, also listed among the 33 EU priority pollutants, are considered as the main contaminants responsible for the

Table 2: Base-neutral compounds at different stages of treatment (µg/L).

Compounds	Raw water	Raw+recycled water	After primary settling	Treated water
Hydrocarbons				
Naphtalene	< 0,1	0,2	< 0,1	0,1
Methylnaphtalene	< 0.1	< 0.1	< 0.1	< 0.1
Alkylbenzenes ΦC_{10}	1.2	6.6	3.8	< 0.1
Alkylbenzenes ΦC_{11}	5.5	17.1	20.5	< 0.1
Alkylbenzenes ΦC_{12}	6.2	24.9	24.2	< 0.1
Alkylbenzenes ΦC_{13}	4.5	9.3	2.3	< 0.1
Fluoranthene	< 0.1	< 0.1	< 0.1	< 0.1
Pyrene	0.5	< 0.1	< 0.1	< 0.1
Methylphenanthrene	< 0.1	< 0.1	< 0.1	0.1
Benzophenanthrene	< 0.1	< 0.1	< 0.1	< 0.1
1-Ethyl-4-methylcyclohexane	< 0.1	0.3	0.2	< 0.1
1-Methyl-3-isobutylcyclopentane	< 0.1	1.7	0.3	< 0.1
1.4-Dimethyl-2-isobutylcyclohexane	< 0.1	0.1	< 0.1	< 0.1
Pentylcyclohexane	0.4	0.2	0.3	< 0.1
Alkylcyclohexane	1.9	1.9	< 0.1	< 0.1
Alkenes from C_{10} to C_{18}	4.8	15.6	11.9	0.3
Alkanes C_{8}- C_{29}	213.8	483.5	338.2	2.0
Squalene	66.9	129.2	69.0	< 0.1
Halogenated compounds				
1.1.2-Trichloroethane	< 0.1	< 0.1	0.2	< 0.1
1.1.2-Trichloropropene	< 0.1	< 0.1	< 0.1	< 0.1
Oxygenated compounds				
2-Ethoxybutane	0.8	7.1	0.4	0.2
2-Methanol-2.4-dimethyl-1.3-dioxolane	1.7	4.9	2.3	< 0.1
Cyclohexanone	< 0.1	23.8	< 0.1	< 0.1
2-Butoxyethanol	110.2	140.6	94.0	< 0.1
[2-Ethoxy-1-methoxyethoxy]ethene	< 0.1	0.9	< 0.1	< 0.1
2-Methoxyethyl ether	1.5	< 0.1	2.0	0.5
Dipropylene glycol methyl ether	10.1	24.8	25.4	0.6
Alcohol	51.0	41.3	19.8	1.3
3.7-Dimethyl-1.7-octadiol	21.0	11.3	12.8	< 0.1
Decanol	0.3	9.3	< 0.1	< 0.1
2-Butoxyethoxyethanol	317.0	1270.0	306.0	< 0.1
4-Tertbutylcyclohexanone	0.9	1.4	< 0.1	0.1

Table 2: (continued).

Compounds	Raw water	Raw+recycled water	After primary settling	Treated water
Oxygenated compounds				
Phenyl pentanoate	< 0.1	3.7	< 0.1	< 0.1
1.1-Methyl-2-(2-propenyloxy)ethoxy-2-propanol	3.9	< 0.1	< 0.1	< 0.1
Dimethanonaphtalenol	5.2	< 0.1	< 0.1	< 0.1
2.4-Diterbutylphenol	0.8	0.6	0.5	0.1
4-Methyl-2.6-ditertbutylphenol (BHT)	1.0	0.5	0.9	0.3
Tributyl phosphate	< 0.1	0.2	< 0.1	0.4
Diethyl phtalate	12.5	26.9	20.6	< 0.1
Nonylphenols	< 0.1	5.9	< 0.1	< 0.1
Hexyl 2-hydroxybenzoate (hexyl salicylate)	< 0.1	0.8	< 0.1	< 0.1
Butyl isobutyl phtalate	0.8	82.1	< 0.1	11.1
Isopropyl hexadecanoate	1.8	2.0	< 0.1	< 0.1
2-Hydroxy-4-methoxybenzophenone	1.6	0.9	< 0.1	< 0.1
Hexadecanal	1.9	5.0	< 0.1	< 0.1
Benzylbutyl phtalate	< 0.1	21.3	< 0.1	< 0.1
Butoxyethanol phosphate	15.7	7.8	< 0.1	0.7
Diisooctyl phtalate	46.4	102.2	42.9	16.3
Nitrogenous and sulfur compounds				
Benzothiazole	< 0.1	< 0.1	< 0.1	< 0.1
Dibutylthiophene	< 0.1	< 0.1	< 0.1	0.3
2-(Methylthio)benzothiazole	< 0.1	0.3	< 0.1	0.4
Methylisothiazole	< 0.1	< 0.1	< 0.1	< 0.1
Pyridine	< 0.1	0.7	0.9	< 0.1
N.N-Dimethylformamide	< 0.1	0.3	0.1	< 0.1
3-Isothiocyanato-1-propene	0.4	0.5	0.2	< 0.1
Indole	< 0.1	16.8	< 0.1	< 0.1
Methylindole	2.5	4.6	< 0.1	< 0.1
3-(1-Methyl-2-pyrrolidinyl)pyridine (Nicotine)	< 0.1	< 0.1	< 0.1	< 0.1
2-Indolinone	< 0.1	< 0.1	0.1	< 0.1
N.N-Diethyl-3-methylbenzamide	< 0.1	< 0.1	< 0.1	0.4
Cafeine	34.1	70.1	52.0	0.3
1-Methyl-2-pyrrolidinone	10.5	8.4	8.9	< 0.1
Terpenes				
Menthane	0.5	< 0.1	0.8	< 0.1
Linalol	12.5	8.4	9.7	< 0.1

Table 2: (continued).

Compounds	Raw water	Raw+recycled water	After primary settling	Treated water
Terpenes				
Carveol	0.7	< 0.1	<0.1	< 0.1
Unknown terpenes $C_{10}H_{16}$	10.2	14.0	7.4	< 0.1
Terpineol	2.6	1.6	2.6	< 0.1
Camphor	3.5	4.7	14.9	< 0.1
Menthenol	10.1	10.9	< 0.1	< 0.1
Plinol	< 0.1	0.7	0.3	< 0.1
Bornyl acetate	1.2	1.0	< 0.1	< 0.1
Sterols				
Cholesteryl pelargonate	5.4	8.2	< 0.1	< 0.1
Coprostanone	28.0	66.5	23.8	0.8
Coprostanol	37.0	207.7	381.7	< 0.1

estrogenicity of wastewater effluents, together with the natural estrogenic hormones from humans. As shown in table 2, these compounds, undetected in the raw water, are brought along with the recycled waters, because they tend to be formed during anaerobic stages of sludge treatment. This is due to the anaerobic degradation of heavier alkylphenolpolyethoxylates which are always present at much higher levels than nonylphenols in raw waters. These precursors were not measured as they require an additional analytical method. Because of their use as plastic additives *tert*-butylphenol and its alkyl derivatives such as BHT, as well as organic phosphate additives, are commonly found in wastewaters. T*ert*-butylphenol and tributylphosphate are considered as ecotoxicologically relevant by the French institute (INERIS) in charge of implementing the Water Framework directive in France.

A number of drugs (caffeine, nicotine) and personal care products (2-hydroxy-4-methoxybenzophenone or Sunscreen UV 15) as well as terpenic compounds used as fragrances in household products were also detected. 3-Isothiocyanato-1-propene has been found carcinogenic to male rats [5], while N,N-dimethylformamide is toxic for reproduction. Pyridine, a synthetic intermediate in laboratory and industry is a highly toxic compound which may cause central nervous system depression, gastrointestinal upset and liver and kidney damage.

Among the ca.75 compounds or groups of compounds found in the base-neutral fraction, only the following exceeded 1 μg/L in the treated water: C_8- C_{29} alkanes (2 μg/L), one alcohol (1.3 μg/L), butylisobutylphtalate (11.1 μg/L) and diisooctylphtalate (16.3 μg/L), confirming the excellent efficiency of the plant.

5 Conclusions

More than 110 volatile and base-neutral compounds or groups of compounds (many compounds comprise multiple isomers or homologues) with concentrations exceeding 0.1 μg/L were identified in a wastewater treatment plant, at different stages of treatment. Individual concentrations ranged from 0.1 μg/L to 1 mg/L. The concentrations of many micropollutants were increased after mixing the raw water with recycled waters from the sludge drying process but the secondary treatment (activated sludge followed by settling) was quite efficient at removing them. Six compounds listed among the 33 priority pollutants from the Water Framework directive were detected, including benzene, trichloroethylene, tetrachloroethylene, fluoranthene, naphthalene and nonylphenols. These compounds were detected at a maximum concentration of 0.5 μg/L at the plant outlet.

Many of the other compounds identified are quite undesirable in natural waters whether for the ecological status or for the production of drinking water due to their toxicological or ecotoxicological properties or because of their potential to induce taste and odour problems in finished drinking waters.

Due to the high efficiency of the treatment process, only the following compounds exceeded 1 μg/L in the treated water: C_8-C_{29} alkanes (2 μg/L), one alcohol (1.3 μg/L), butylisobutylphtalate (11.1 μg/L) and diisooctylphtalate (16.3 μg/L) and 1,1,1-trichloroethane (17.2 μg/L). Because the dilution ratio into the receiving river exceeds a 100 fold factor, even during low summer flow conditions, this would entail a maximum input of about 20 ng/L in the worst case, after full mixing, and of less than 1 ng/L for most individual compounds identified. As a matter of fact, the present analytical campaign showed that only 1,1,1-trichloroethane and LAS (not shown here) increased in the river 500 m downstream of the plant, from 0.2 to 0.8 μg/L for trichloroethane and from 35 to 56 μg/L for LAS.

The high dilution ratio of this wastewater effluent into the Seine River also implies that the six priority pollutants identified, which are found at a maximum concentration of 0.5 μg/L in the treated water, will not exceed their Environmental Quality Standard in the river.

References

[1] Nguyen, D.K., Bruchet, A. & Arpino, P., High Resolution Capillary GC-MS Analysis of Low Molecular Weight Organic Compounds in Municipal Wastewater. *Journal of High Resolution Chromatography*, 17(3), pp. 153-159, 1994.

[2] Bruchet, A., Prompsy, C., Filippi, G. & Souali, A., A broad spectrum analytical scheme for the screening of endocrine disruptors (Eds), pharmaceuticals and personal care products in wastewaters and natural waters. *Water Science and Technology*, 46(3), pp. 97-104, 2002.

[3] Directive 2000/60/CE. *Official Journal of the European Communities*, L327-73, 22.12.2000

[4] Proposal for a directive of the European Parliament and of the Council on environmental quality standards in the field of water policy and amending Directive 2000/60/EC, COM (2006) 397 final.
[5] Circulaire DCE 2005/12 relative à la definition du "bon état" et à la constitution des référentiels pour les eaux douces de surface (cours d'eau, plans d'eau), en application de la directive européenne 2000/60/DCE du 23 octobre 2000, ainsi qu'à la démarche à adopter pendant la phase transitoire (2005-2007), 28 juillet 2005.www.environnement.gouv.fr.
[6] National Toxicology Program, Carcinogenesis bioassay of allyl isothiocyanate (CAS NO. 57-06-7) in F344/N rats and B6C3F_1 mice (gavage study), Technical Report Series N°. 234, October 1982.

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Removal of natural organic matter by conventional and enhanced coagulation in Nicaragua

I. García[1] & L. Moreno[2]
[1]*Department of Chemical Engineering and Environment, National University of Engineering, Managua, Nicaragua*
[2]*Department of Chemical Engineering and Technology, Royal Institute of Technology, Stockholm, Sweden*

Abstract

Enhanced coagulation was applied to raw water from a drinking water plant in Nicaragua through bench scale jar test in order to reduce the presence of natural organic matter (NOM) and decrease the trihalomethanes (THMs) formation which has been linked to carcinogenic diseases. Due to the lack of information about the presence of chlorination by-products (CBPs) like trihalomethanes, a study of their formation by varying pH, contact time, temperature and chlorine dose was also performed; following conventional or enhanced coagulation treatment. The results show that enhanced coagulation decreases considerably the formation of THMs because it reduces strongly the presence of organic matter due to the fact that higher alum doses were used in comparison with conventional coagulation utilized at the facility. The removal of dissolved organic carbon (DOC) was improved from 44% at the facility to 67% with enhanced coagulation. Trihalomethanes concentration increases drastically when extreme conditions of the four parameters evaluated were applied exceeding the maximum contaminant levels of USEPA (80 μg/L) but not the Nicaraguan target value (460 μg/L) for both coagulation types.
Keywords: enhanced coagulation, fulvic acid, humic acid, natural organic matter, SUVA, trihalomethanes.

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1 Introduction

After the discover that chlorine can react with natural organic matter present in the aquatic environment to form chlorination by-products (CBPs) like trihalomethanes (THMs) and haloacetic acids (HAAs) (Rook [1]), many researches have been carried out to reduce the presence of NOM previous to the disinfection step. Chlorination by-products are correlated with cancer diseases and also considered of high mutagenicity (Cedergren *et al* [2]; Takanashi *et al* [3]). The levels of mutagenicity in tap waters have been found 19 times greater than in raw water (Takanashi *et al* [3]); and that mutagenicity increases dramatically as a result of chlorination during the water purification process.

The concern about the health risks for the population incentived the fact that many countries have established maximum contaminant levels (MCLs) for THMs and HAAs. In 1998, the United States Environmental Protection Agency (USEPA) published Stage 1 disinfection by-products (DBPs) rule that specified the maximum contaminant levels allowable in drinking water at 0.080, 0.060, 0.010 and 1 mg/L for THMs, $HAA_{5,}$ bromate, and chlorite respectively (USEPA [4]). It also recommended enhanced coagulation (EC) as one of the best ways of removing CBP precursor material; diminishing the formation of CBPs. In 2002, USEPA [5] issued the Stage 2 disinfectants and disinfection by-products rule (DBPR), which retains the same maximum concentrations as in Stage 1. The best available technologies suggested in the Stage 2 are granular activated carbon (GAC) and nanofiltration (NF) with the reservation regarding the formation of dioxin as a by-product of GAC regeneration, while nanofiltration alone cannot effectively remove THMs and HAA_s (USEPA [5]). In Nicaragua, four years after the publication of the DBP regulations (USEPA [4]), maximum contaminant levels of THMs (0.460 mg/L) and HAAs (0.25 mg/L) were included in the guideline for drinking water (CAPRE [6]) but the values are far from those defined by USEPA in 1998 (USEPA [4]).

Since technologies such as nanofiltration, ultrafiltration, granular or powder activated carbon, etc, which are suitable for the removal of NOM, are difficult to be afforded by a developing country like Nicaragua; the use of enhanced coagulation might be a good option to decrease the presence of NOM (Edzwald and Tobiason [7]). Crozes *et al* [8] indicated that enhanced coagulation is an uncostly method of controlling CBPs formation which does not require considerable capital investment.

Enhanced coagulation is defined as an optimized coagulation process for removing NOM which is precursor of CBPs. In general, the term enhanced coagulation is applied at higher coagulant dose and lower pH values. NOM consists of a great diversity of organic compounds including simple sugars, amino acids, proteins and many others; being humic substances the major components of aquatic NOM. The presence of NOM is measured through surrogate parameters as total organic carbon (TOC), dissolved organic carbon (DOC), ultraviolet absorbance at 254 nm (UV_{254}), specific ultraviolet absorbance (SUVA), colour and turbidity.

The aim of this paper was to study whether enhanced coagulation is or not appropriate for the removal of organic matter from raw water of Nicaragua drinking water plants in order to decrease the formation of trihalomethanes. In addition, a study of the impact of parameters such as pH, contact time, temperature and chlorines doses on the formation of THMs was also performed.

2 Methods

2.1 Sampling

Water samples were taken from the drinking water plant of Boaco. The conventional treatment used in the Boaco facility consists of mixing with aluminium sulphate, as coagulant, and calcium hydroxide to adjust the pH, followed by flocculation, settling, rapid sand filtration and disinfection with chlorine (Figure 1).

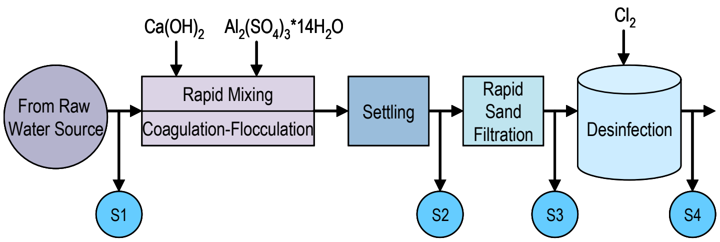

Figure 1: Diagram of the conventional treatment at the drinking water plant and the location of the sample points.

Sample 1 (S1) was used to determine the characteristics of the raw water used in the treatment plant. Water from this point was also used to analyze the NOM surrogate parameters as total organic carbon (TOC), dissolved organic carbon (DOC), ultraviolet absorbance at 254 nm (UV_{254}) and specific ultraviolet absorbance (SUVA); and for enhanced coagulation tests at the laboratory using the procedures described by USEPA [4]. Samples 2 (S2) and 3 (S3) were taken to determine the removal of NOM and some other characteristics of the water after conventional coagulation was applied; and following rapid sand filtration step, respectively. Sample 4 (S4) was employed to establish the presence of THMs in the facility after disinfection with chlorine at their own working conditions. Twelve samples per point were collected over a period of 10 months during the rainy season (May-October) of 2004 and 2005. All the samples were preserved according to the analysis to be carried out by using the respective protocol of each test described in Standard Method (SM) [9] or HACH water analysis handbook [10].

2.2 Analytical procedures

Turbidity (turbidimeter HACH 2100P), colour (HACH Method 8025), water temperature (thermometer), pH (pHmeter HACH 2010), and residual chlorine

(SM 4500-Cl) were measured at the sample site. UV absorbance was determined using a UV/Vis spectrophotometer (Genesis II) at 254 nm in 1 cm quartz cells (SM 5910B) prior sample filtration with pre-washed 0.45 μm filter. Alkalinity was analyzed using SM 2320B procedure. The samples for TOC and DOC were taken using amber glass containers of 120 ml having a screw cap with Teflon septum. They were preserved with ice and were analyzed immediately after its arrival to the laboratory. The samples for DOC were filtered using pre-washed 0.45 μm fibre filter (Whatman). The Persulphate Oxidation Method (HACH 10129) was used for TOC and DOC analyzes. THMs were determined by HACH method 10132. The accuracy of both HACH methods was known by using the standard addition method. Besides, some samples were sent to Sweden to be compared with HACH methodology.

2.3 Jar test enhanced coagulation experiments

Water taken at the sampling point S1 was used to enhance coagulation experiments at the laboratory, following the procedure described by (USEPA [4]). The same coagulant type ($Al_2(SO_4)_3*14H_2O$) as that used at the drinking water plant was applied in these experiments. The common dose of coagulant at the facility is 20-30 mg/L of $Al_2(SO_4)_3*14H_2O$, depending on the turbidity of the raw water source. Enhanced coagulation experiments were performed using two Jar Test apparatuses (Phipps and Bird); initially the rapid mixing was 1 min to 100 rpm after adding the coagulants, following 20 minutes of slow mixing to 20 rpm, finally floc settling time of 30 minutes was used. The coagulant dose used in the enhanced coagulation tests depends on the TOC removal requirement; alum is applied in increment of 10 mg/L until the pH is lowered to the target pH value based on the water alkalinity (USEPA [4]). The ranges of doses used for the different raw waters were 20-70 mg/L.

2.4 THMs and the influences of pH, chlorine dose, contact time and temperature

The influences of pH, chlorine dose, contact time and temperature on the formation of trihalomethane were investigated for raw water treated with conventional and enhanced coagulation. According to the factorial design, 81 set of parameters were considered with three samples in each case. This yields a total de 243 experiments data. The description of the experiments is presented in Table 1.

For water treated with conventional treatment, the samples were taken in the facility in point S3 after conventional coagulation and rapid sand filtration. For water treated by enhanced coagulation at the laboratory, samples were taken after filtration with paper filter No. 1 (Whatman). Both water samples were treated in the laboratory using different chlorine doses, pH values, contact times, and temperatures. The ranges were chosen so that they included the values used at the facility (Table 1).

Reagent grade sodium hypochlorite was used as chlorine source. The pH was adjusted by addition of HCl or NaOH. A constant temperature was maintained using thermostats (LAUDA, M40). 300 ml of water was used for each test.

Table 1: Parameters to be evaluated.

Parameter	Level		
A: Chorine Doses (mg/l)	1	3	5
B: pH	5	7	10
C: Temperature (^oC)	20	25	35
D: Reaction Time (h)	24	50	100

3 Result and discussion

3.1 Natural organic matter (NOM) and other characteristics at the raw water

Natural organic matter is ever present in global aquatic systems, the mass concentration ranging from 0.5 to 100 mg/L of organic carbon (Frimmel [11]). They are classified according to their aqueous solubility, with fulvic acids being more soluble than humic acids. Because there is not a unique parameter that could characterize NOM, surrogate parameters such as TOC, DOC, UV_{254}, SUVA, Turbidity and Colour were measured to the raw water of the facility in order to determine the type of organic matter present.

TOC mean is 16.4±9.0 mg/L, value which is in the range of 1-40 mg/L for surface raw water (Croué *et al* [12]), but it is considered a high value and indicative of erosion problems at the watershed. It is reinforced with the high average value found for colour (97.7±90.4 mg/L Pt-Co). DOC value (7.6±5.1 mg/L) was within the typical range of surface waters (Croué *et al* [12]), DOC concentration is indicator of mass of organic material. The high SUVA (4.1±2.0 L/mg-m) value indicates that predominantly organic matter is hydrophobic DOC, mainly contains aquatic humic material of high molecular weight, and can be removed easily by enhanced coagulation (Edzwald and Tobiason [7]).UV_{254} on the other hand (0.27±0.16 cm^{-1}) was relatively low compared with water samples with similar DOC values (Croué *et al* [12]). UV absorbance at 254 nm specifies the humic or aromatic nature of NOM. A lower UV per mass of DOC would likely result in less CBP formation since UV and CBP are strongly correlated. The average value of turbidity was 35.9±43.5 NTU, alkalinity 101±28.4 mg/L, pH 8.05±0.39 and temperature 23.6±1.2^oC. According with the procedure of enhanced coagulation described in USEPA [4] when the raw water has an alkalinity ranging between 60-120 mg/L and TOC higher than 8 mg/L, the average TOC removal at the facility should be 40%, five of the twelve samples had TOC removal lower of the requirement of USEPA [4] and enhanced coagulation (Step 2) should be applied.

3.2 Natural organic matter after conventional (CC) and enhanced coagulation (EC)

Enhanced coagulation removes significantly more NOM that conventional coagulation (Table 2). It is explained by the fact that high coagulant doses (20-70 mg/L) were applied to remove natural organic matter and as result the reduction of the repulsive forces that keep the NOM particles separated; it is the opposite to conventional coagulation where doses ranging from 20 to 30 mg/L were used and thus lower NOM reduction was obtained. Increasing the coagulant doses resulting in higher amount of NOM is adsorbed due also to the high screening of electrostatic interactions between surface-adsorbed NOM molecules (Vermeer *et al* [13]). Other reasons are: raw water has an average pH of 8.05 causing that the humic compounds are more negatively charged; besides, raw water is rich in humic content (SUVA 4.1 L/mg-m), thus, more humic NOM was amenable to be removed when coagulation was applied, achieving higher removal with enhanced coagulation. Since the alum solubility is extremely low; as soon as the alum dose is augmented, the fraction of aluminium hydroxide precipitated as the hydrolyzed Al^{+3} species increases. Therefore, more humic acid can be adsorbed onto the aluminium hydroxide precipitates, causing higher removal of humic NOM.

The average reduction in DOC concentration using conventional coagulation was 44%, whereas a DOC removal with enhanced coagulation was 67%. A removal of 50% of DOC has been reported by Amy [14]; nevertheless, higher NOM removals of over 80% has been found by Parsons *et al* [15] which is quite high in comparison to the average removal obtained in this study. The degree of DOC removal by coagulation depends on the chemical characteristic of the organic matter, also of the physicochemical properties of the raw water, and type of coagulant employed.

Table 2: NOM parameters after coagulation processes.

Techniques	TOC (mg/l)	DOC (mg/l)	UV (cm^{-1})	SUVA (l/mg-m)	*Colour (mg/l Pt-Co)*
CC	8.6 ± 3.8	4.8 ± 3.5	0.06 ± 0.03	2.8 ± 1.9	*11.0 ± 10.5*
EC	*2.7 ± 1.0*	*1.2 ± 0.5*	*0.04 ± 0.02*	*1.4 ± 0.6*	*1.9 ± 1.3*

On Table 2, it can also notice that organic carbon removal was accompanied with a higher reduction in UV absorbance at 254 nm after coagulation process was applied, when compared to DOC removal, the average percentage of UV reduction was higher for both coagulation type (58% CC, 74% EC) indicating that coagulation was more effective in removing UV absorbing materials than DOC. Also, SUVA decreases considerably after coagulation to values is much lower than 3 L/mg-m, indicating that the DOC remaining is possibly of hydrophilic character, low in molecular weight, low in charge density and therefore, less reactive with chlorine because the aromatic carbon has been preferentially removed. However, because some NOM remained in the treated water after both coagulation processes, THMs can still be formed at the disinfection step.

3.3 Presence of trihalomethanes after conventional and enhanced coagulation

Comparison of the results of both coagulation procedures with the guideline of USEPA [4] and CAPRE [6] showed that the mean values of THMs found are much less that the MCLs values in both guidelines (Table 3). Nevertheless, with conventional coagulation, the MCLs of USEPA [4] is exceeding in some opportunities, which never occurred with enhanced coagulation. It is explained by the fact that in conventional coagulation SUVA average value was 2.8 L/mg-m which indicates that the remaining organic matter still present in the disinfection step is a mixture of hydrophobic and hydrophilic NOM of intermediate molecular weight, therefore, it is still quite reactive with chlorine. On the contrary, average SUVA for enhanced coagulation was 1.4 L/mg-m being the organic matter mainly non-humic, of low hydrophobicity and low molecular weight; as a consequence with lowest aromatic carbon, less reactive with chlorine, forming much lesser trihalomethanes. A mean THM level of 80 μg/L was reported by European survey of surface waters (Villanueva *et al* [16]), being lower than the average THMs concentration (57.5 μg/L) found for this facility.

Table 3: Trihalomethanes.

THMs (μg/l)	Maximum Value	Minimum Value	Mean Value	EPA MCL Guideline	*CAPRE MCL Guideline*
CC	130.0	15.0	57.5 ± 39.8	80	*460*
EC	*35.3*	*2.4*	*12.2 ± 13.4*	*80*	*460*

3.4 Trihalomethanes formation: Impact of the operating parameters

The influence of pH, chlorine dose, contact time and temperature on the formation of trihalomethane was investigated with filtered water treated with conventional or enhanced coagulation (Table 1). The database was generated using coagulant doses of 20-30 mg/L and 20-70 mg/L; DOC concentration of 1.19-2.02 and 0.76-1.23 mg/L; and UV_{254} values of 0.05-0.09 and 0.04-0.09 cm^{-1} for conventional and enhanced coagulation respectively. One example of the results is depicted in Figures 2. The lines in the figures correspond to the average values of three samples taken at different times in 2004. The red solid, blue dashed and green dash-dotted lines show the experimental results for pH 5, 7 and 10 respectively. In the figure, the effect of enhanced coagulation (EC, right side) for reducing the formation of trihalomethanes in comparison with conventional coagulation (CC, left side figure) can be observed.

Nevertheless, higher chlorine doses, pH, temperature and time increases the formation of THMs as can see in Figures 2 and 3. At Figure 2 with a chlorine dose of 1 mg/L, it can notice that at pH 10, time of 100 h; trihalomethanes lightly exceed the MCL of 80 µg/L of USEPA [4] when conventional coagulation is applied. However, with enhanced coagulation a value of 72 µg/L of THMs was found using the same conditions. On the other hand, in Figure 3 with a chlorine dose of 5 mg/L, the concentration of THMs was 158 µg/L with CC at pH 10 and time of 100 h, whereas 112 µg/L was obtained by EC under the same circumstances. Therefore, the formation of THMs was much lower with enhanced coagulation than conventional coagulation. When the chlorine dose was increased from 1 to 5 mg/L, the THMs concentration for both coagulation types exceeded the MCL of 80 µg/L of USEPA [4] in some cases. However, in any case MCL THMs (460 µg/L) used in Nicaragua was overpassed (CAPRE [6]). The higher THMs found in this study were 176 and 145 µg/L for CC and EC respectively to the extreme conditions of pH 10, chlorine dose 5 mg/L, temperature 35^{o}C and 100 h of time (Figure 3).

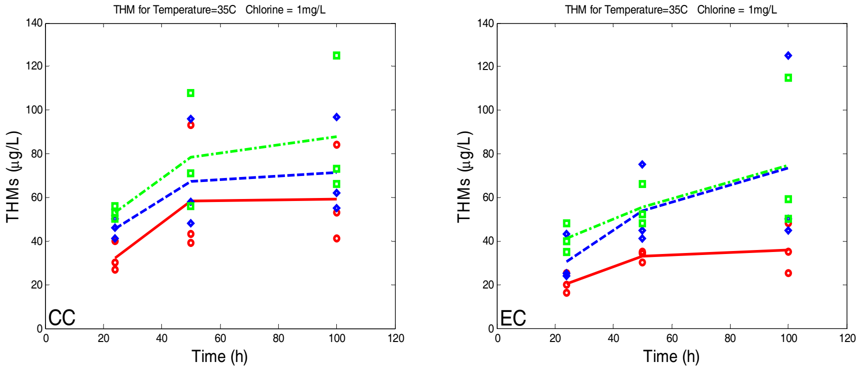

Figure 2: Influence of temperature, pH and time on THMs for chlorine doses 1 mg/L.

The tendency to increase the concentration of THMs when high values of pH, temperature, contact time and chlorine dose are used have been reported by others researchers (Xie [17]; Rodriguez and Serodes [18]; Amy *et al* [19]). It is because the high pH creates an alkaline environment which causes a fast chlorination, the high chlorine dose provokes more halogenations and opening of the aromatic structures of the still available NOM after conventional and enhanced coagulation. As a consequence of fast substitution and oxidation of the organic matter structures by chlorine, more THMs are formed. On the contrary, when pH value of 6 or lower is used, the environment is acid and the reaction is too slow to form end products as THMs, only intermediate products can be formed (Xie [17]). Also, the temperature has influenced in the formation of THMs (Figure 4(a)); high values of temperature increase the reactivity between chlorine and NOM. This reaction leads to a fast rupture of aromatic bonds, allowing halogenations and more formation of THMs as final end products. With low temperature, the formation of THMs is much lower.

THMs are typically hydrolysis products and chlorination end products (Xie [17]). Therefore, increasing the reaction time will lead to an increase in the formation of THMs. As shown in Figure 4(b), THMs increase with time, especially within the first 50 h. A similar behaviour has been observed by Krasner [20]. The reason is that a higher chlorine activity, the THMs formation rate is faster at the beginning; and decreases when the chlorine concentration decreases. THMs are continuously formed after 1-2 days but at a slower rate.

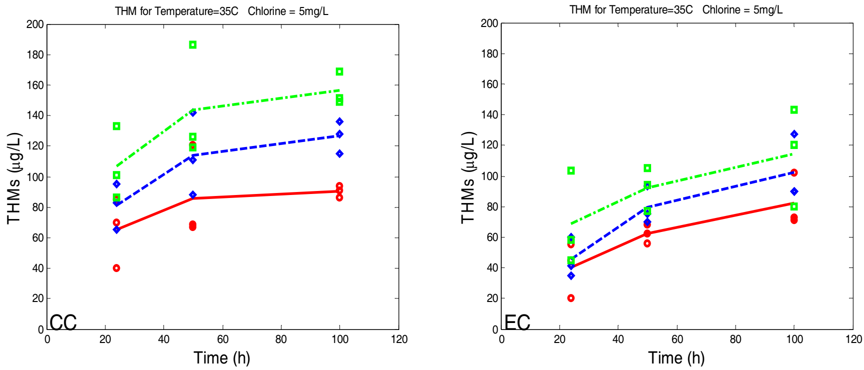

Figure 3: Influence of temperature, pH and time on THMs for chlorine doses 5 mg/L.

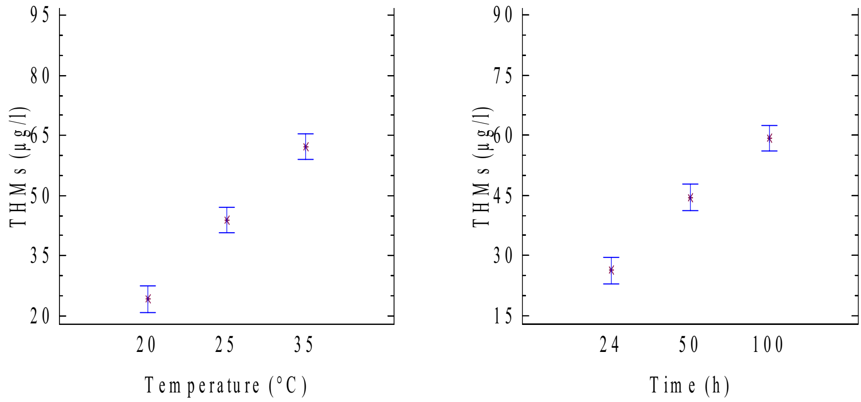

Figure 4: Influence of temperature (a) and time (b) on the formation of THMs after enhanced coagulation treatment.

4 Conclusions

Organic matter was removed easily by enhanced coagulation due mostly to humic types rich in aromatic carbon; therefore, to high coagulant doses, higher amounts of NOM are adsorbed due to the screening of electrostatic interactions between surface-adsorbed NOM molecules.

Comparison between water treated by conventional or enhanced coagulation shows that enhanced coagulation dropped considerably the THM concentration and never exceeds the existing guidelines to the ongoing working conditions of evaluated drinking water plant because the organic matter still present after the filtration step was mainly non-humic, of low molecular weight and low hydrophobicity (SUVA 1.4 L/mg-m) and consequently less reactive with chlorine.

The study of the trihalomethane formation by the variation of pH, time, temperature, and chlorine dose, using water treated by conventional or enhanced coagulation demonstrates that higher values of those parameters increase greatly the formation of THMs because to an alkaline environment result in fast chlorination provoking more halogenations and opening of the remaining NOM where THMs are formed easily as final end product.

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References

[1] Rook, J.J., Formation of haloforms during chlorination of natural waters. *J. Water Treatment Examination.* **23**, pp. 234-239, 1974.

[2] Cedergren, M.I., Selbing, A.J., Lofman, O. & Källen, A.J., Chlorination by-products and nitrate in drinking water and risk for congenital cardiac defects. *J. Environmental Research Section,* **A(89)**, pp. 124-130, 2002.

[3] Takanashi, H., Urano, K., Hirata, M., Hano, T. & Ohgaki, S., Method for measuring mutagen formation potential (MFP) on chlorination as a new water quality index. *J. Water Research,* **35(7)**, pp. 1627-1634, 2001.

[4] USEPA, Enhanced coagulation and enhanced precipitative softening guidance manual. Environmental Protection Agency. United States. Office of Water. *EPA 815-R-99-012*. 1999.

[5] USEPA, Long term 2 enhanced surface water treatment preamble and rule language. Environmental Protection Agency. United States. Office of Water. Draft.

[6] CAPRE Guidelines., Regional committee for drinking water institution and sanitation for Central America, Panama and Dominican Republic, pp. 10-3, 2000.

[7] Edzwald, J. & Tobiason, J., Enhanced coagulation: US requirements and a broader view. *J. Water Science and Technology,* **40(9)**, pp. 63-70, 1999.

[8] Crozes, G., White, P. & Marshall, M., Enhanced coagulation: Its effect on NOM removal and chemical costs. *J. American Water Work Association,* **87**(1), pp. 78-89, 1995.

[9] Standard methods for the examination of water and wastewater, American Public Health Association/American Water WorksAssociation/Water Environment Federation. Washington DC, USA. 20th Edition, 1998.

[10] HACH water analysis handbook. www.hach.com

www.witpress.com, ISSN 1743-3541 (on-line)

[11] Frimmel, F.H., Characterization of natural organic matter as major constituents in aquatic systems. *J. Contaminant Hydrology,* **35**, pp. 201-216, 1998.
[12] Croué, J.P., Debroux, J.F., Amy, G., Aiken, G.R., & Leenheer, J.A., Natural organic matter: structural characteristic and reactive properties. In formation and control of disinfection by-products in drinking water. *AWWA Publication,* pp. 56-92, 2001.
[13] Vermeer, A.W.P., Leermakers, F.A. & Koopal, L.K. Adsorption of weak polyelectrolytes on surfaces with a variable charge. Self-consistent-field calculations. *J. Langmuir,* **13**(6), pp. 4413-4421, 1997.
[14] Amy, G.L., Using NOM characterization for the evaluation of treatment. In: Proceeding workshop on NOM in Drinking Water. Chamonix, France. September 19-22, 1993.
[15] Parsons, S.A., Jarvis, P. & Jefferson, B., Floc structural characteristic using conventional coagulation for a high DOC, low alkalinity surface water source. *J. Water Research*. **40**, pp. 2727-2737, 2006.
[16] Villanueva, C.M., Kogevinas, M., & Grimalt, J.O., Haloacteic acids and trihalometahnes in finished drinking waters from heterogeneous sources. *J. Water Research*. **37 (2)**, pp. 953-958, 2003.
[17] Xie, Y.F., Disinfection by-products in drinking water: formation, analysis and control. *Lewish Publishers.* USA, 2004.
[18] Rodriguez, M.J & Serodes, J.B., Spatial and temporal evolution of trihalomethanes in three water distribution system. *J. Water Research,* **35(6),** pp. 1572-1586, 2001.
[19] Amy, G., Siddiqui, M., Ozekin, K., Zhu, H.W., & Wang, Ch., Empirically based models for predicting chlorination and ozonation by-products; trihalomethanes, haloacetic acids, chloral hydrate, and bromate. *USEPA Reports CX 819579*, 1998.
[20] Krasner, S.W., Chemistry of disinfection by-products formation on formation and control of disinfection by-products in drinking water. Chapter 2. AWWA., 1999.

Titanium-based photocatalysis as the pretreatment for ultrafiltration of secondary municipal effluent with low concentration of organic matters

H. Liu & L. Song
Department of Civil Engineering, Centre for Water Research, National University of Singapore, Singapore

Abstract

Ultrafiltration (UF) has been regarded as a cost-effective option compared to NF and RO for the municipal effluent reclamation process. But the fouling caused by organic matter from the biologically treated sewage effluent is still a serious problem in the process. The photocatalysis was employed as the pretreatment measure for the ultrafiltration of secondary effluents. The fouling potential k was significantly lowered by this pretreatment measure. The effects of catalyst type (anatase, rutile, and Degussa P-25), catalyst dosage, pH value, and UV light intensity on fouling potential variation and dissolved organic carbon (DOC) removal were investigated. The results showed that the presence of UV and TiO_2 improved DOC removal ratio. The effect of photocatalyst type on fouling potential abatement was compared. The pH value and catalyst concentration were optimized to get the highest fouling potential reduction and DOC removal efficiency.
Keywords: secondary effluents, photocatalysis, titanium dioxide, ultrafiltration, membrane fouling, fouling potential, pre-treatment, wastewater reclamation.

1 Introduction

The low pressure-driven membrane processes such as microfiltration and ultrafiltration are increasingly applied in drinking water, industrial water production and wastewater treatment these years. The membrane process has the advantage of compactness, good retention and separation effects for suspended

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and colloidal organic and inorganic substances, pathogenic bacteria, and microorganism, compared with conventional separation processes [1]. However, the fouling caused by the organic and inorganic substances is still a disadvantage for the development of membrane technology. In the area of wastewater reclamation, ultrafiltration (UF) has been regarded as a cost-effective option compared to NF and RO for the municipal effluent reclamation process [2]. But the fouling caused by organic matter from the biologically treated sewage effluent is still a serious problem in the process [3].

Photocatalysis has recently become a common word and various products using photocatalytic functions have been commercialized [4]. Among many candidates for photocatalysis, TiO_2 based photocatalysis is more frequently used for industrial application at present and also probably in the future due to its efficient photoactivity, the highest stability, the lowest cost and, more importantly, its safety. Upon the absorption of sufficiently energetic light (i<387 nm) by TiO_2, an electron is promoted to the higher energy conduction band (CB), leaving a positively charged hole in the valence band (VB). The electron and hole can migrate to the surface of the semiconductor. The electron–hole pairs can either recombine or participate in chemical reactions with surface adsorbed species. Oxidation of water (or hydroxide ions) by the valence-band holes (h^+_{VB}) can produce the hydroxyl radical (HO•). The conduction-band electrons (e^-_{CB}) can react with molecular oxygen to form the superoxide radical-anion ($O_2•^-$) or hydroperoxyl radicals ($HO_2•$). In addition, h^+_{VB} and e^-_{CB} can react directly with adsorbed pollutants. The radicals thus formed, dioxygen and water can participate in further reactions, resulting ultimately in mineralization of the organic pollutants. Many organic contaminants can be almost completely mineralized by UV/TiO_2. The bio-degradation of these pollutants is often very slow and conventional treatments are mostly ineffective and not environmentally compatible, while photocatalysis offers faster, controllable and environmentally benign process with respect to conventional treatment technologies [4].

The objective of this study is to investigate the possibility of photocatalysis as the pretreatment for hollow fiber ultrafiltration membrane organic fouling reduction in the process of reclamation of biologically treated secondary effluent. Accordingly the factors influence the fouling reduction effect will be addressed. Our hypothesis that the photocatalysis can influence the membrane fouling formation negatively or positively is based on the two understanding of the process. First, the photocatalysis treatment can lower the organic load to membrane surface due to the mineralization effect (converted to CO_2 and N_2). On the other hand, photocatalysis process can effectively degrade the long chain organic components or macromolecules of to smaller ones, which can decrease the aromaticity of the feed water and accordingly the adsorption effect of degraded organic matters to the membrane surface will be decreased, which will retard the formation of fouling. Relatively, few studies using photocatalysis as pretreatment have been conducted for the enhancement of permeation flux and abatement of fouling in the ultrafiltration membrane process.

As a method for membrane fouling reduction, the biologically treated secondary effluent was firstly pretreated by photocatalytical reaction before the ultrafiltration process. The parameters, such as photocatalyst type, UV irradiation, reaction time, pH values, etc were changed to investigate their effects on the fouling potential of the fabricated hollow fiber ultrafiltration membrane.

2 Materials and methods

2.1 Hollow fiber ultrafiltration set-up

In this study, the cross-flow hollow fiber ultrafiltration membrane (Nitto Denko Corp.) was used to study the effect of photocatalystical pretreatment on the membrane performance. The schematic diagram of cross-flow ultrafiltration experimental set-up is shown in Figure 1. The outside-in cross-flow membrane module was fabricated with two (OD3/8 inches) SS316 Tees and transparent Swagelok Perfluoroalkoxy(PFA) or Quartz tube (OD 3/8 inches). The hollow fiber membrane lumens were sealed with epoxy resin (chemically stable tested under the experimental environment) into the two ends of the stainless steel tees.

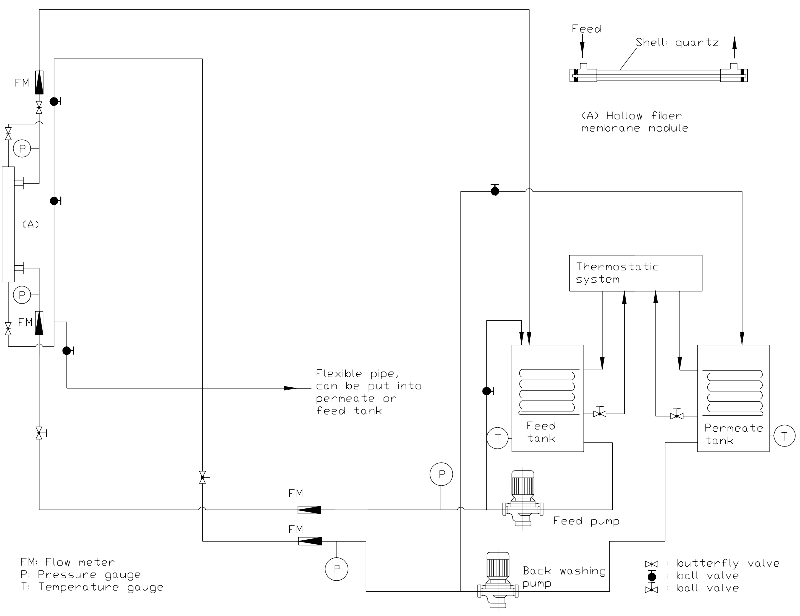

Figure 1: Schematic diagram of the experimental set-up.

The schematic diagram of cross-flow hollow fiber ultrafiltration membrane module is shown in Figure 2.

The wastewater with and without pretreatment was pumped into the hollow fiber membrane module (the effective hollow fiber length 20 cm, effective membrane surface area $0.0035 m^2$). The operating pressure and cross-flow velocity were controlled at 123 kPa and 0.5 m/s by means of by-pass and regulating valves. Reynolds number and shear stress at the wall were 1682 and

1.33 Pa, respectively. The permeate flux was recycled into feed tank. The membrane used in this study was external pressure type hollow fiber UF membrane NTU-3306 (Nitto Denko Corp., Japan) with an average pore size of 6,000 MWCO. Capillary I.D./O.D 1.3/1.9 mm Material: Polysulfone The hollow fiber module was back washed with deionized water and then cleaned with 0.01N NaOH and 0.01N HCl solution and deionized water after each test. The permeate flux can be recovered to 95-98% of that of the virginal membrane. After 10 times' test with one hollow fiber membrane module, a new membrane module will be employed.

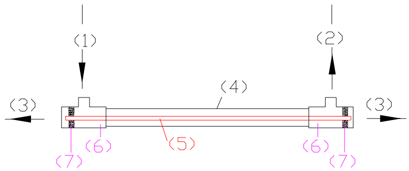

Figure 2: Hollow fiber ultrafiltration membrane module (1) feed water (2) retentate (3) permeate (4) module shell (material, PFA or quartz) (5) hollow fiber membrane (6) stainless steel tee, 3/8 inches (7) epoxy.

2.2 Apparatus

2.2.1 Total organic carbon

TOC was measured by 1010 Total Organic Carbon Analyzer (O. I. Analytical, College Station, TX, USA). All samples were filtered through 0.45 µm membrane (Pall corp.) prior to the TOC measurement. Thus, the TOC values obtained are, in fact, dissolved organic carbon (DOC) values.

2.2.2 Molecular weight (MW) distribution

The wastewater effluent after each pretreatment was subjected to molecular weight distribution measurement. High performance size exclusion chromatography (HPSEC, Shimadzu Class VP series Shimadzu Corp., Japan) was used to determine the MW distribution of wastewater. Standards of MW of various polystyrene sulfonates (PSS: 4,300, 6,800, and 13,000 Da) and Acetone (58 Da) were used to calibrate the equipment. Two columns (Waters Ultrahydrogel 250 and 120) connected in series were used to cover a molecular weight range of 200-80,000 Da. The column temperature was maintained at room temperature and the mobile phase (phosphate buffer 0.0026 M KH_2PO_4+0.0023M K_2HPO_4+ 0.090 M NaCl, pH 6.8, producing an ionic strength of 0.1 M) flow rate was maintained at 0.7 mL/min.

2.3 Irradiation experiments

2.3.1 Reagents

The photocatalyst powder used in this work was commercial titanium dioxide TiO_2, Degussa P25 (about 70% anatase, with a BET surface area of

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approximately 50 m^2/g^{-1}, Degussa AG, Germany), TiO_2 anatase nanopowder (99.7%, spec. surface 200-220 m^2/g, Aldrich Corp.) and TiO_2 rutile nanopowder (99.5%, spec. surface 130-190 m^2/g, Aldrich Corp.). TiO_2 suspensions in wastewater at concentrations of 50, 100 and 500 mg/l were prepared for the photocatalystic reaction.

2.3.2 Photocatalytic activity

Photocatalytic degradation tests were carried out in a thermostatic (33°C), and magnetically agitated reactor. The UV lamp was put directly 10-12 cm above the solution surface in the reactor. Three UV lamps were employed in the experiment, 365nm low pressure mercury lamp, (FL 15BLB 15W×2, Sanyo Denki, Japan), germicidal 254nm low pressure mercury lamp (Sunlite G20T10, 20W×2, USA), customer-made medium high pressure mercury lamp 400W (XHDSHP-400, China) emitting visible and near-UV light. The lamps were turned on for at least 30 min prior to irradiation of wastewater solution samples to ensure uniform lamp output. The UV intensity was measured with a digital spectrophotometer (Model E2, B. Hagner AB, Solna, Sweden). At regular intervals of time, water sample of 30 ml were collected, filtered through a disc filter (pore size 0.45 mm, Pall Corp.) and analyzed with TOC and HPSEC.

3 Results and discussion

3.1 Wastewater photooxidation kinetics

In the presence of TiO_2 and UV light, the organic matter was degraded gradually. At the low TOC level, less than 10 mg/l, TiO_2-photocatlyzed wastewater oxidation process generally exhibits pseudo first-order kinetics with respect to the dissolved TOC (Figure3); comparable kinetics were observed at relatively high TOC level, 18.9 mg/l (data not shown). This behaviour is consistent with previous observations by Chen et al [5].

The apparent first-order constant *K* of the degradation process (generally in min^{-1}) will be obtained from kinetic analysis. The initial concentration of the reactant (c_0), the apparent first-order constant *K* (generally in min^{-1}) could be obtained from experimental data plotted according to:

$$\ln\left(\frac{c_0}{c}\right) = Kt ,$$

with c is the concentration (in mg/l) of the reactant at the illumination time (t) in min. Once plotted according to above equation, data presented a linear correlation with $R^2>90$. The TOC degradation rate *K* is deeply affected by the catalyst concentration, and 0.5g/l P25 TiO_2 is the optimal concentration for TOC degradation, compared with other concentrations under the experiment condition. In addition, the degradation efficiency of three catalysts, P25, anatase, rutile under the 365nm medium high pressure UV lamp was also compared (Figure 4). At the same concentration, 0.5 g/l, nanopowder anatase exhibits the highest TOC removal efficiency, while that of the rutile phase TiO_2

nanopowder is the lowest. The efficiency of P25 is just between them, and this also coincides with catalyst's composition, about 70% is anatase phase TiO_2. The experiment shows that the anatase phase TiO_2 is the major component which is responsible for the effective TOC degradation. This coincides with some other researcher's works. Antase TiO_2 is the most catalytically active form for the photodegradation of aqueous organic pollutants [6].

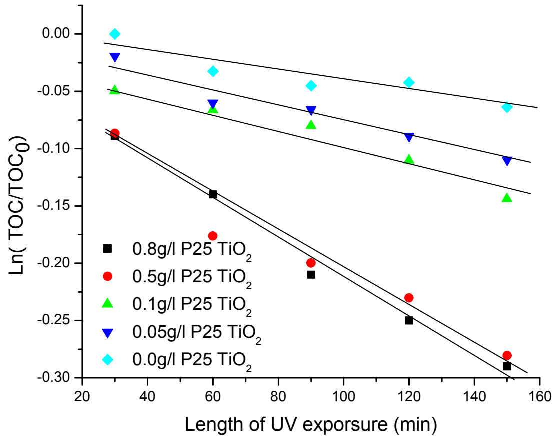

Figure 3: Photocatalysis reaction simulation with the pseudo first-order kinetics.

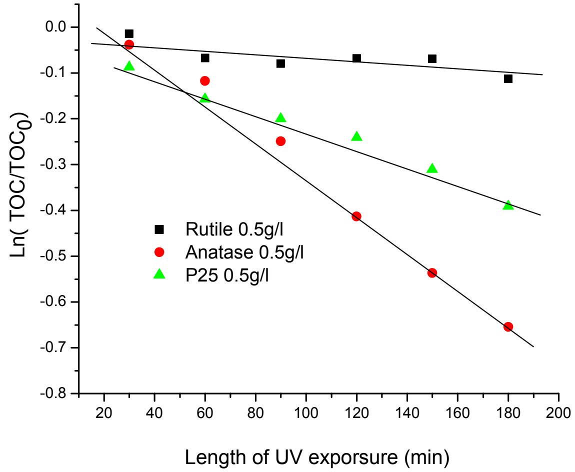

Figure 4: The effect of catalyst type on TOC degradation rate.

3.2 Fouling potential of treated water on ultrafiltration membrane

The photocatalytically treated wastewater was further put into the hollow fiber module to test the fouling potential. Fouling potential is crucial parameter to exam the fouling tendency of the feed water. The fouling potential of feed water is defined as the increment of membrane resistance per unit volume of permeates collected for a unit area of membrane surface [7].

$$k = \frac{R_t - R_0}{V_t}$$

where k is the fouling potential, R_t is the resistance of the membrane at a given time t, R_0 is the clean membrane resistance, V_t is the total volume of permeate per unit membrane area generated at a given time t from the beginning of the filtration process. The detailed deduction and calculation methods can refer to the works of Song et al [7].

Table 1: The fouling potential of the photocatalytically treated secondary effluent.

TiO_2 type and concentration (mg/l)	UV type	pH	Reaction time(min)	Fouling potential (Pa s/m^2)
0	N. A.	7.81	0	2.130E+10
P25, 500	UV365 MHP	7.63	120	6.824E+09
P25, 500	UV365 MHP	7.56	150	3.039E+09
P25, 500	UV365 LP	7.8	150	9.859E+09
P25, 500	UV254 LP	7.6	120	8.846E+09
Anatase,500	UV365 MHP	7.8	150	1.426E+9
Rutile, 500	UV365 MHP	7.6	150	1.532E+10
P25, 500	UV254 LP	4.35	120	4.045E+09
P25, 500	UV254 LP	3.31	120	6.585E+09

MHP: medium high pressure lamp, LP: low pressure lamp.

The fouling potential tests showed that the photocatalysis pretreatment greatly lowered the fouling potential of the feed water, and reduced even to about one magnitude in some cases. For instance, the fouling potential of the photocatalytically degradated wastewater reduced about 90% at the condition of 0.5 g/l anatase TiO_2, 150min reaction time. The UV intensity, catalyst type, the pH value and UV exposure time will greatly influence the fouling potential of the photooxidized wastewater (Table 1). Generally speaking, the longer the reaction time, the greater the fouling potential reduced. At the same pH level, reaction time and with the same catalyst, the wastewater under medium high pressure UV lamp irradiation demonstrated the highest fouling porential reduction. This is due to the higher UV intensity of medium high pressure mercury lamp (23.1 mW/cm^2) than the other two low pressure lamps, germicidal 254 nm and black light blue 365nm (intensity, 2.3 mW/cm^2 and 0.5 mW/cm^2 respectively). In addition, the lower pH value showed higher fouling potential reduction than high pH value, and this is also coincident with the observation of photooxidation kinetics. As one of the major components responsible for the oxidization for the organic matters, the concentration of hydroxyl radical [HO•] is critical for the degradation rate of the reaction. According to the reaction equations, the concentration of [HO•] increases with the concentration of [H^+] in the water. At low pH value, about 3-4, and hydrogen concentration is high. Accordingly, the photodegradation rate is greater than that at relatively high pH 7.8, so the lower pH value showed higher fouling potential reduction.

3.3 The change in the properties of the photooxidized wastewater

The photooxidation process will inevitably lead to the changes in the physical and chemical properties, such as the molecular weight and aromaticity of the organic matters [9]. The HPLC-SEC test gave us qualitative information about the changes in molecular weight distribution by the reaction. Long chain organic compounds were degraded to relatively small molecules. And some organic substrates were converted to carbon dioxide, nitrogen and water. The average molecular weight of reactants declines as the function of reaction time.

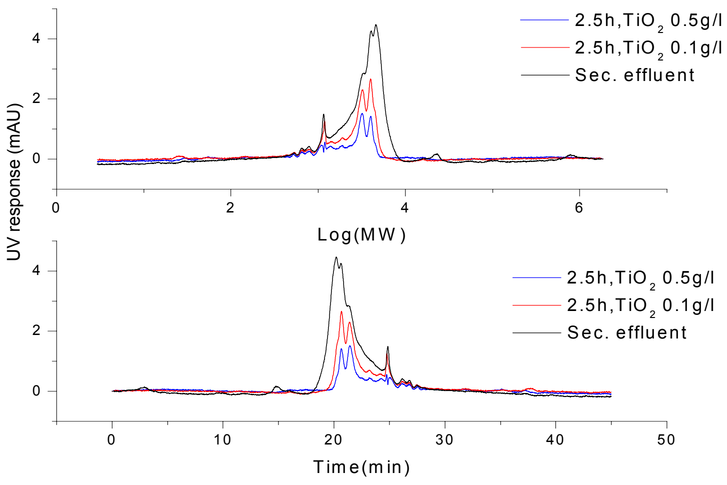

Figure 5: MW distributions of secondary effluent and UV/TiO_2 treated secondary effluent measured by the HPLC-SEC with UV detection.

Generally speaking, long chain molecules or macromolecules have more negative effect for the fouling control than micromolecules [8]. The change in the molecular size contributes to the decrease of fouling potential to some extent. Compared with the photooxidized effluent, the raw effluent has relatively larger molecular size, as shown in Figure 5. There are some bigger molecules in the raw effluent, about 20k Da, corresponding to the peak at about 15min, while that peak is completely disappeared in the treated effluent. It means that the bigger molecules were cleaved into smaller ones (the sharp peak at about 22min), or mineralized. In addition, the smaller molecules in the raw water, as the sharp peak shown at 25min, also disappeared, which is the evidence of the mineralization of small organic molecules. The test also showed that the treated effluent narrows the molecular weight distribution (from about 1k Da to 6k Da, at 0.5g/l P25 TiO_2, 150min reaction) compared with the raw effluent (from about 300 Da to 20k Da). The ratio of weight-average molecular weight (Mw) to number-average molecular weight (Mn), Mw/Mn, changes from 2.35 to 1.21, which implies that the dissolved organic components in the treated secondary effluent become simpler. Another distinctive feature is that the amplitudes of the peaks of treated water are greatly reduced, which means that the concentration of the organic matters are abated after the treatment. Additionally, the concentration of the catalyst will affect the oxidation process. Lower catalyst concentration will produce relatively wider molecular weight distribution with the same UV exposure length (Figure 5). Almost all the phenomena in fouling potential test are reflected on the photooxidation kinetics. The oxidation progress is closely connected with the molecular properties, such as the morphology and aromaticity of organic components, and correspondingly affects the fouling potential of the oxidized wastewater.

4 Conclusion

Titanium dioxide based photocatalysis is an effective measure for the fouling abatement on hollow fiber ultrafiltration membrane. The fouling potential k can be greatly lowered to as much as 90% by this pretreatment measure. On the one hand, photocatlysis can effectively reduce the organic load to membrane surface by the mineralization process, and on the other hand its alteration to the properties of organic matters, such as molecular weight distribution, aromaticity also favorably reduces the potential of fouling formation. The anatase phase TiO_2 is more efficient for the fouling reduction than P25 and rutile TiO_2. The reduction in fouling potential is higher at low pH value about 4.35 and under higher UV intensity, which coincides with the photooxidation kinetics. Titanium dioxide based photocatalysis is a novel process for the fouling reduction of ultrafiltration membrane, and more research efforts are needed to further test and optimize the process.

References

[1] K. Azrague, P. Aimar, 2006. Applied Catalysis B: Environmental 72 (2006) 197–205
[2] H.K. Shon, S. Vigneswarana. 2004. Water Research 38 1933–1939
[3] Nidal Hilal, Oluwaseun O. Ogunbiyi. 2005. Separation Science and Technology, 40: 1957–2005,

[4] Hoffmann, M., et al. 1995. Chem. Rev. 95, 69-96
[5] Chen, P.H., Jenq, C.H., 1998. Environ. Intern. 24 (8), 871–879.
[6] Nádia R.C. Fernandes Machado. 2005. Catalysis Today 107–108, 595–601
[7] L. Song, K.L. Chen. Colloid Interface Sci. 271 (2004) 426.
[8] I.C. Escobar, E.M. Hoek; C.J. Gabelich; F.A. DiGiano; et al (2005) Amer. Wat. Works Ass. J 97,8
[9] Namguk Her, Gary Amy (2003) Water Research 37, 4295–4303

Section 6
Waste water treatment and management

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The effects of flow regime and temperature on the wastewater purification efficiency of a pilot hybrid constructed wetland

I. Zaytsev, K. Nurk, E. Põldvere, A. Noorvee & Ü. Mander
Institute of Geography, University of Tartu, Tartu, Estonia

Abstract

A pilot scale experimental hybrid constructed wetland (CW) was established in 2005 for the treatment of mixed effluents from municipal sources and food processing plants in a small village in southern Estonia. The aim was to find optimal operating regimes (varying hydraulic loading rates, loading intervals, recirculation regimes and regimes of hydraulic retention time) for the use of analogous CWs in cold climates. The hybrid wetland system consists of two parallel systems designed on the same principle: a vertical subsurface flow (VSSF) filter followed by a horizontal subsurface flow (HSSF) filter. The VSSF filter of one system (the left filter) is filled with crushed limestone, while the other (right) is filled with light weight aggregates (LWA). Both HSSF filters are filled with LWA. From December 2005 until December 2006, water samples from the outlet of the septic tank and the outlet of both VSSF and HSSF filters were taken once a week. In both systems, a combination of decreasing hydraulic load (from 52 to 14 mm d-1) and increasing recirculation rate (from 34 to 300%) caused a significant increase in the purification efficiency of BOD7 (from 58 to 99%), N_{tot} (from 11 to 82%), and NH4-N (from -16 to 83%). In both systems, the removal efficiency of P_{tot} did not show any significant correlation with hydraulic load, whereas in the right (LWA) system, the increasing recirculation rate significantly increased P_{tot} removal. The dynamics of the COD value was analogous to that of the BOD7 value, but no significant differences were found between the removal efficiency of total suspended solids (TSS) of different flow regimes. Likewise, water temperature did not show a significant correlation between the purification efficiency of all of the studied water quality indicators. For the successful functioning of hybrid CWs in a cold climate, optimal hydraulic load should be ≤20 mm d-1 with a recirculation rate of 150-300%.
Keywords: aeration, design, hybrid constructed wetland, limestone, LWA, recirculation.

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1 Introduction

Constructed wetlands (CW) of various types are becoming popular worldwide [1]. In addition to their satisfactory performance, multifunctionality in terms of biodiversity and landscape services, and cost-effectiveness, CWs have one significant disadvantage: high area requirement [2]. The idea of compact systems is to sustain systems with lower area requirement and lower building costs. In cold climates, CWs are often designed with a reserve in order to compensate for lower temperatures during winter [3–5]. Providing measures that can help achieve proper results without over-dimensioning would make CW a much more attractive wastewater treatment technology. One possible operational method to compensate for their small area and short retention time is to recirculate the wastewater. As the effluent is being re-circulated, additional oxygen for aerobic microbial activities can be transferred into the wastewater. In addition, as the suspended solids are predominantly removed by filtration, recirculating the effluent increases the chances for the suspended solids to be trapped in the system [6]. These factors should account for the improvement of overall purification processes.

Hydraulic loading and hydrologic detention time are key factors of purification efficiency in all types of constructed wetlands [1]. Excessively high hydraulic load has been reported as a main limiting factor of purification processes in both subsurface flow wetland systems [7, 8] and free water surface wetlands (FWSW [9, 10]). The recirculation of treated effluent from CWs as a procedure enhancing aeration and purification processes has been used in various laboratory experiments and pilot- and full-scale CW systems [11]. Several studies have been carried out in VSSF filters of tidal flow for the treatment of municipal wastewater [12–14] and piggery wastewater [11, 15]. Also, in downflow regime VSSF filters of 1-stage [16–18], 2-stage [19], and 3-stage [20], HSSF filters [21], and hybrid (combined from VSSF and HSSF filters) CWs [22–24] treating domestic and farm effluents, recirculation has been successfully used. Likewise, recirculation has significantly enhanced the purification of landfill leachates [25, 26]. Of water quality indicators, the values of BOD and COD [14, 15, 18, 20, 21], and concentrations of N_{tot} [11, 17, 22, 23], NH_4-N [11, 14, 15, 18–20], P_{tot} [13, 14], total suspended solids (TSS [14, 18]), and polyphenols [21] have been reduced. Several studies report enhanced aeration and increased O_2 consumption by microorganisms [12, 20, 24]. On the other hand, there are studies that point out no significant influence of recirculation on the removal of P_{tot} [18, 27], NH_4-N [16, 28], and indicator bacteria (total and faecal coliforms, faecal streptococci [29]). However, in some countries the recirculation of wastewater in subsurface flow filters has been included in the official guidelines on CWs [16, 27].

Temperature is another key factor controlling purification processes in CWs [1]. However, the influence of temperature in full-scale treatment wetlands and pilot systems demonstrates temperature effects that differ from the results acquired at the laboratory-scale or by modelling. The majority of studies show that in FWSWs, all the purification processes slow down significantly with decreasing temperature [30–34], whereas in some cases the export of nutrients (P_{tot}) from wetlands in winter has been observed [33]. Some authors report that the P retention and mineralization of organic matter (based on BOD value) are the less temperature-affected processes in FWSWs [34], and in some specific cases no differences in the removal of BOD, TSS, NH_4-N and P_{tot} between the warm and cold period were found [35, 36].

Various subsurface flow CWs show a much more diverse pattern of temperature effects on purification processes. The LWA-or Filtralite-P-based hybrid CWs combined from VSSF and HSSF filters in series and treating domestic wastewater show equally high long-term efficiency in both summer and winter [3, 37–40]. Likewise, hybrid subsurface flow CWs used for landfill leachate treatment [41], as well as single or 2-stage VSSF and HSSF filters treating municipal effluents [42, 43] show equally high performance in summer and winter. In contrast, some studies report lower NH_4-N and P removal in subsurface flow wetlands in winter [44, 45], whereas Kuschk et al. [46] report on a significantly lower N removal in an experimental HSSF in the winter season.

The main objective of this study is to determine optimal loading and operational regimes for LWA-based and Limestone-LWA based two-stage hybrid wetland systems in cold climate conditions for the treatment of municipal and agro-industrial wastewater. This paper presents the purification efficiencies during different hydraulic loadings and recirculation regimes and analyses these results against the background of critical factors for water purification.

2 Material and methods

2.1 Site description

The Nõo experimental CW is located on the territory of the active sludge wastewater treatment plant (AWP) of Nõo village. The wastewater (domestic wastewater combined with dairy and meat industry wastewater) is pumped into the CW before it reaches the grid of the AWP. The exact water volume is controlled by a timer-operated pump. A certain amount of wastewater is first pumped into a septic tank (2 m^3). After the septic tank, the wastewater is divided equally between both parallel experimental systems (area of both VSSF filters: 2 x 4 m^2; area of both HSSF filters: 2 x 10 m^2) (Fig. 1).

The pilot scale hybrid CW in Nõo consists of two analogous systems with different operating regimes designed on the same principle: a vertical subsurface flow (VSSF) filter 0.7 m in depth followed by a horizontal subsurface flow (HSSF) filter 1 m in depth. The filters were covered (air temperatures fell to –35°C) with 5 cm thick insulation slabs during winter. The systems were not initially planted. During the vegetation period, plant cover developed spontaneously on the VSSF part of both systems.

Table 1 reports the cross-section of VSSF filters, constructed such that the bottom layer has the highest and the upper layer the lowest hydraulic conductivity. Both HSSF filters are filled with light weight aggregates (LWA) with particle size 2-4 mm.

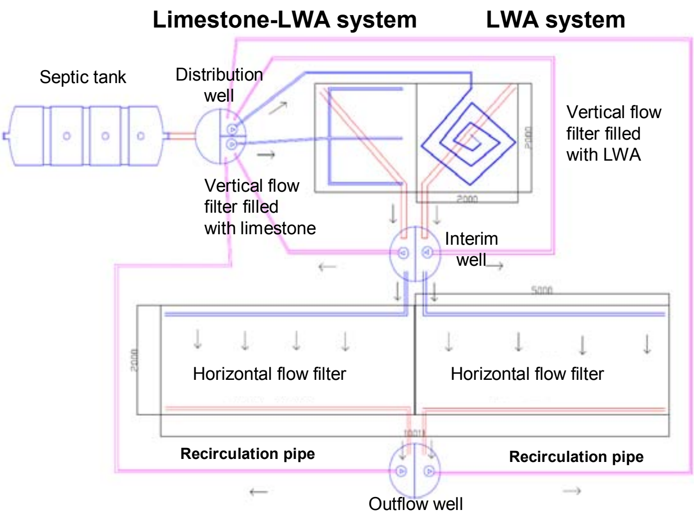

Figure 1: Schematic layout of the experimental pilot system of the hybrid constructed wetland in Nõo, Estonia.

Table 1: Cross-sections of vertical flow filters.

Layers	Right bed	Left bed
Upper layer (20 cm)	crushed limestone Ø 2-8 mm	LWA Ø 2-4 mm
Middle layer (20 cm)	crushed limestone Ø 8-16 mm	LWA Ø 4-10 mm
Bottom layer (25 cm)	crushed limestone Ø 12-32 mm	LWA Ø 10-20 mm

2.2 Operational regimes of hybrid constructed wetland system

There was a possibility to recirculate wastewater from the outflow well of the VSSF filters (interim well), as well as from the outflow of the HSSF filters (outflow well; Fig. 1) using timer-controlled pumps.

This paper covers 6 different operational regimes that were applied simultaneously to both parallel systems. The degree of recirculation was enhanced during the sequential operational regimes, but was not equal for the parallel systems (Table 2).

2.3 Water sampling and analysis

Water samples were taken from 07.12.05 to 20.12.06 once a week simultaneously from both systems, from the outlet of the septic tank and the outlet of both VSSF and HSSF filters. The number of water samples per operational regime (1-6) varied in the following order: 1st – 4 samples, 2nd – 11 samples, 3rd, 4th and 5th – 9 samples each, and 6th – 11 samples. In the laboratory of the AS Tartu Veevärk, water samples were analyzed for BOD_7, TSS, COD, N_{tot}, NH_4-N, NO_2-N, NO_3-N, P_{tot} using Standard Methods for the Examination of Water and Wastewater [47].

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Table 2: Operational regimes and respective recirculating regimes (% from hydraulic loading rate), hydraulic loading rates and average wastewater temperatures in the Nõo hybrid constructed wetland system.

Operational regime	Recirculating regimes of studied systems* LWA		Limestone-LWA		Hydr. loading rate $m^3 d^{-1}$	Average water temp. (^oC)	
	From interim well %	From outflow well %	From interim well %	From outflow well %		Inflow	Out-flow
1	0	34	0	24	0.73	6.5	5.2
2	0	34	0	25	0.37	3.9	1.5
3	48	0	73	0	0.29	7.4	6.2
4	0	250	250	0	0.22	16.4	17.8
5	0	300	150	150	0.3	16.9	16.1
6	150	150	0	300	0.2	8.3	7.2

* summarized recirculation rates from outflow and interim well were used in correlation analysis.

2.4 Statistical analysis

This paper presents data and compares the purification efficiencies of 6 different operational regimes in the limestone-LWA and 6 different operational regimes in the LWA experimental systems. All parameters were controlled for normality (Kolmogorov-Smirnov, Lilliefors' and Shapiro-Wilk's tests). The differences between the purification efficiencies of operational regimes were analysed using Kruskal-Wallis ANOVA. The Mann-Whitney test was used for the comparison of purification efficiencies, and in the case of nitrites and nitrates, for the comparison of the values of the inflow/outflow concentration ratios of the sequential operational regimes. Pearson correlation coefficients were detected between influencing factors and purification efficiencies of BOD_7, N_{tot}, NH_4-N and P_{tot}. We used *Statistica 7.0* for data analysis.

3 Results and discussion

3.1 Inflow concentrations

The inflow concentrations of the studied water quality indicators were typical of the ordinary municipal wastewater, except for COD, which was somewhat higher than in ordinary municipal wastewater [1]. This could be related to the inflow of high concentrations of chemicals used in the local meat and dairy industry (see Table 3).

3.2 Average purification of organic pollutants, suspended solids and nutrients

The analysis of the differences of the experimental cycles by Kruskal-Wallis ANOVA revealed that all studied indicators of the LWA-side (left side) of the CW, except for TSS, showed significant differences between the cycles

($p<<0.01$). The Limestone-LWA side (right side) also showed significant differences for all studied indicators (for BOD_7, COD, N_{tot}, NH_4-N, $p<<0.01$; P_{tot}, NO_2-N and NO_3-N, $p<0.05$), except for TSS.

Table 3: Average and standard deviation values of the concentrations (mg L^{-1}) of studied water quality parameters in the inflow of the studied hybrid constructed wetland system during sequential operational regimes (the header of the columns is indicated by numbers).

Water quality parameter	1	2	3	4	5	6
BOD_7	405±135	413±66	446±216	644±138	640±159	375±75
TSS	132±17	116±36	219±192	287±154	387±396	117±19
COD	690±133	745±120	808±519	1076±369	1135±533	590±59
N_{tot}	65±6	72±22	54±18	91±33	112±33	78±10
NH_4-N	50±11	52±16	47±14	77±39	80±33	62±11
NO_2-N	0.013 ±0.002	0.001 ± 0.0003	0.168 ± 0.22	0.019 ± 0.03	0.013 ± 0.02	0.006 ± 0.003
NO_3-N	0.42 ±0.16	0.12 ±0.1	0.03 ±0	0.41 ±0.35	0.6 ±0.2	0.4 ±0.1
P_{tot}	20±3	21±4	20±8	31±9	44±15	26±5

The lack of significant differences in the purification efficiency of TSS between the cycles is probably related to the relatively high variability of the sedimentation rates of different size fractions of TSS compared to the variability of the sedimentation conditions, such as flow rate during different experimental cycles, and also to the compensating effects of the different stages of wastewater purification. For example, while recirculation decreases sedimentation due to the enhanced speed of the water flow inside the HSSF, this may be compensated by higher oxidation of TSS in the VSSF.

3.3 The comparison of the studied water quality indicators during the sequential operational regimes

The comparison of the purification efficiency of BOD_7, N_{tot}, NH_4-N, and P_{tot} between the sequential operational regimes in both LWA and Limestone-LWA systems during the experiment period (Mann-Whitney tests) showed general growth trends for all studied parameters, except for P_{tot} in the Limestone-LWA system (Fig. 2).

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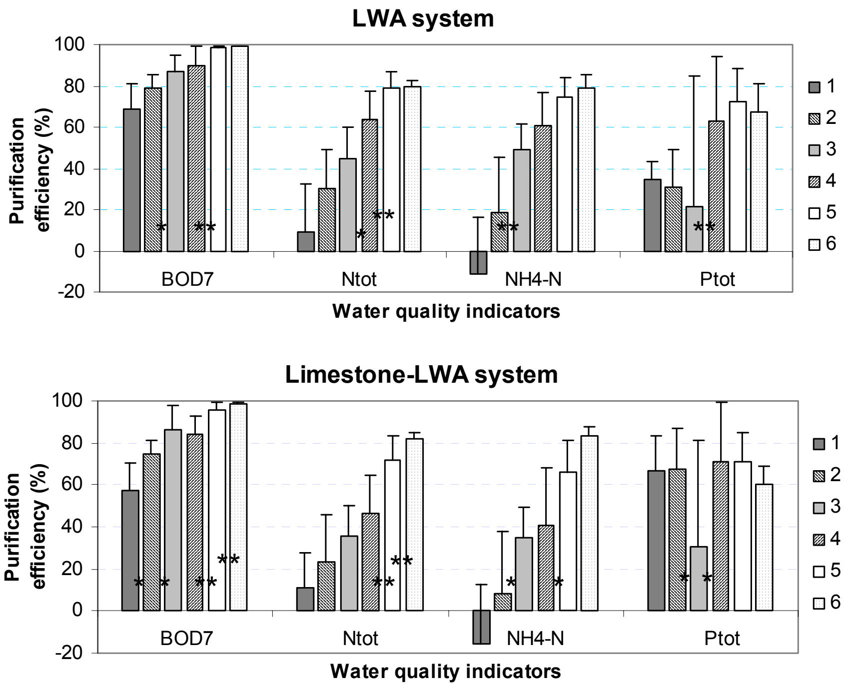

Figure 2: Average values and standard deviation (bars) of purification efficiency of BOD_7, N_{tot}, NH_4-N and P_{tot}, (%) in both LWA and Limestone-LWA system beds during various operational regimes (1-6; see Table 2). Asterisks indicate significant differences (Mann-Whitney test) between neighbouring operational regimes: * - $p < 0.05$, ** - $p < 0.01$.

The LWA system could remove up to 99% BOD, 81% N_{tot}, 79% NH_4-N, and 67% P_{tot}, whereas in the Limestone-LWA system these values were 99, 82, 83 and 60% respectively (Fig. 2).

3.3.1 LWA system

The growth of the removal efficiency of BOD_7 (Fig. 2) in the 3rd operational regime compared to the 2nd operational regime could be caused by the enhanced recirculation rate, decreasing rates of wastewater inflow to the CW, and also by increasing water temperature due to seasonal changes. However, growth in the 5th operational regime compared to the 4th operational regime was obviously caused by enhanced recirculation, which could have a crucial effect on relief of oxygen deficit due to higher water temperatures.

N_{tot} removal efficiency showed an increasing trend throughout the study period, and thus the purification processes of N_{tot} were influenced by recirculation, which enables sufficient aeration for the degradation of organic N and oxidation of ammonia. Significant growth of the purification efficiency of

NH_4-N in the 3rd operational period compared to the 2nd operational period that occurred in the relatively colder season was probably caused by higher oxygen solubility in lower temperatures, indicating a lower temperature optimum for nitrification compared to denitrification. The significant growth of P_{tot} after the steep enhancing of the recirculation rate in the 4th operational regime compared to the 3rd operational regime indicates that the processes of P retention in the LWA system were supported by enhanced aeration [1].

3.3.2 Limestone – LWA system

The comparison of the purification efficiency of BOD_7 between the sequential operational regimes showed additional significant differences between the operational regimes compared with the LWA system (Fig. 2). These differences in the systems could be caused by the relatively poor properties of limestone compared to LWA in terms of aeration and insulation, which enhance the effects of other factors such as hydraulic loading rate and temperature regime. The same is obviously valid for N_{tot} and NH_4-N, which also gave some additional significant differences between operational regimes compared with the LWA system. The purification efficiency of P_{tot} during the first two operational regimes was nearly twice as higher in the limestone-LWA system than in the LWA system, which could be related to the higher P retention capacities of $CaCO_3$-rich limestone. The significantly lower purification efficiency of P_{tot} during the 3rd operational regime compared to the preceding and following operational regime probably reflects the influence of the lower water temperature and the switching of the recirculation from the outflow well to the interim well. This could have an effect on the reaction balances in the vertical filter, because enhanced inflow of O_2-rich water from the interim well may suppress the solubility of CO_2. Thus the average CO_2 concentration could become the limiting factor in the geochemical pathway that leads to the final sedimentation of P.

3.4 The changes in the inflow/outflow concentration ratio for nitrites and nitrates during sequential operational regimes

The inflow/outflow concentration ratio of NO_2-N was significantly lower in both systems during the 1st operational regime compared to the 2nd operational regime. The comparison of the latter cycles did not reveal significant changes in the production of NO_2-N (Table 4). These commonalities are probably related to the initial phases of the development of a nitrifying microbial community. The initial phases of nitrification – nitrite production – was not limited by NH_4-N availability. The next phase of nitrification, nitrate production – is limited by the production of nitrite, and thus the community of nitrite producers should be developing faster and should be located closer to the inflowing resource, NH_4, especially in the case of diffusion-limited mass transport processes in the wetland substrate, which is discussed in greater detail by Austin et al. [48]. We suggest that this situation should gradually change towards complete nitrification, especially when high recirculation rates are applied.

Significant growth of the inflow/outflow concentration ratio of NO_3-N in the 6th operational regime compared to the 5th operational regime in the LWA system is probably caused by the enhanced oxygen solubility due to the lowered wastewater temperature in the autumn. LWA-based vertical filter has a better ability to enrich the water with oxygen, and thus the effect of lowered temperature could also have a stronger effect on nitrification. The lower inflow/outflow concentration ratio of NO_3-N in the Limestone-LWA system during the 4th operational regime compared to the 3rd operational regime could also be caused by enhanced wastewater temperature in the 4th operational regime, which lowers the solubility of O_2 (Table 4).

Table 4: Average (aver) and standard deviation (stdev) values of the inflow/outflow concentration ratio for nitrites and nitrates in both LWA and limestone beds. << indicates significantly differing values between the neighbouring operational regimes (Mann-Whitney test, $p < 0.01$).

Cycles		1		2	3		4	5		6
NO_2 aver	LWA	0.23	<<	79.6	26.3		43.5	26.6		18.9
	stdev	0.31		158.5	50.8		65.2	43.3		27.4
NO_2 aver	limestone	0.29	<<	1.06	6.11		19.6	19.5		22.3
	stdev	0.29		0.39	16.5		29.4	19.3		28.0
NO_3 aver	LWA	0.54		1.05	1.00		3.03	0.92	<<	2.2
	stdev	0.40		0.51	0.00		5.13	1.14		1.4
NO_3 aver	limestone	0.46		0.90	1.00	<<	0.68	0.99		1.8
	stdev	0.30		0.33	0.00		0.25	0.62		1.1

3.5 Impact of temperature and flow regimes

Pearson correlation coefficients between the purification efficiency of BOD_7, N_{tot}, NH_4-N, P_{tot} and the main regulators of wastewater purification efficiency (hydraulic load, recycling regime, wastewater temperature) were detected.

3.5.1 Impact of water temperature

Wastewater temperature in the inflow and outflow had no significant correlations with the purification efficiency of BOD_7, N_{tot}, NH_4-N and P_{tot}. There were only weak positive relationships below the accepted level of significance, and R^2 varied between 0.10 and 0.58. This situation seems to be typical to CW-s due to the trade-off between the effects of temperature-sensitive factors such as O_2 solubility in the water and (bio)chemical reaction rates.

3.5.2 Impact of hydraulic load and recirculation regime

The most influential factor for wastewater purification efficiency was the wastewater recirculation regime.

In the LWA system, recirculation regime had highly significant positive correlations with the purification efficiency of N_{tot} ($p<0.01$, $R^2= 0.85$), NH_4-N

($p<0.01$, $R^2= 0.94$), P_{tot}, ($p<0.01$, $R^2= 0.94$) and a significant correlation with BOD_7 removal efficiency ($p<0.05$, $R^2= 0.76$), whereas hydraulic load was negatively correlated with BOD_7 ($p<0.05$, $R^2= 0.74$) and N_{tot} purification efficiency ($p<0.05$, $R^2= 0.71$), and NH_4-N had a non-significant tendency towards a negative relationship with hydraulic load ($p=0.07$, $R^2= 0.59$).

The more significant effect of the recirculation regime on the purification efficiencies of N-compounds is consistent with general knowledge about environmental needs for nitrogen processing pathways. A significant correlation between recirculation regime and P purification efficiency could be explained by the redox-sensitivity of the Fe and Al reactions of P, which is transformed into stable, non-soluble form in aerobic conditions, and into soluble form in anaerobic conditions [1].

The Limestone-LWA system also showed a highly significant positive correlation between recirculation rate and N_{tot} ($p<0.01$, $R^2= 0.86$), and significant positive correlations of recirculation rate with NH_4-N ($p<0.05$, $R^2= 0.81$) and BOD_7 purification efficiency ($p<0.05$, $R^2= 0.87$), but lacked a correlation of recirculation rate with P_{tot} purification efficiency ($p=0.56$, $R^2= 0.09$), whereas hydraulic load was negatively correlated with BOD_7 ($p<0.05$, $R^2= 0.79$) and NH_4-N purification efficiency ($p<0.05$, $R^2= 0.71$), and had a non-significant tendency of a negative relationship between hydraulic load and N_{tot} purification efficiency ($p=0.093$, $R^2= 0.55$). The absence of a positive correlation for P_{tot} purification efficiency and recirculation regime in this system could be related to the previously described mechanism that depends on dissolved CO_2 (see section 3.3.2), does not depend on the redox potential in the filter media and therefore is not influenced by the recirculation rate.

The lack of correlation between the hydraulic loading rate and P_{tot} purification efficiencies in both systems indicates that P transformation and subsequent retention in the CW was not limited by the diffusion rates of P compounds and was controlled by the reaction rates at specific steps of P transformation. The stronger correlation of NH_4-N purification efficiency and recirculation regime in the LWA system than in the Limestone-LWA system shows that NH_4-N purification processes in the LWA system were more sensitive to additional aeration than in the limestone LWA system, which could be explained by the greater specific area of the LWA-based vertical filter.

Our results suggest that optimal hydraulic load for the hybrid CWs in cold climates should be up to 20 mm d^{-1} with a recirculation rate of 150-300%. This is coherent with the data in the literature. In milder climatic conditions higher hydraulic loadings can be optimal [8] and also 100-200% recycling rate can give 52-65% N_{tot} removal [17] and even 65-70% NH_4-N removal with a detention time of only 3.5 days [19]. In tropical regions already 50% recirculation guarantees increase in NH_4-N removal from 71 to 85% [22]. In cold climate regions a recirculation rate up to 150% can increase the removal of BOD, and NH_4-N up to 81 and 62% respectively [18]. Thanks to the higher recirculation rate, the results of our study were higher (Fig. 2).

4 Conclusions

Purification efficiencies of BOD_7, N_{tot} and NH_4-N responded negatively to the increases in hydraulic loading rates, and positively to the enhancing of recirculation rate, whereas the purification efficiency of P_{tot} only showed a positive response to enhanced recirculation rate in the LWA system, and was not affected by the changes in the hydraulic loading rates.

The purification efficiency of the studied indicators did not depend on the wastewater temperature. The overall production of NO_2 throughout the studied operational regimes in both systems shows that CWs were in the state of the initial phase of the development of a nitrifying microbial community.

The removal efficiency of BOD_7 depends equally on the enhancing of the recirculation rate and the lowering of the hydraulic loading rates, whereas purification efficiencies of N compounds were related to the increase in recirculation.

Thanks to the higher specific area of the filter material, the LWA system showed somewhat higher sensitivity of N removal to increased recirculation rate than the Limestone-LWA system.

The fact that the purification efficiency of the P_{tot} in the Limestone-LWA system was somewhat higher than in the LWA system and did not depend on recirculation rates as in the LWA system indicates the different mechanisms of P retention were predominant in different systems. The Limestone-LWA system probably retained P through sedimentation reactions with Ca, whereas the LWA system retained P in redox-potential dependent reactions of P with soil Fe and Al.

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References

[1] Kadlec, R.H. & Knight, R.L., *Treatment Wetlands*. Lewis Publisher, Boca Raton, New York, 893 p., 1996.

[2] Vymazal, J. Types of constructed wetlands for wastewater treatment: their potential for nutrient removal. *Transformations of Nutrients in Natural and Constructed Wetlands*, Vymazal, J.. Backhuys Publishers, Leiden, The Netherlands, pp. 1-96, 2001.

[3] Jenssen, P.D., Maehlum, T. & Krogstad, T., Potential use of constructed wetlands for waste-water treatment in northern environments. *Water Sci. Technol.*, 28(10), pp. 149-157, 1993.

www.witpress.com, ISSN 1743-3541 (on-line)

[4] Wallace, S., Parkin, G. & Cross, C., Cold climate wetlands: design and performance. *Water Sci. Technol.*, 44(11-12), pp. 259-265, 2001.

[5] Werker, A.G., Dougherty, J.M., McHenry, J.L. & Van Loon, W.A., Treatment variability for wetland wastewater treatment design in cold climates. *Ecol. Eng.*, 19(1), pp. 1-11. 2002.

[6] Sun G., Gray K.R., Biddlestone A.J., Allen S.J. & Cooper D.J., Effect of effluent recirculation on the performance of a reed bed system treating agricultural wastewater. *Process Biochem.*, 39(3), pp. 351-357, 2003.

[7] De Sousa, J.T., van Haandel, A.C. & Guimaraes, A.A.V., Post-treatment of anaerobic effluents in constructed wetland system. *Water Sci. Technol.*, 44(4), pp. 213-219, 2001.

[8] Weedon, C.M., Compact vertical flow constructed wetland systems – first two years' performance. *Water Sci. Technol.*, 48(5), pp. 15-23, 2003.

[9] Kadlec, R.H., Nitrogen farming for pollution control. *J. Environ. Sci. Heal. A*, 40(6-7), pp. 1307-1330, 2005.

[10] Toet, S., Van Logtestijn, R.S.P., Schreijer, M., Kampf, R. & Verhoeven, J.T.A., The functioning of a wetland system used for polishing effluent from a sewage treatment plant. *Ecol. Eng.*, 25(1), pp. 101-124, 2005.

[11] Sun, G., Zhao, Y. & Allen, S., Enhanced removal of organic matter and ammoniacal-nitrogen in a column experiment of tidal flow constructed wetland system. *J. Biotechnol.*, 115(2), 189-197, 2005.

[12] Sun, G., Gray, K.R., Biddlestone, A.J. & Cooper, D.J., Treatment of agricultural wastewater in combined tidal flow-downflow reed bed system. *Water Sci. Technol.*, 40(3), pp. 139-146, 1999.

[13] Farahbakhshazad, N. & Morrison, G.M., Phosphorus removal in a vertical upflow constructed wetland system. *Water Sci. Technol.*, 48(5), pp. 43-50, 2003.

[14] Zhao, Y.Q., Sun, G. & Allen, S.J., Purification capacity of a highly loaded laboratory scale tidal flow reed bed system with effluent recirculation. *Sci. Total Environ*. 330(1-3), pp. 1-8, 2004.

[15] Sun, G., Zhao, Y., Allen, S. & Cooper, D., Generating "tide" in pilot-scale constructed wetlands to enhance agricultural wastewater treatment. *Eng. Life Sci.*, 6(6), pp. 560-565, 2006.

[16] Bahlo, K. 2000. Treatment efficiency of a vertical-flow reed bed with recirculation. *J. Environ. Sci. Heal. A*, 35(8), pp. 1403-1413, 2000.

[17] Arias, C.A., Brix, H. & Marti, E., Recycling of treated effluents enhances removal of total nitrogen in vertical flow constructed wetlands. *J. Environ. Sci. Heal. A*, 40(6-7), pp. 1431-1443, 2005.

[18] He, L.S., Liu, H.L., Xi, B.D. & Zhu, Y.B., Enhancing treatment efficiency of swine wastewater by effluent recirculation in vertical-flow constructed wetland. *J. Environ. Sci.*, 18(2), pp. 221-226, 2006.

[19] White, K.D., Enhancement of nitrogen removal in subsurface flow constructed wetlands employing a 2-stage configuration, an unsaturated zone, and recirculation. *Water Sci. Technol.*, 32(3), pp. 59-67, 1995.

[20] Sun, G., Gray, K.R. & Biddlestone, A.J., Treatment of agricultural and domestic effluents in constructed downflow reed beds employing recirculation. *Environ. Technol.*, 19(5), pp. 529-536, 1998.
[21] Del Bubba, M., Checchini, L., Pifferi, C., Zanieri, L. & Lepri, L., Olive mill wastewater treatment by a pilot-scale subsurface horizontal flow (SSF-h) constructed wetland. *Ann. Chim.-Rome*, 94(12), pp. 875-887, 2004.
[22] Kantawanichkul, S., Neamkan, P. & Shutes, R.B.E., Nitrogen removal in a combined system: vertical vegetated bed over horizontal flow sand bed. *Water Sci. Technol.*, 44(11-12), pp. 137-142, 2001.
[23] Rustige, H. & Platzer, C., Nutrient removal in subsurface flow constructed wetlands for application in sensitive regions. *Water Sci. Technol.*, 44(11-12), pp. 149-155, 2001.
[24] Shi, L., Wang, B.Z., Cao, X.D., Wang, J., Lei, Z.H., Wang, Z.R., Liu, Z.Y. & Lu, B.N., Performance of a subsurface-flow constructed wetland in Southern China. *J. Environ. Sci.*, 16(3), pp. 476-481, 2004.
[25] Connolly, R., Zhao, Y., Sun, G. & Allen, S., Removal of ammiacal-nitrogen from an artificial landfill leachate in downflow reed beds. *Proc. Biochem.*, 39(12), pp. 1971-1976, 2003.
[26] Zhao, Q.L., Liu, X.Y., Qi, X.D. & Liu, Z.G., Landfill leachate production, quality and recirculation treatment in northeast China. *J. Environ. Sci.*, 18(4), pp. 625-628, 2006.
[27] Brix, H. & Arias, C.A., The use of vertical flow constructed wetlands for on-site treatment of domestic wastewater: New Danish guidelines. *Ecol. Eng.*, 25(5), pp. 491-500, 2005.
[28] Moreno, C., Farahbakhshazad, N. & Morrison, G.M., Ammonia removal from oil refinery effluent in vertical upflow macrophyte column systems. *Water Air Soil Poll.*, 135(1-4), pp. 237-247, 2002.
[29] Arias, C.A., Cabello, A., Brix, H., Johansen, N.-H., Removal of indicator bacteria from municipal wastewater in an experimental two-stage vertical flow constructed wetland system. *Water Sci. Technol.*, 48(5), pp. 35-41, 2003.
[30] Kadlec, R.H. & Hey, D.L., Constructed wetlands for river water quality improvement. *Water Sci. Technol.*, 29(4), pp. 159-168, 1994.
[31] Newman, J.M., Clausen, J.C. & Neafsey, J.A., Seasonal performance of a wetland constructed to process dairy milkhouse wastewater in Connecticut. *Ecol. Eng.*, 14(1-2), pp. 181-198, 2000.
[32] Spieles, D.J. & Mitsch, W.J., The effects of season and hydrologic and chemical loading on nitrate retention in constructed wetlands: a comparison of low- and high-nutrient riverine system. *Ecol. Eng.*, 14(1-2), pp. 77-91, 2000.
[33] White, J.S. & Bayley, S.E., Nutrient retention in a northern prairie marsh (Frank Lake, Alberta) receiving municipal and agro-industrial wastewater. *Water Air Soil Poll.*, 126(1-2), pp. 63-81, 2001.
[34] Kadlec, R.H. & Reddy, K.R., Temperature effects in treatment wetlands. *Water Environ. Res.*, 73(5), pp. 543-557, 2001.

[35] Mander, Ü. & Mauring, T., Constructed wetlands for wastewater treatment in Estonia. *Water Sci. Technol.*, 35(5), pp. 323-330, 1997.

[36] Smith, E., Gordon, R., Madani, A. & Stratton, G., Year-round treatment of dairy wastewater by constructed wetlands in Atlantic Canada. Wetlands, 26(2), pp. 349-357, 2006.

[37] Maehlum, T., Jenssen, P.D. & Warner, W.S., Cold-climate constructed wetlands. *Water Sci. Technol.*, 32(3), pp. 95-101, 1995.

[38] Maehlum, T. & Stalnacke, P., Removal efficiency of three cold-climate constructed wetlands treating domestic wastewater: Effects of temperature, seasons, loading rates and input concentrations. *Water Sci. Technol.*, 40(3), pp. 273-281, 1999.

[39] Jenssen, P.D., Maehlum, T., Krogstad, T. & Vrale, L., High performance constructed wetlands for cold climates. *J. Environ. Sci. Heal. A*, 40(6-7), pp. 1343-1353, 2005.

[40] Öövel, M., Tooming, A., Mauring, T. & Mander, Ü., Schoolhouse wastewater purification in a LWA-filled hybrid constructed wetland in Estonia. *Ecol. Eng.*, 29(1), pp. 17-26, 2007.

[41] Bulc, TG., Long term performance of a constructed wetland for landfill leachate treatment. *Ecol. Eng.*, 26(4), pp. 365-374, 2006.

[42] Schönborn, A., Züst, B. & Underwood, E., Long term performance of the sand-plant-filter Schattweid (Switzerland). *Water Sci. Technol.*, 35(5), pp. 307-314, 1997.

[43] Merlin, G., Pajean, J.L. & Lissolo, T., Performances of constructed wetlands for municipal wastewater treatment in rural mountainous area. *Hydrobiologia*, 469(1-3), pp. 87-98, 2002.

[44] Sikora, F.J., Tong, Z., Behrends, L.L., Steinberg, S.L. & Coonrod, H.S., Ammonium removal in constructed wetlands with recirculating subsurface flow: Removal rates and mechanisms. *Water Sci. Technol.*, 32(3), pp. 193-202, 1995.

[45] Steer, D., Fraser, L., Boddy, J. & Seibert, B., Efficiency of small constructed wetlands for subsurface treatment of single-family domestic effluent. *Ecol. Eng.*, 18(4), pp. 429-440, 2002.

[46] Kuschk, P., Wiessner, A., Kappelmeyer, U., Weissbrodt, E., Kastner, M. & Stottmeister, U., Annual cycle of nitrogen removal by a pilot-scale subsurface horizontal flow in a constructed wetland under moderate climate. *Water Res.*, 37(17), pp. 4236-4242, 2003.

[47] APHA. *Standard Methods for the Examination of Water and Waste Water*. American Public Health Organisation, 17th edition, Washington, 1989.

[48] Austin, D., Maciolek, D., Davis, B. & Wallace, S., Damköhler number design method to avoid plugging of tidal flow constructed wetlands by heterotrophic biofilms. *10th International Conference on Wetland systems for Waste Water Pollution Control, September 23-29, 2006, Lisbon, Portugal, IWA*, pp. 1147-1156, 2006.

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Urban wastewater reuse: water treatment and effectiveness on antibiotic-resistant bacteria abatement

G. Del Re[1], A. Di Donato[1], R. Volpe[1] & M. G. Perilli[2]
[1]*Department of Chemistry, Chemical Engineering and Materials, L'Aquila University, Italy*
[2]*Department of Biomedical Sciences and Technologies, L'Aquila University, Italy*

Abstract

In Abruzzo Region (Italy) the implementation of European Community and National regulations for urban wastewater reuse has been carried out since 2003, when the Environmental National Authority issued the D.M. 185 giving technical rules and water quality standards for three types of reuse: irrigation, civil aims and industry. Pilot plant experiences of tertiary treatments added to conventional urban wastewater treatment plants (UWWTPs) have been carried out, in order to test the suitability of different disinfection technologies to reach the standards required for an effluent to be reused. In addition other parameters affecting the quality of reclaimed water, and for which there is currently no regulation or control, have been investigated; among them, antibiotic resistant bacteria concentration. To evaluate the extent of problems related to antibiotic resistant bacteria and the effectiveness of tertiary treatment on their abatement level, a significant portion of the wastewater network of the city of L'Aquila (Italy) and the related wastewater treatment plant were monitored. Antibiotic resistant bacteria content in non-treated sewage from municipal hospital and residential areas was evaluated. The UWWTP treating this sewage is of activated sludge type and is located near the Aterno River. The wastewater coming from the secondary treatment and from the tertiary treatments was analysed in order to investigate the influence on the antibiotic-resistant bacteria content. An analogous investigation was carried out in the river upstream and downstream the point of discharge of the UWWTP.

Keywords: wastewater treatment, water reuse, antibiotic-resistant bacteria.

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1 Introduction

Water treatment matters are never obsolete and continuously evolving. One of the most recent concerns is about wastewater reuse. Water reuse practice is a viable and effective sustainable development policy application, meaning significant reduction of water sources withdrawals and, on the other hand, corresponding increase of potable water availability. In particular, urban wastewater reuse should allow the saving of a great part of the whole water consumption in the developed world if it would become a common practice and not only a contingent solution for drought situations. However, care must be paid on microbiological safety of reclaimed water, even for the aspects not yet considered by both national and international regulations but involving matters of potential risks for human health; one of them, investigated in this work, is the presence of antibiotic-resistant bacteria in wastewater distribution network.

Italian Government established a regulation on water reuse which consists of decree "D.M. n.185/2003", issued by the National Environmental Authority (Ministero dell'Ambiente) complying with Article 26 of the Italian Unified Body of Laws for protection of waters, the Decree n. 152/1999. Decree n.152/1999 implements the European Directives "Urban Waste Water Treatment - Directive 91/271/EEC" and "Protection of waters against pollution caused by nitrates from agricultural sources – Directive 91/676/EEC".

As reported in detail elsewhere [1], D.M. n.185/2003 establishes the quality standards and technical rules for urban wastewater reuse for three reuse categories: irrigation, industrial and civil. D.M. n.185/2003 establishes that regional Authorities must individuate a first set of urban wastewater treatment plants (UWWTPs) whose effluents have to be reused. Such plants are those one whose effluents, in the current configuration, have only few parameters exceeding the limits established for water reuse, allowing to meet the standards for water reuse with affordable investments for plant modifications. Regional Authorities have also to choose, among the suitable best available technologies, the additional treatments needed to meet the quality standards for water reuse and they have to indicate, for each UWWTP, the most convenient types of reuse, suggested by an accurate analysis of the plant surrounding area. Wastewater reclamation in agriculture appears to be a key topic, especially in Southern Italian regions [2]. The quality standards for water reuse established by D.M. 185/2003 are very stringent, in comparison to the water quality indicated by the regulation for protection of surface waters (Decree n.152/1999), especially for parameters such as total suspended solids (TSS), microbial pollution indicators (fecal coliform Escherichia Coli, EC) or heavy metals; moreover, it introduces parameters not considered in the emission limits for urban wastewater plants, such as total trihalomethanes (TTHM) and chlorinated pesticides.

Most of the UWWTPs in operation in the Abruzzo Region are of activated sludge or rotating biodisc type. When they are properly designed and operated their effluents have negligible heavy metal concentrations, and the only parameters which do not comply with the standards for water reuse are TSS and EC; consequently, in order to meet the standards for water reuse, UWWTPs have to be modified including a proper filtration section for TSS reduction and a disinfection process (for EC reduction) based on technology alternative to chlorine disinfection. The chlorine concentration to be used to meet the stringent limits on microbiological parameters is incompatible with the chlorine residuals. In addition, when using chlorine, harmful chloro-organic by products can be formed.

The best disinfection technologies alternative to chlorine disinfection commercially available are based on the use of UV, ozone and microfiltration [3, 4]; Peracetic Acid (PAA) has been proposed for wastewater disinfection treatments, but its behaviour is still under assessment [5, 6]. Its use has some drawbacks, such as the increase of organic content in the treated effluent, the potential microbial re-growth due to the residual acetic acid, the limited efficiency against viruses and parasites and the strong dependence on wastewater quality [7].

Some water reuse projects are in operation in Europe: Bixio et al [8], have identified about 200 reuse projects, most of them of small (0,1-0,5 Mm^3/y) or medium size (0,5-5 Mm^3/y). The experiences of the different projects are not directly comparable, as they are based on different technologies, are intended for different uses and refer to different water reuse criteria.

The lack of a systematic and properly managed experience of wastewater reuse and, consequently, the lack of reliable data asserting the effectiveness of the different filtration and disinfection treatments to meet the enforced target indicators, the reasonable doubts expressed by the scientific community on the exhaustive character of the enforced laws in the limitation of the risk involved in wastewater reuse practice [9–12] make local authorities and water management stakeholders to urge researchers and technicians to conduct suitable experimental works at pilot plant scale before initializing any massive intervention.

An experimental campaign was conducted on a municipal biodisc WWTP serving a community of 7,000 P.E.; the effluent was treated in a tertiary treatment pilot plant (TTPP) consisting of rotary filtration (RF) and UV disinfection.

In addition, an investigation of the presence of antibiotic-resistant bacteria at the inlet and outlets of the TTPP and at different locations of a 28,000 P.E. activated sludge UWWTP were carried out, as presented in the following paragraphs. Antibiotic-resistant bacteria collected from and disposed on environment and travelling through the whole wastewater treatment network is an urgent, and perhaps underestimated, problem of the 21st century [13–19].

2 Experimental

This work presents the results of two experiences of water treatment monitoring in which an antibiotic bacteria presence investigation has been carried out.

The first experience deals with a typical domestic wastewater treatment plant in which a tertiary pilot scale plant, consisting of rotary filtration followed by UV disinfection, was added and its effectiveness on antibiotic-resistant bacteria abatement was compared with the performances of the existent tertiary treatment, i.e. chlorination.

The second experience reports the preliminary results of antibiotic-resistance investigation in a network collecting and treating sewage of different origins, not only typical domestic as in the first case: wastewater from hospitals, universities, houses and small-medium farms are collected, then treated in an activated sludge plant and disposed in a river.

Paragraphs 2.1 and 2.2 report sampling locations and analysis methods for the two experiences, respectively; paragraphs 3.1 and 3.2 report the corresponding results and discussion.

2.1 Pilot plant experience

The schematic diagram of the biodisc UWWTP plant is shown in Figure 1, together with the added tertiary treatment pilot plant (TTPP). The plant serves a community of about 7,000 P.E., no significant industrial activities are present in the served area, so the waste water treated by the plant is substantially domestic water.

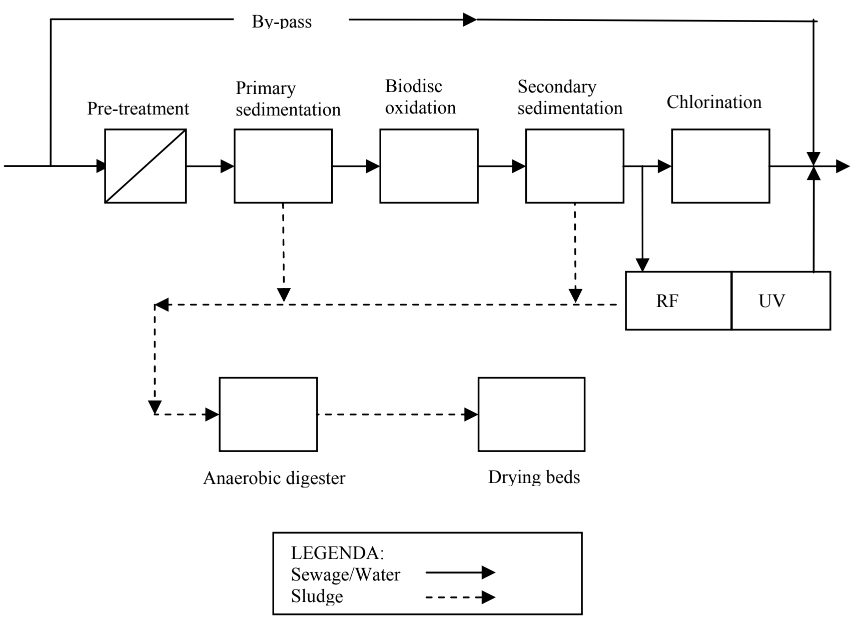

Figure 1: UWWTP schematic diagram, with the TTPP (RF+UV) section.

The TTPP was designed to treat a flow-rate of about 7 m^3/h, but different flow rates have been used during the experience in order to test the effect of different UV doses and to evaluate the operative conditions that allow one to best attain the water quality targets.

The period of the experience was of 14 weeks, from spring to summer, with an average water temperature at UWWTP output of about 15°C.

The main objective of the work, as reported in [20], was to evaluate performances and reliability of a tertiary treatment of a municipal WWTP effluent, in order to meet the quality standards for water reuse.

In this work we want to focus on the investigation of the presence of antibiotic-resistant bacteria at the inlet and outlets of the TTPP.

Six 250 ml samples were collected from the influent and effluent of the TTPP, distributed over the experience period. All samples were collected in sterile propylene bottles and transported to the laboratory for immediate processing. 1 ml from every sample was diluted with physiological sterile NaCl solution (0,9 g/l NaCl). 100 μl volume from each dilution was then plated on a Nutrient Agar culture medium. The plates were incubated for 24 h , 48 h and 72 h at 37°C for the determination of the bacterial counts and converted on CFU/ml. In order to obtain isolated colonies for preliminary identification, dilutions of each sample were plated on selective medium. The agar plates were incubated for 24 h at 37°C. For pure cultures, a representative for each different colony morphology was re-isolated on clear plates and incubated for 24 h at 37°C. The Enterotube (Becton Dickinson), API-20E (Biomerieux) and API-20NE (Biomerieux) systems were used for the preliminary identification of some gram-negative isolates. All isolated strains were identified by Phoenix system (Becton Dickinson).

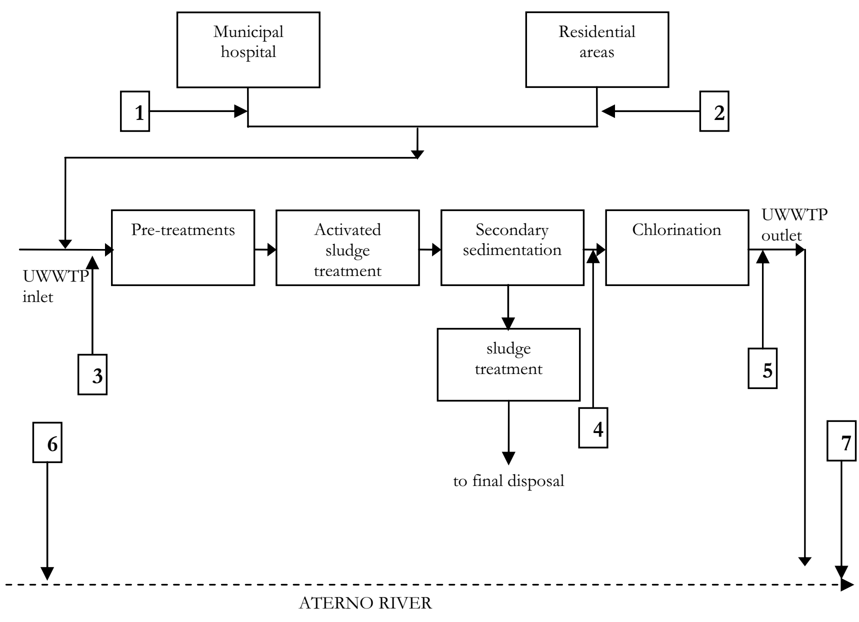

Figure 2: Wastewater network monitoring scheme.

2.2 Wastewater network monitoring

It was scheduled to monitor the antibiotic-resistant bacteria presence by sampling wastewater in different locations, as shown in Figure 2: non-treated sewage from municipal hospital (Sampling Location 1) and residential areas (SL 2); UWWTP inlet (SL 3); secondary treatment outlet (SL 4) and tertiary treatment outlet (SL 5); in the river upstream (SL 6) and downstream (SL 7) the point of discharge of the UWWTP.

Sampling frequency was about once a month. The work is still in progress; in this work we report the preliminary results for the first six sampling series, obtained from June 2006 to January 2007.

Samples have been collected and analysed for total bacterial count in the same way as reported for the first experience. The isolates identification in this case has been carried out using Imipenem 2 μg/ml and Ceftadizime 4 μg/ml, i.e. two of the most effective antibiotics used nowadays.

Table 1: Total number of strains selected in this study.

Strains	*Total Number of Strains*	*N. of strains after UV* (%)	*N. of strains after chlorination* (%)
Achromobacter spp.	2	-	-
Acinetobacter lowffii	4	3 (75%)	1 (25%)
Aereomonas caviae	18	2 (11%)	5 (28%)
Aereomonas hydrophila	3	1 (33.3%)	-
Aereomonas sobriae	7	2 (28%)	1 (14.2%)
Aereomonas veronii	1	1 (100%)	-
Alcaligenes faecalis	1	-	-
Citrobacter braaki	1	-	1 (100%)
Citrobacter farmeri	1	-	-
Citrobacter freundii	7	-	7 (100%)
Citrobacter werkmanii	1	-	-
Citrobacter youngae	1	-	1 (100%)
Comamonas testosteroni	2	1 (50%)	-
Delfia acidovorans	2	2 (100%)	-
Enterobacter asburiae	1	-	-
Enterobacter cloacae	4	-	4 (100%)
Enterobacter intermedius	2	-	2 (100%)
Escherichia coli	17	2 (11.7%)	10 (58.8)
Escherichia vulneris	1	-	1 (100%)
Klebsiella oxytoca	7	-	3 (42.8)
Klebsiella pneumoniae	3	-	3 (100%)
Kluyvera ascorbata	1	-	1 (100%)
Mannheimia haemolytica	1	-	1 (100%)
Morganella morganii	1	-	-
Pantoea agglomerans	1	-	-
Providencia rettgeri	1	-	-
Pseudomonas aeruginosa	1	-	-
Pseudomonas mendocina	2	-	2 (100%)
Pseudomonas putida	2	-	1 (50%)
Pseudomonas species	2	-	2 (100%)
Shigella boydii	1	-	-
Tatumella ptyseos	1	-	-
Total	*100*	*14 %*	*46 %*

3 Results and discussion

3.1 Pilot plant results

When microorganisms were growth on the selective medium 100 different strains belonging to the several species were identified. The most common organism detected was Aeromonas cavie (18 strains) followed by E. coli (17 strains) and Klebsiella oxytoca (7 strains). The other species were represented only by few isolates.

As shown in Table 1 after UV treatment only 14% of total strains were detected whereas 46% of strains were detected after chlorination treatment.

Some bacterial species were completely destroyed by both treatments. In particular Enterobacter spp, Klebsielle spp., Pseudomonas spp and Citrobacter freundii strains were eliminated only after UV treatment, whereas Aeromonas hydrophila, Aeromonas veronii, Comomonas testosteronii and Delfia spp. were destroyed only by chlorination treatment. The phenotypic profile of all 100 strains collected was carried out by minimal inhibitory concentration experiments using ß-lactam antibiotics and an inoculum of 10^5 CFU/ml. Citrobacter spp., E. coli, Klebsiella spp., Aeromonas spp. and Pseudomonas spp. strains showed an evident resistance phenotype toward penicillins and cephalosporins tested. In particular several bacterial were resistant to oxyimino-cephalosporins such as cefotaxime and ceftazidime. The resistance toward these two ß-lactams were typical of Extended Spectrum ß-Lactames (ESßL)-producing strains that represents an emergent problem in hospitals and communities.

3.2 Wastewater network monitoring results

The waste waters and sampling locations were chosen so as to identify the sources of antibiotic-resistant bacteria and their potential abatement in a waste water treatment plant. Another key point is the distinction between clinical and environmental isolates in the different samples.

Table 2 reports the types of strains identified in each sampling location. Looking at the results reported in Table 2, some apparent inconsistencies can be noticed: Klebsiella pneumoniae that has been detected in SL5 (downstream the chlorination treatment) and in SL7, has not been detected in the sampling locations upstream the chlorination treatment. This is due to the difficulty in isolating the different colonies when the total bacteria count is very high, as it happens upstream the disinfection section.

On the other hand, a significant results is that the strains found in SL 5, i.e. after the tertiary treatment of chlorination, are the minimum number of strains found all over the network investigated; this shows the effectiveness of the chlorination. Anyway, even after chlorination, some clinical isolates, such as Criseobacterium meningosepticum also found in SL1, are still present. The incomplete abatement of antibiotic resistant bacteria, especially when they are represented by clinical isolates, is obviously a potential risk for human health.

Also a sample of sludge from the waste water treatment plant was analyzed and antibiotic resistant bacteria were found. As often sludge is used for compost production, attention should be paid to this aspect to avoid contamination of food chain.

Table 2: Strains identified in each sampling location and number of isolates.

SL	STRAIN	SL	STRAIN
1	Pseudomonas pseudomallei: 2	4	Citrobacter freundii: 1
	Enterobacter agglomerans: 3		Enterobacter spp.: 2
	Pseudomonas putida: 2		Stenotrophomonas maltophilia: 3
	Stenotrophomonas maltophilia: 3		Klebsiella oxytoca: 1
	Criseobacterium meningosepticum/indologenes: 1		A.idroph./vibrio fluvialis: 2
	Areomonas hydrophila: 4	5	E.coli: 1
	Citrobacter freundii: 2		Klebsiella pneumoniae: 1
	Klebsiella oxytoca: 2		Criseobacterium meningosepticum: 1
2	Areomonas hydrophila: 2	6	Areomonas hydrophila: 3
	Pseudomonas spp.: 3		Klebsiella oxytoca: 1
	Enterobacter agglomerans: 2	7	Klebsiella pneumoniae: 1
	Stenotrophomonas maltophilia: 4		Areomonas hydrophila: 2
	Kluyvera spp: 1		Erwinia spp.: 1
	Vibro fluvialis: 2		E. coli: 2
3	A.idroph./vibrio fluvialis: 1		
	Vibrio vulnificus: 1		
	Enterobacter cloacae: 2		
	Areomonas hydrophila: 3	Dried sludge	Areomonas hydrophila: 2
	Crisiobacterium meningosepticum: 1		Enterobacter sakazakii: 1
	Areomonas spp: 4		
	Acinetobacter spp.: 2		
	Citrobacter freundii: 1		
	Stenotrophomonas maltophilia: 2		
	Salmonella spp: 1		

4 Conclusions

It has been evidenced that even in domestic waste waters various antibiotic resistant bacteria are present in relevant concentration. It has been shown that chlorination and UV have different effectiveness on the abatement of different antibiotic resistant bacteria strains, as they show different sensitivity to chlorination and UV, the latter giving the best results.

The preliminary results obtained in the wastewater network monitoring have confirmed the presence of antibiotic resistant bacteria in urban waste waters. In addition waste waters from a municipal hospital and mixed waste waters of different origin have been characterized, finding both clinical and environmental isolates.

Even if the enforced law does not consider the presence of antibiotic resistant bacteria in defining the parameters for water reuse, they are a key factor that should be taken into account for water quality assessment.

References

[1] Caputi, P., Del Re, G., Di Donato, A., Petrongolo, A., Volpe R., *Urban wastewater re-use in the Abruzzo region*, WIT Transactions on Ecology and the Environment, Vol. 80, WIT Press, Conference Proceedings, pp. 455-464, 2005.

[2] Bonomo, L., Nurizzo, C., Rolle, E., *Advanced wastewater treatment and reuse: related problems and perspectives in Italy*, Wat. Sci. Tech., Vol.40. No. 4-5, pp. 21-28, 1999.

[3] Liberti, L., Notarnicola, M., *Advanced treatment and disinfection for municipal wastewater reuse in agriculture*, Wat. Sci. Tech. Vol.40, No. 4-5, pp.235-245, 1999.

[4] Urkiaga, A., *Best available technologies for water reuse and recycling: needed steps to obtain the general implementation of water reuse*, 9th European Roundtable on Sustainable Consumption and Production (ERSCP), Bilbao, Spain, 2004.

[5] Dell'Erba, A., Falsanisi, D., Liberti, L., Notarnicola, M., Santoro, D., *Disinfecting behaviour of peracetic acid for municipal wastewater reuse*, Desalination, Vol. 168, pp. 435-442, 2004.

[6] Nurizzo, C., Antonelli, M., Profaizer, M., Romele, L., *By-products in surface and reclaimed water disinfected with various agents*, Desalination, Vol. 176, pp. 241-253, 2005.

[7] Baldry, M.G.C., Fraser, J.A.L., *Disinfection with peroxygens*, Critical Reports in Applied Chemistry, K.R. Payne ed., Vol.23, John Wiley & Sons, Chichester, pp 91-116, 1988.

[8] Bixio, D., Thoeye, C., De Koning, J., Joksimovic, D., Savic, D., Wintgens, T., Melin, T., *Wastewater reuse in Europe*, Desalination, Vol. 187, pp. 89–101, 2006.

[9] Asano, T., Cotruvo, J.A., *Groundwater recharge with reclaimed municipal wastewater: health and regulatory considerations*, Water Research, Vol. 38, pp. 1941-1951, 2004.

[10] Nurizzo, C., *Reclaimed water reuse in the Mediterranean region: some considerations on water resources, standard and bacterial re-growth phenomena*, Wat. Sci. Tech.: Water Supply, Vol.3, No.4, pp. 317-324, 2003.

[11] Nwachuku, N., Gerba, C.P., *Microbial risk assessment: don't forget the children*, Current Opinion in Microbiology, Vol.7, pp. 206-209, 2004.

[12] Reinthaler, F.F., Posch, J., Feierl, G., Wüst, G., Haas, D., Ruckenbauer, G., Mascher, F., Marth, E., *Antibiotic resistance of E.Coli in sewage and sludge*, Water Research, Vol. 37, pp.1685-1690, 2003.

[13] Brookes, J. D., Antenucci, J., Hipsey, M., Burch, M. D., Ashbolt, M. J., Ferguson, C., *Fate and transport of pathogens in lakes and reservoirs*, Environment International, Vol. 30, pp. 741-759, 2004.

[14] Koivunen, J., Siitonen, A., Heinonen-Tanski, H., *Elimination of enteric bacteria in biological-chemical wastewater treatment and tertiary filtration units*, Water Research, Vol. 37, pp. 690-698, 2003.

www.witpress.com, ISSN 1743-3541 (on-line)

[15] Koren, H. S., Crawford-Brown, T., *A framework for the integration of ecosystem and human health in public policy: two case studies with infectious agents*, Environmental Research, Vol. 95, pp. 92-105, 2004.
[16] Kümmerer, K., *Drugs in the environment: emission of drugs, diagnostic aids and disinfectants into wastewater by hospitals in relation to other sources – a review*, Chemosphere, Vol. 45, pp. 957-969, 2001.
[17] Kümmerer, K., Alexy, R., Hüttig, J., Schöll, A., *Standardized tests fail to assess the effects of antibiotics on environmental bacteria*, Water Research, Vol. 38, pp. 2111-2116, 2004.
[18] Reinthaler, F.F., Posch, J., Feierl, G., Wüst, G., Haas, D., Ruckenbauer, G., Mascher, F., Marth, E., *Antibiotic resistance of E.Coli in sewage and sludge*, Water Research, Vol. 37, pp.1685-1690, 2003.
[19] Sanderson, H., Johnson, D. J., Reitsma, T., Brain, R. A., Wilson, C. J., Solomon, K. R., *Ranking and prioritization of environmental risks of pharmaceuticals in surface waters*, Regulatory Toxicology and Pharmacology, Vol. 39, pp. 158-183, 2004.
[20] Del Re, G., Di Donato, A., Volpe, R. and Pellegrini, C., *Urban wastewater reuse: a pilot plant experience*, Int. J. Environmental Technology and Management, in press.

Removal of organic compounds from waste streams: a combined approach

J. O'Brien, T. Curtin & T. F. O'Dwyer
Chemical and Environmental Sciences Department,
Materials and Surface Science Institute, University of Limerick, Ireland

Abstract

A combination of factors, including costs associated with wastewater treatment and more restrictive and stringent controls on industrial effluent pollution have increased interest in water usage reduction, recycling and re-use opportunities. In this study a novel two-step approach to treatment of low concentrations of organic compounds in waste streams was investigated. Zeolite ZSM-5 was investigated with a view to examining its potential to function as both adsorbent and catalyst in the removal of aniline from aqueous solutions. The H-ZSM-5 zeolite was first loaded with copper before the aniline adsorption step. The aqueous stability of the exchanged copper on the zeolite was assessed as a function of solution pH. Minimum copper leaching was observed in the range pH 5 to pH 11 thus providing a stable working pH range for the 1.4Cu-ZSM-5 to function as an adsorbent material. Aniline adsorption was then carried out and an uptake level of 40 mg g^{-1} aniline on the 1.4Cu-ZSM-5 zeolite was achieved. The adsorption process followed the Langmuir model and aniline uptake was largely uninfluenced by temperature. The kinetics of the adsorption process indicated that maximum uptake of aniline occurred within thirty minutes. In the second step of the process, the aniline loaded Cu-ZSM-5 adsorbent samples were subjected to a catalytic oxidation process and mass spectroscopic analysis of the catalytic reactor exhaust gases suggested clear oxidation of the aniline to carbon dioxide, water and nitrogen, with minor amounts of nitrogen oxide gases being produced.
Keywords: organics, ZSM-5, adsorption, catalytic oxidation, waste stream treatment.

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1 Introduction

Water plays an integral role in both domestic and industrial life. The UN estimates that by the middle of this century, at best, 2 billion people in forty eight countries will experience water scarcity, i.e. their annual fresh water resources will be less than 1000m^3 per capita per year. At worst, 7 billion people in sixty countries will experience severe water shortage [1]. In industry, new, innovative and cost-effective technologies are necessary to develop the recycling and reuse potential of industrial process waste streams. Examples of industries with significant water usage and potential for treatment and recycling include the pharmaceutical, refining, food and beverage, chemical, petrochemical, pulp and paper and electronics sectors [2]. In general the reduction of wastewater in any given industry can be achieved in a number of ways including process modification, changes in raw materials used or water consumption reduction programmes [3, 4]. In practice, significant waste stream volumes are produced and where feasible the recycling and regeneration of these streams is of significant environmental and economic benefit.

A number of well-established technologies exist for waste stream treatment and recycling, including reverse osmosis (RO), micro filtration (MF) and ultra filtration (UF) [2, 5]. Alternatives for treating waste streams containing organic type impurities include specific oxidation methods which can be divided into the following categories: chemical oxidation and air/oxygen based catalytic (or non-catalytic) oxidation. The former category includes advanced oxidation processes (AOPs). Typical AOPs use ozone, hydrogen peroxide and UV radiation to generate hydroxyl radicals used for oxidation [6]. However, these processes tend to be limited by their intensive costs. The latter category includes dry oxidation, wet air oxidation and catalytic wet air oxidation.

Currently, there is a strong interest in developing alternative adsorbents for the removal of organic pollutants from aqueous waste streams. Zeolites are one type of alternative adsorbent material and have the ability to selectively adsorb or reject molecules based upon molecular size, shape and other properties including polarity. To date, limited studies have been carried out on zeolite adsorption of organic molecules from aqueous solutions [7–9]. Zeolites also have the ability to function as catalysts. In this work ZSM-5 zeolite is loaded with copper to promote and enhance its catalytic activity. Copper exchanged zeolites have been shown to be extremely active for the catalytic oxidation of ammonia, to nitrogen and water with low levels of NO and N_2O formed [10]. Centi and Perathoner [11] studied the adsorption and subsequent wet and dry oxidation of low levels of sodium gluconate and triethanolamine onto a series of mixed oxides. Our present work sets out to build on this knowledge.

The aqueous contaminant of interest (aniline) is chosen as the test compound as it is a nitrogen-containing hydrocarbon typical of compounds occurring in waste streams from dye, pigments, pharmaceuticals, rubber additives and pesticides manufacture. It will firstly be removed by adsorption onto a selective adsorbent Cu-ZSM-5 zeolite and, secondly, oxidation of the adsorbed pollutant into carbon dioxide, water and nitrogen with the simultaneous regeneration of the adsorbent/catalyst will be carried out (figure 1).

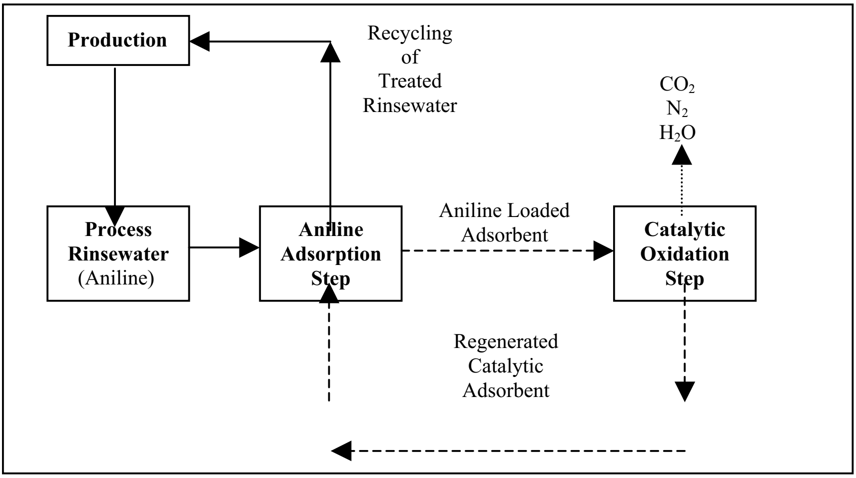

Figure 1: Adsorption/catalytic process schematic for treating and recycling waste streams.

2 Methodology

2.1 Materials and zeolite characterisation

Copper exchanged ZSM-5 zeolites were prepared using a one step procedure similar to that reported by Iwamoto et al. [12]. Briefly, the appropriate amount of copper nitrate trihydrate (Aldrich) was dissolved in water and the zeolite (SiO_2:Al_2O_3 = 80:1) added to the aqueous salt solution. The suspension was stirred for 24 hours at room temperature and adjusted to pH 7.0 with a NH_4OH solution. The sample was filtered and dried at 80°C for 10 hours, calcined at 450°C for 5 hours and sieved to a particle size range of 212 - 850μm. A 1.4Cu-ZSM-5 indicates a zeolite containing 1.4wt% copper (measured by AA) on the ZSM-5 zeolite support prepared by ion exchange. Typically, 0.1g of the copper exchanged zeolite was dissolved in 3 mls of hydrofluoric acid and diluted for AAS analysis. The samples were characterized by nitrogen gas adsorption/desorption isotherms using a Micromeritics Gemini ASAP 2010 system. Samples were pretreated at 150°C for 17 hours and zeolite surface areas were measured using the Brunauer–Emmett–Teller (BET) method [13]. The zeolites were also characterised by X-ray diffraction using a Philips X'pert PRO MPD (multi purpose diffractometer) X-ray diffractometer PW3050/60 θ–θ with a scan range of 5-60° (2θ) using nickel filtered Cu Kα radiation (λ = 1.542Å) at 40kV with a current of 35mA.

2.2 Copper leachability testing

The influence of aqueous solution pH on the possible leaching of the copper exchanged onto the ZSM-5 support was evaluated for solutions with initial pH between pH 1 and pH 11. Specifically, the pH of 150ml distilled water was adjusted appropriately using either HCl or ammonia solution (35% NH_3) and 0.5g of the copper exchanged adsorbent material was added to the solution. The solution was stirred continuously and samples were periodically withdrawn, vacuum filtered, centrifuged at 3,500 rpm and analysed by AAS.

2.3 Adsorption isotherms

An adsorption isotherm was prepared for aniline adsorption onto the 1.4Cu-ZSM-5 at 6°C and 24°C using a batch technique. The adsorbent, (0.1g), was allowed to reach equilibrium with aniline solutions (10ml) of known concentrations (50 and 4000 mg dm^{-3}). The solutions were stirred for one hour. The contents of each adsorption flask were then separated by centrifugation at 3,500 rpm for 20 min and filtered under vacuum. Pre and post-adsorption concentrations of aniline were determined by a Cary UV-Visible spectrometer (λ_{max} 230nm) and the amount adsorbed was calculated by the difference.

2.4 Catalytic oxidation experiments

The aniline loaded adsorbent/catalyst (100 mg) was placed in the quartz reactor and held in place by quartz wool plugs (figure 2). Before testing, the spent adsorbent sample was pre-treated for 60 minutes in a stream of a 3% oxygen-helium mixture at a flow rate of 50ml/min. The sample was then subjected to a temperature increase of 10°C/min up to 750°C. Products leaving the reactor in the exhaust gases were continuously monitored using the Mass Selective Detector (MSD).

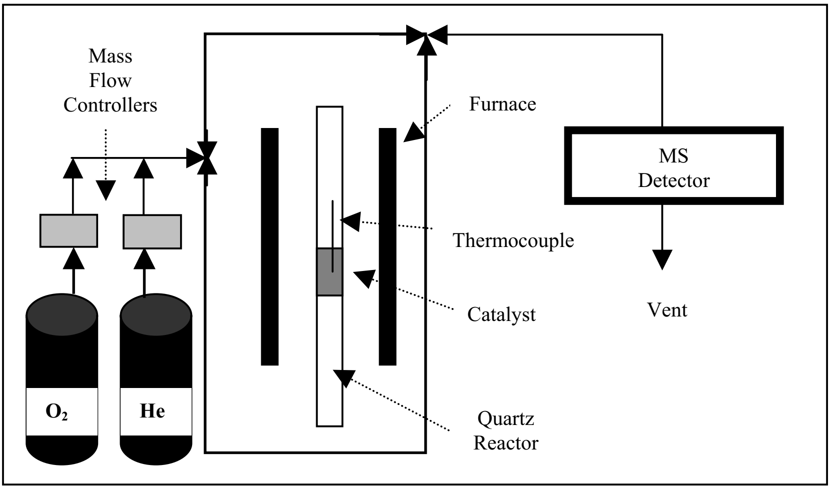

Figure 2: Catalytic oxidation setup.

3 Results and discussion

3.1 Characterization

In this study, the actual copper loading on the zeolite was 1.4wt%, which corresponded to an ion-exchange level of 108%. This exchange level was based on the assumption that a single Cu^{2+} charge balanced a pair of negatively charged aluminium sites on the zeolite framework. The X-ray diffraction patterns of the copper-exchanged zeolite was identical to that of the parent zeolite indicating good dispersion of the copper on the catalyst and the absence of any crystalline copper phase. The surface area of the copper-exchanged ZSM-5 zeolite adsorbent was estimated to be 223 m^2g^{-1} as compared to 275 m^2g^{-1} for the unmodified H-ZSM-5.

3.2 Stability of the copper loaded ZSM-5 zeolite

Leaching tests were undertaken on the 1.4Cu-ZSM-5 sample in order to assess the extent of copper leaching as a function of aqueous pH. The results of this study are summarized in Table 1. From these results it is clear that the 1.4Cu-ZSM-5 material was most stable over the range pH 5 to pH 9, where the leached copper levels were less than 1 mg dm^{-3}. Complete copper leaching was calculated to give a resultant aqueous concentration of 46 mg dm^{-3}. At the acidic pH's copper leaching was rapid and in all cases maximum leaching occurred in less than 30 minutes. A study into successive copper leaching on the 1.4Cu-ZSM-5 samples showed that no subsequent leaching occurs after initial leaching. Therefore, the 1.4Cu-ZSM-5 zeolite can be used as a stable adsorbent material in the aqueous phase for the purposes of this work.

Table 1: Leaching tests on 1.4Cu-ZSM-5 (pH range 1 – 11).

	1.4Cu-ZSM-5					
pH	1	3	5	7	9	11
Copper Leached (%)	80	21.4	0.3	<0.1	<0.1	1.5
Aqueous Cu^{2+} (mg dm^{-3})	37	9.8	0.14	<0.1	<0.1	0.7
Maximum Leaching (mins)	<20	<20	N/A	N/A	N/A	<20

3.3 Adsorption capacity of 1.4Cu-ZSM-5 for aniline

A preliminary test to establish the time to reach maximum adsorption revealed maximum uptake of aniline within 30-40 minutes of contact with the adsorbent material. The adsorption isotherms for aniline on 1.4Cu-ZSM-5 at 6°C and 24°C are displayed in Figure 3. All samples show maximum aniline adsorption of between 37 - 42 mg g^{-1} suggesting that at this level the zeolite is saturated. This significant level of uptake indicates that the pore diameters of the ZSM-5 zeolite facilitate removal of significant quantities of aniline from an aqueous environment. The level of aniline adsorbed onto the zeolite support was largely unaffected by a change in temperature.

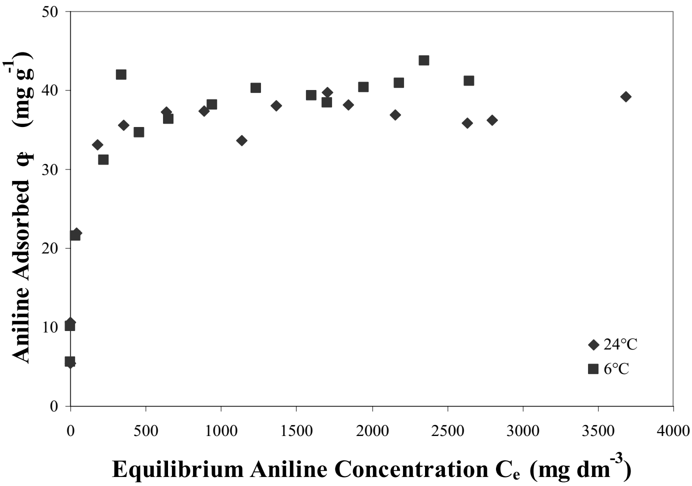

Figure 3: Adsorption isotherm for aniline on 1.4Cu-ZSM-5 (■ 6°C, ♦ 24°C).

Table 2: Langmuir constants for aniline adsorption on 1.4 Cu-ZSM-5 zeolite at 6°C and 24°C.

		1.4Cu-ZSM-5	Langmuir Constants	
Temp (^{o}C)	K_L ($dm^3 g^{-1}$)	A_L ($dm^3 mg^{-1}$)	K_L/ A_L ($mg\ g^{-1}$)	R^2
6	1.44	0.037	38.9	0.997
24	1.47	0.035	41.8	0.997

Both the Langmuir [14] and Freundlich [15] models, as defined below in Equations (1) and (2), respectively, were applied to the adsorption data.

$$q_e = \frac{K_L C_e}{1 + A_L C_e} \tag{1}$$

$$q_e = K_F C^{1/n} \tag{2}$$

where q_e is the amount of solute adsorbed per gram of adsorbent and K_L and A_L are Langmuir constants. A plot of C_e/q_e versus C_e from the linear form of Equation (1) was drawn to determine the values of K_L (intercept) and A_L/K_L (slope). Maximum uptake on the zeolite adsorbent surface was then obtained as K_L/A_L. The data in Table 2 indicate that the sorption of the aniline from aqueous solution onto the zeolite material showed good fit to the Langmuir model.

3.4 Kinetics and thermodynamics of aniline adsorption

Figure 3 suggests that temperature change from 6^{o}C to 24^{o}C has little impact on the aniline adsorption level. The results of the kinetic experiments undertaken on 1.4Cu-ZSM-5 are represented in Table 3.

Table 3: Kinetic data displaying the equilibrium time required for aniline adsorption at 24°C onto 1.4Cu-ZSM-5.

Initial Aniline Concentration (mg dm^{-3})	Maximum Adsorption Level (mg g^{-1})	Time to Maximum Adsorption (Minutes)
125	11	30
250	23	30
500	32	30

The findings revealed that equilibrium aniline uptake generally occurred within approximately 30 minutes of contact time irrespective of the initial aniline concentration used in the kinetic study.

3.5 Catalytic oxidation of adsorbed aniline

Figure 4 illustrates the desorption profile for aniline over H-ZSM-5 and 1.4Cu-ZSM-5 during catalytic oxidation. The H-ZSM-5 sample was used to assess the influence of copper loading on aniline oxidation. In the case of the H-Beta zeolite, unreacted aniline is clearly visible in the exhaust gases from the reactor. The presence of copper on the zeolite (1.4Cu-ZSM-5) considerably reduces the concentration of aniline in the exhaust gases indicating strong oxidation of the adsorbed aniline to breakdown products.

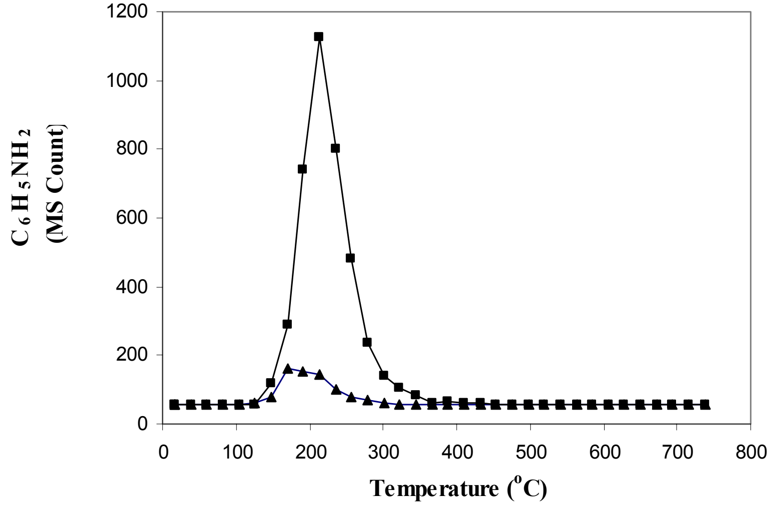

Figure 4: Comparison of the desorption of aniline on the ZSM-5 zeolites (100mg of zeolite sample containing 10.5mg aniline, 3% O_2/He gas flow, to 750°C at 10°C/min) (■ H-ZSM-5, ▲ 1.4Cu-ZSM-5).

The degradation of aniline sorbed on 1.4Cu-ZSM-5 should be accompanied by a corresponding increase in carbon dioxide level in the exhaust gases from the reactor as evidenced in figure 5. While aniline desorbs from the zeolite bed at 200°C, subsequent conversion into carbon dioxide does not occur until temperatures higher than approximately 400°C. Significantly, the CO_2 profiles in figure 5 also show that the presence of copper loading on the 1.4Cu-ZSM-5 zeolite effectively reduces the temperature required for oxidation of the aniline as evidenced by the lower temperature emission of CO_2. Preliminary work has also shown similar increases in nitrogen and water formation in the exhaust gases. Further work on the reproducibility of the 1.4Cu-ZSM-5 catalytic activity for aniline degradation over successive cycles is currently being researched.

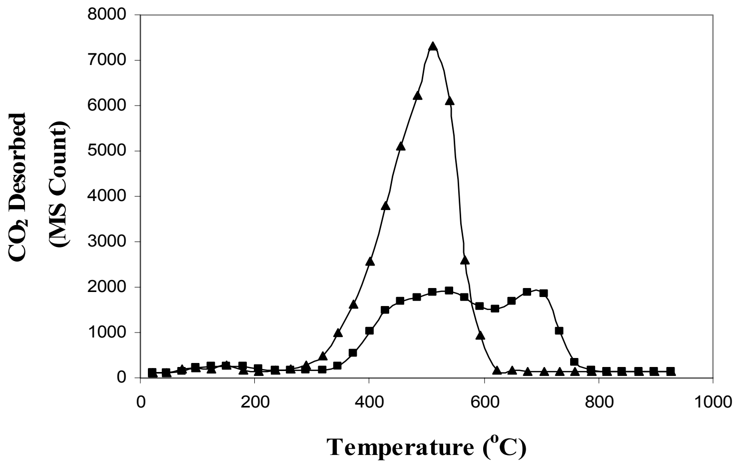

Figure 5: Comparison of the desorption of CO_2 on the ZSM-5 zeolites (100mg of zeolite sample containing 10.5mg aniline, 3% O_2/He gas flow, to 750°C at 10°C/min) (■ H-ZSM-5, ▲ 1.4Cu-ZSM-5).

4 Conclusion

A two-stage system was designed whereby initially the aniline was removed from solution by an adsorption step and subsequently the adsorbed aniline was catalytically oxidized to carbon dioxide, water and nitrogen with minor quantities of other gases. The 1.4Cu-ZSM-5 adsorption process (Step 1) has been shown to be effective with aniline uptake levels of 40 mg g^{-1} being achieved. The catalytic oxidation process (Step 2) has been shown to effectively destroy the aniline sorbed on 1.4Cu-ZSM-5. The system has shown considerable promise and further work is currently being carried out specifically to address a number of key issues including the suitability of the process for stripping other organic compounds, the potential selectivity towards specific organics, the mechanism by which the catalytic oxidation process occurs, potential de-activation of the

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1.4Cu-ZSM-5 adsorbent/catalyst, the emission of carbon dioxide and the physical development of a larger scale system.

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References

[1] Knighton, C.S., (Ed.), Water for People Water for Life - UN World Water Development Report, UNESCO Publishing, Paris, 2003

[2] Marcucci, M., Ciardelli, G., Matteucci, A., Ranieri, L. and Russo, M., Experimental Campaigns on Textile Wastewater for Reuse by Means of Different Membrane Processes, Desalination, 149, pp. 137-143, 2002.

[3] Hancock, F. E., Catalytic Strategies for Industrial Water Reuse, Catalysis Today, 53, pp. 3-9, 1999.

[4] Zbontar Zver, L. and Glavic, P., Water Minimization in Process Industries: Case Study in Beet Sugar Plant, Resources, Conservation and Recycling, 43, pp. 133 -145, 2005.

[5] Saha, N.K., Balakrishnan, M. and V.S. Batra, Improving Industrial Water Use: Case Study for an Indian Distillery, Resources, Conservation and Recycling, 43, pp. 163-174, 2005.

[6] Bastaki, A. and Nader, M., Performance of Advanced Methods for Treatment of Wastewater: UV/TiO2, RO and UF, Chemical Engineering and Processing, 43, pp. 935-940, 2004.

[7] Shu, H.T., Li, D., Scala, A.A. and Ma, Y.H., Adsorption of Small Organic Pollutants from Aqueous Streams by Aluminosilicate-Based Microporous Materials, Separation and Purification Technology, 11, pp. 27-36, 1997.

[8] Narita, E., Horiguchi, N. and Okabe, T., Adsorption of Phenols, Cresols and Benzyl Alcohol from Aqueous Solution by Silicalite, Chem. Lett., pp. 787, 1985.

[9] Khalid, M., Joly, G., Renaud, A. and Magnoux, P., Removal of Phenol from Water by Adsorption using Zeolites, Ind. Eng. Chem. Res. 43, pp. 5275, 2004.

[10] Lenihan, S. and Curtin, T., Copper Exchanged Beta Zeolites for the Catalytic Oxidation of Ammonia, Chem. Commun., pp. 1280-1281, 2003.

[11] Centi, G. and Perathoner, S., Recycle Rinse Water: Problems and Opportunities, Catalysis Today, 53, pp. 19-47, 1999.

[12] Iwamoto, M., Yahiro, H., Torikai, Y., Yoshioka, T. and Mizuno, N., Novel Preparation Method of Highly Copper Ion-Exchanged ZSM-5 Zeolites and their Catalytic Activities for NO Decomposition, Chemistry Letters, pp. 1967-1970, 1990.

www.witpress.com, ISSN 1743-3541 (on-line)

[13] Brunauer, S., Deming, L.S., Deming, W.E. and Teller, E., On A Theory of the Van der Waals Adsorption of Gases, J. Am. Chem. Soc., 62, pp. 1723-1732, 1940.
[14] Langmuir, I., The Adsorption of Gases on Plane Surfaces of Glass, Mica and Platinum, J. Am. Chem. Soc., 40, pp. 1361-1403, 1918.
[15] Freundlich, H.A., Uber die Adsorption in Losungen, Z. Phys. Chem., A 57, pp. 385, 1906.

A comparison between different advanced oxidation processes for the remediation of PCP contaminated wastewaters

R. Mecozzi & L. Di Palma
Department of Chemical Engineering,
University of Rome "La Sapienza", Italy

Abstract

In this paper a comparison between different advanced oxidation processes (AOP), for the remediation of a wastewater contaminated with 100 mg/l of pentachlorophenol (PCP), is proposed. The AOP chosen were: Fenton or Fenton-like reactions in which the soluble iron catalyst salt was added as Fe(II) or Fe(III) or a mixture of Fe(II) and Fe(III), and ozonation. In the latter case the effects of the presence of a solid iron oxide (hematite) were also evaluated. For the Fenton's reactions, the investigated parameters were: pH (from pH=1.5 to pH=5), reaction time, H_2O_2/PCP ratio (from 5/1 to 20/1), Fe/H_2O_2 ratio (from 0 to 2/1) and the effect of a H_2O_2 stabilizer: KH_2PO_4. The highest TOC removal (75%) was reached, using stabilized H_2O_2 and Fe(II), when H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1; in this case, dechlorination was 96% and KH_2PO_4 increased H_2O_2 lifetime significantly. Fe(III) or the mixture of Fe(II) and Fe(III), proved to be more efficient than Fe(II) when H_2O_2/PCP<15/1 and Fe/H_2O_2<1/1. For ozonation the effects of pH and of the ratio between airflow and the solution's volume, were investigated. The highest efficiencies (TOC removal=35%, Cl^- release=80%) were reached when O_3=0.24 mg/l at pH=11; however diminishing the volume of treated solution, 47% TOC removal and stoichiometric Cl^- release were observed. The addition of hematite resulted in a slight increase of TOC removal and Cl^- release. By comparing the experimental results, among the tested AOP systems, Fenton's reaction proved to be the most effective for PCP remediation.
Keywords: Fenton, Fenton-like, PCP, AOP, ozone, iron oxide, hydrogen peroxide.

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1 Introduction

Advanced oxidation processes (AOP) can be considered as possible pre-treatment steps to either achieve complete mineralization of organic contaminants to CO_2 and H_2O, or transform them into more biodegradable intermediates. In effect, the toxicity of the pollutants can inhibit successive biological steps. One of the most commonly used oxidizing agents is hydrogen peroxide (H_2O_2), which acts as a source of free radicals when a metal catalyst is added. The effectiveness of H_2O_2 when in contact with a transition metal catalyst added as a soluble salt or as a solid, in Fenton or Fenton-like reactions, has been proven in the treatment of hazardous compounds in the aqueous phase [1–3]. In particular, when $FeSO_4$ is added to a H_2O_2 solution in Fenton systems, reactions that lead to the formation of hydroxyl radical ($OH^•$), and numerous other competing reactions occur. Among these reactions are the production of hydroperoxyl radicals, the cycling of Fe(III) to Fe(II) and the quenching of $OH^•$ by Fe(II) and H_2O_2 [4, 5].

Ozonation has been widely used for drinking water disinfection, and thanks to the high oxidation power of ozone, it can achieve high efficiencies avoiding the addition of other compounds. The degradation of organic compounds may proceed via direct reaction with O_3 or via free radicals formation such as $OH^•$, or via the both mechanisms concurrently [6]. The pH of the solution strongly influences ozone decomposition [7]. In particular at $pH<4$, dissolved O_3 reacts directly with the substrate, while at $pH>9$ the $OH^•$ formation, which is catalyzed by OH^- prevails, and at $4<pH<9$ both the mechanisms are present [8]. Ozone's oxidation efficiency may also be enhanced through the addition of transition metals cations [7, 8]. In this work air was utilized for ozone production thus limiting the cost of oxygen supply, although introducing CO_2.

This paper presents a comparison between different AOP for the treatment of a wastewater contaminated with 100 mg/l of pentachlorophenol (PCP). PCP is a preservative agent for woods, vegetable fibres, leathers and a pesticide. It is listed by the US EPA as a priority pollutant because it is toxic, hardly biodegradable and highly persistent in the environment. It was therefore chosen as a model compound because it is very difficult to oxidize and the available literature concerning water treatments generally focuses on other chlorophenols or really dissimilar concentrations, making it very difficult to compare the different treatments.

2 Experimental

PCP (99% pure) was obtained from Aldrich. The contaminated solution was prepared by dissolving 100 mg of PCP in 1l of distilled water and alkalinising to pH=11 with NaOH to help PCP dissolution. H_2O_2 (30% w/w unstabilized) was from Merck, Fe(II) and Fe(III) sulphate ACS, were from Carlo Erba Reagenti. Total organic carbon (TOC) was monitored using a ShimadzuTOC-5000 analyser after alkalinising the samples with NaOH to ensure complete dissolution of residual PCP, Cl^- release was determined by ionic chromatography using a DionexDX120 ion chromatograph equipped with a IONPAC AS 12A column (20 cm, 4 mm i.d.) and guard column (5cm, 4mm i.d.).

Fenton's reactions were conducted using Fe(II), Fe(III) or a mixture 50% w/w of the two salts, on 50 ml samples in 100 ml beakers covered with parafilm. The Fe catalyst was added first, and the pH was adjusted with H_2SO_4. H_2O_2 was then added to the solution and the samples were stirred at 750 rpm with a magnetic stirrer. The pH was manually monitored and kept constant. A first series of tests was performed to investigate pH, using Fe(II) as a catalyst and the following reagents' ratios: H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1. At the optimised pH value, other experiments were performed using H_2O_2/PCP=5/1, 10/1 and 15/1, for different Fe/H_2O_2 ratios (1/10, 1/5, 1/1, 2/1). A further series of tests was realized to evaluate the influence of the addition of 16 g/l KH_2PO_4 to the 30% stock H_2O_2 solution, using Fe(II) as a catalyst. Initial tests were performed to investigate the optimal pH, for the following reagents' ratios: H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1. At the optimised pH value, other experiments were performed using H_2O_2/PCP=5/1, 10/1 and 15/1, for different Fe/H_2O_2 ratios (1/3, 1/2, 1/1, 2/1). Dissolved oxygen was monitored with an Aqualytic OX 22 O_2-meter. Hydrogen peroxide was determined by a colorimetric method using a Merk RQ-flex reflectometer.

Ozonation experiments were conducted on 1l samples in a covered glass reactor. Mixing was provided by a magnetic stirring bar. Ozone was produced using an ozone generator INO3MAX-04 functioning with air, and was bubbled in the solution trough a porous media. The gas flow was 3.5l per minute and the O_3 concentration in the gas was 0.24 mg/l. The pH of the solution was manually controlled, and kept constant or not. The investigated pH values were pH=11, 9 and 7. Hematite when added, weighed 0.5g. Other experiments were conducted on 75 ml of solution with the same flux and ozone concentration.

3 Results and discussion

3.1 Fenton's reaction

3.1.1 Influence of pH

The optimal pH range for Fenton's reaction is between pH=2 and pH=4. In effect, at low pH, the solubility of the Fe(II) increases and H_2O_2 decomposition decreases [9]. Results presented in Figure 1. indicated pH=2.5 as the best operating pH for TOC removal and Cl^- release. The reduced efficiency observed at pH<2.5 was probably due to the concurrent effects of: the quenching of $OH^\bullet$ by the excess of H^+, the presence of PCP as an insoluble particulate [10, 11], and the transformation of H_2O_2 in the $H_3O_2^+$ ion, which is electrophillic and therefore does not react well with Fe(II) [12]. At higher pH, Fe(II) may be present in a colloidal form less reactive with H_2O_2 and in addition, the oxidation potential of $OH^\bullet$ decreases [12]. In experiments whose pH was not controlled, we observed a decrease of its value, from pH=7 to pH=3 and from pH=2.5 to pH=1.9, which could be associated with the formation of acid species that are usual in oxidation treatments [8, 13, 14].

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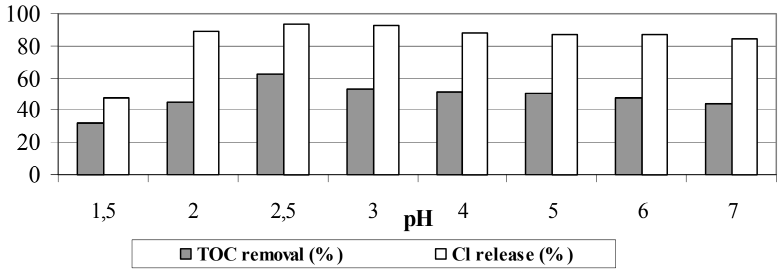

Figure 1: Effect of pH variation on TOC removal and chloride release.

3.1.2 Catalyst evaluation

Once pH value was fixed at pH=2.5, the three types of iron catalyst were compared in terms of TOC removal and chloride release (Figure 2).

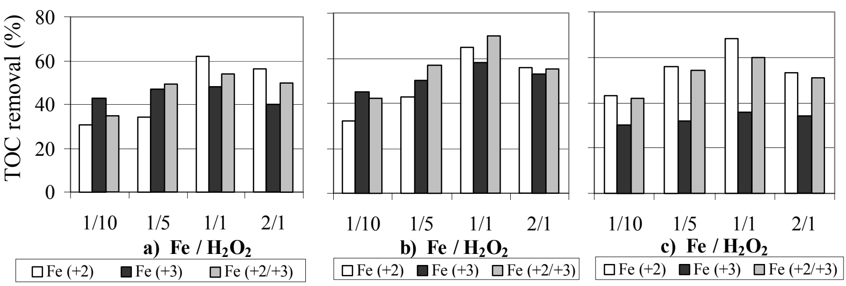

Figure 2: TOC removal: a) H_2O_2/PCP=5/1, b) H_2O_2/PCP=10/1, c) H_2O_2/PCP=15/1.

When no iron was added, no TOC removal was observed. When H_2O_2/PCP<15/1 and Fe/H_2O_2<1/1, Fe(III) or the mixture of the two irons proved to be more efficient than Fe(II). Increasing the H_2O_2/PCP ratio up to 15/1 augmented TOC removal when Fe(II) was used. Further experiments, conducted with H_2O_2/PCP=20/1, showed indeed a significant efficiency decrease (data not shown). This could be attributed to the quenching of radical species operated by H_2O_2. An increase in the Fe/H_2O_2 ratio up to 1/1, showed similar behaviour for the three Fe species: the maximum removal efficiency was reached when Fe/H_2O_2=1/1, and a further addition of iron (Fe/H_2O_2=2/1) resulted inefficient probably because undesired reactions, between Fe(II) and the radical species, became predominant. When H_2O_2/PCP=15/1, the dosage of Fe(III) appeared to be irrelevant leading to quasi-similar TOC removals and, in general, it was not really affected by the variation of experimental conditions. The performance of the mixture of Fe(II) and Fe(III) was more comparable to the one of Fe(II).

To explain these results some considerations must be made. First Fe(III) is less soluble than Fe(II) and possibly takes part mostly as a precipitate to the reaction, explaining the little differences between the removal efficiencies. Then, PCP is not completely soluble at acidic pH and, therefore, takes also part as a particulate to the reaction. Problems related to the difficulties in the radical attack on PCP particulates have been discussed [10]. Partitioning might segregate PCP from the $OH^{\bullet}$ that is presumably generated in the aqueous phase [15]. Since PCP degradation products are more soluble than the parent compound, they could be oxidized more easily as soon as they were formed [16]. In addition, the soluble PCP undergoes acid-base equilibrium with pK_a=4.7 [6]. At pH=2.5 it is mostly present in the undissociated form and therefore, it is less easily attacked by the $OH^{\bullet}$ radical. $OH^{\bullet}$ is a strong electrophilic compound, which reacts rapidly with alkenes and aromatic compounds in a non-selective way. Some authors [17] evidenced, as possible reaction pathway of PCP with $OH^{\bullet}$, an ipso-addition followed by an electron transfer reaction. However steric hindrance must be taken into account and, as the 5 electron withdrawing chlorine atoms decrease the electron density of the aromatic ring and deactivate it [6, 16], the radical attack could also be of nucleophilic nature. A study [18], has evidenced the reactivity of the superoxide radical anion $O_2^{\bullet-}$, which is a strong nucleophile. The simultaneous presence of $OH^{\bullet}$, $O_2^{\bullet-}$,$HO_2^{\bullet}$ and HO_2^- could then provide a pool of oxidants of enhanced reactivity capable to partially mineralize PCP and to achieve nearly complete dechlorination.

Figure 3 presents the results of the Cl^- release analyses. When no iron was added, little Cl^- release was observed (max 10% when H_2O_2/PCP=15/1). Between the Cl^- release and the previously illustrated TOC removal data, a considerable difference exists. In fact, almost independently from the Fe/H_2O_2 ratio and slightly from the H_2O_2/PCP ratio, nearly complete dechlorinations were always achieved. A trend similar to the one of TOC was observed only when Fe(III) was used and H_2O_2/PCP=15/1.

The maximum release efficiency was reached when Fe/H_2O_2=1/1 and a further addition of Fe (Fe/H_2O_2=2/1) resulted inefficient, as previously observed.

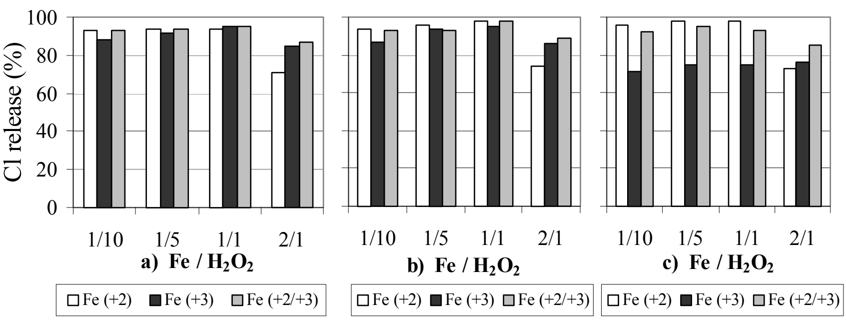

Figure 3: Cl release: a) H_2O_2/PCP=5/1, b) H_2O_2/PCP=10/1, c) H_2O_2/PCP=15/1.

Considering that the removal of six atoms of organic C must be accompanied by removal of five atoms of Cl, we calculated the MCR (Minimum Chloride Release) [19]. When the Cl^- release of the treated solution is approaching MCR value, we can assume that a certain selectivity of the reaction did occur. The comparison between the Cl^- release and the MCR value, showed a decrease of their difference with increasing Fe/H_2O_2 ratios, when Fe(II) or Fe(II)/Fe(III) were used, indicating a more selective reaction pathway. A little decrease was also observed with increasing H_2O_2/PCP ratios. The results obtained using Fe(III) were only a little affected by the variation of these parameters, as previously discussed. It can be concluded that for low Fe dosages, the reactions were slightly selective while, for high dosages, reactions proved to be highly selective especially in the case of Fe(II). The abundance of Fe(II) in effect, may enhance reactions that end with the formation of reactive species other than $OH^{\bullet}$.

3.1.3 Influence of KH_2PO_4

Results of tests performed to evaluate the influence of pH, are presented in Figure 4a). They indicated pH=2.5 as the best operating pH for TOC removal and Cl^- release, with marked differences between the reactions conducted with and without stabilizer (compare with Figure 1.)

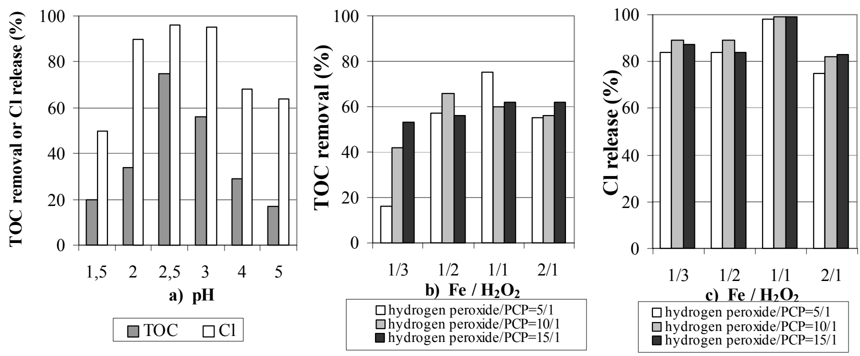

Figure 4: a) pH variation H_2O_2 /PCP=5/1 b) TOC removal %, c) Cl release %.

The observed difference may be explained by the combined effects of the lower solubility of Fe(II) and of this decreased availability. The KH_2PO_4 stabilizer lowers dissolved metal concentrations through either precipitation reactions or, in the presence of excess phosphate, conversion to relatively stable complexes [9]. As a result Fe(II) activity as Fenton catalyst decreases as it becomes less reactive with H_2O_2. Phosphate may also function as a radical scavenger quenching $OH^{\bullet}$ and finishing the chain decomposition reactions [9]. The results, in terms of TOC removal and Cl^- release, of experiments conducted at pH=2.5 are presented in Figure 4b) and 4c). The best TOC removal was achieved when H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1. A comparison between data obtained in the same conditions without stabilizer, showed that KH_2PO_4 enhanced to some extent the TOC removal efficiency only when H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1. In this particular condition, also Cl^- release was a little enhanced. For higher Fe concentrations, although the presence of KH_2PO_4 did not produce any modification in TOC removal, it augmented the Cl release. In addition, for H_2O_2/PCP=5/1 and Fe/H_2O_2>1/3, the stabilizer lowered the differences between the Cl^- release and the MRC value, indicating a different reaction pathway.

Analyses of H_2O_2 and dissolved oxygen concentrations, in the presence or not of the stabilizer when H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1, showed lower H_2O_2 depletion rates and a lower oxygen production when KH_2PO_4 was used (data not shown). In Fenton's systems, oxygen is generated by undesired quenching reactions between H_2O_2 and the radical species, or between the radical species themselves [8]. In this case, the stabilizer proved to be effective in lowering H_2O_2 consumption and in increasing TOC removal.

3.2 Ozonation experiments

These experiments were conducted fluxing ozonized air. The pH value was consequently expected to decrease because of the introduction of CO_2 in the reaction vessel, and this was confirmed by our experimental data in which we observed a decrease from pH=9 down to pH=3. The formation of acid by-products [8, 13] was proven by the decrease of pH measured during the reaction of PCP solutions that was greater than the one of control experiments conducted without any substrate. In addition, during the experiments, a strong solution's coloration, which disappeared by the end of reaction, was observed. The colouration that was associated with the presence of quinone and ketone compounds [6], depended upon pH value and was pink at controlled pH=11 and 9, while at pH=7 and at initial pH=11 and 9, it was yellow.

As prevalent reaction pathway, Weavers et al. [20] reported an $OH^\bullet$ addition at the ortho and para position, while Hong and Zeng [6], the nucleophilic attack of dissolved ozone with PCP, which could be followed by a subsequent radical degradation of the reaction intermediates, and would be independent from the solution's pH.

3.2.1 Influence of pH control

Results of tests performed to evaluate the influence of the control of the pH value, are presented in Figure 5.

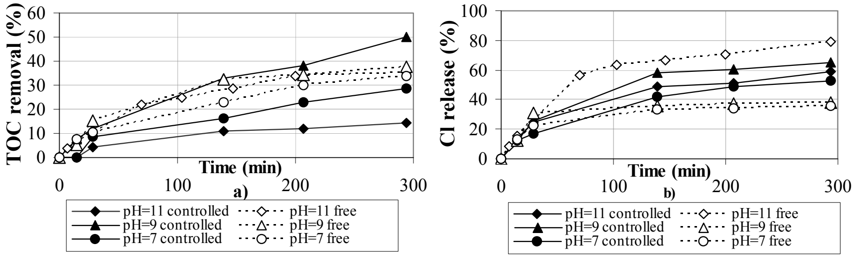

Figure 5: a) TOC removal (%), b) Cl release (%).

The highest TOC removal (50%) was achieved at the fixed pH=9 value, while when pH=11, the pH control caused a decrease of the removal efficiency. The experiments conducted without any pH adjustment, generally reached higher efficiencies. To explain the observed data, the presence of HCO_3^- and CO_3^{2-} ions in the solution must be considered. These ions are known to be $OH^\bullet$

scavengers [21] and were produced by the fluxing of CO_2 into the alkaline solution. The results obtained at pH=9 were consistent with other results [8] where the rate of oxidation of chlorophenol at pH=9 was related to the high rate of reaction between the phenolate ion and the dissolved ozone. At neutral or alkaline pHs, PCP was expected to exist in the deprotonated form that could react more easily with the oxidant species. The highest Cl^- release was observed at non-controlled pH=11, but generally the pH-controlled experiments, guaranteed greater dechlorinations. This could be explained by the higher $OH^\bullet$ formation rate in alkaline conditions. In effect, although PCP seems to react preferentially with O_3, its degradation products could be degraded by free radicals and the impact of different reaction's pH could become relevant [6].

Results of tests conducted fluxing the same gas in a littler reactor without hematite addition indicated, for all pHs after 90 min of reaction, higher TOC removals and Cl^- release (data not shown). Stoichiometric dechlorination was obtained at pH=11.

3.2.2 Influence of hematite

To increase TOC removal and Cl^- release, hematite was added as solid catalyst. This Fe oxide proved to be efficient in catalyzing the Fenton-like degradation of PCP [19]. Results presented in Figure 6 evidenced an increase of approx 15% TOC removal for the reactions conducted without pH control and, apart from the one at pH=9, also for the pH controlled ones.

Also the Cl^- release was increased, up to 74% at pH=11, and the same trend than in the absence of hematite was observed. The activity of metal oxides is mainly based on the catalytic decomposition of O_3 and the enhanced generation of $OH^\bullet$. These can be affected by changes in the catalyst surface properties due to different solution's pH. The proposed mechanism generally assumes that ozone is adsorbed on the catalyst's surface and subsequently leads to surface bond oxygen radicals or to $OH^\bullet$ [7]. In our case, owing to the low concentration of solid hematite used (0.5 g/l), we can assume that the heterogeneous catalysis occurred together with the homogeneous one.

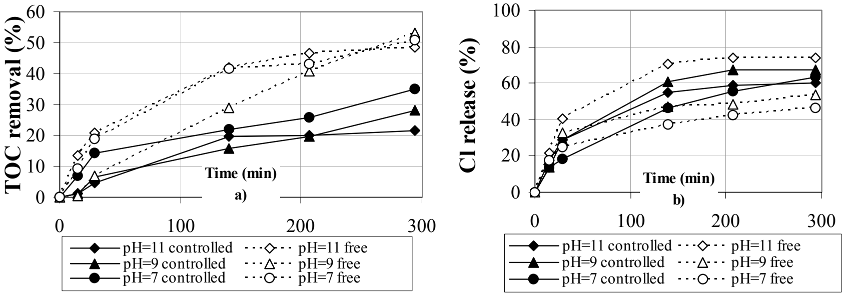

Figure 6: a) TOC removal (%), b) Cl release (%).

4 Conclusions

In this paper, a comparison between the Fenton's reaction catalyzed with Fe(II), Fe(III) or a mixture 50%(w/w) of the two species and ozonation was described.

Results indicated Fe (II) as the most efficient catalyst when high doses of hydrogen peroxide were employed. The highest TOC removal (75%) was reached using stabilized H_2O_2 when H_2O_2/PCP=5/1 and Fe/H_2O_2=1/1; dechlorination was 98% and KH_2PO_4 increased H_2O_2 lifetime significantly. Without stabilizer TOC removal reached 68% and Cl release 98% when H_2O_2/PCP=15/1 and Fe/H_2O_2=1/1.

Ozonation experiments resulted in lower TOC removals but in stoichiometric dechlorination at pH=11 in the littler reactor. These results demonstrate the difficulties in utilizing air as gas feed for the ozonator instead of oxygen, due to the excessive increase of the concentration of powerful hydroxyl radical scavengers. Hematite proved to increase the reaction's yield and seems promising due to its low cost.

References

[1] Lucking, F., Koser, H., Jank, M. & Ritter, A., Iron powder, graphite and activated carbon as catalysts for the oxidation of 4-chlorophenol with hydrogen peroxide. *Water Research*. **32(9)**, pp. 2607-2614, 1998.

[2] Gallard, H. & Laat, J. D., Kinetics of oxidation of chlorobenzenes and phenyl-ureas by Fe(II)/H2O2 and Fe(III)/H2O2. Evidence of reduction and oxidation reactions of intermediates by Fe(II) or Fe(III). *Chemosphere*. **42**, pp. 405-413, 2001.

[3] Teel, A. L., Warberg, C. R., Atkinson, D. A. & Watts, R. J., Comparison of mineral and soluble iron Fenton's catalysts for the treatment of trichloroethylene. *Water Research*. **35**, pp. 977-984, 2001.

[4] Ensing, B., Buda, F. & Baerends, E. J., Fenton-like chemistry in water: oxidation catalysis by Fe(III) and H2O2. *Journal of Physical Chemistry A*. **107**, pp. 5722-5731, 2003.

[5] Mohanty, N. R. & Wei, I. W., Oxidation of 2,4-dinitrotoluene using Fenton's reagent: reaction mechanisms and their practical applications. *Hazardous Waste Hazardous Materials*. **10**, pp. 171-183, 1993.

[6] Hong, P. K. A. & Zeng, Y., Degradation of pentachlorophenol by ozonation and biodegradability of intermediates. *Water Research*. **36**, pp. 4243-4254, 2002.

[7] Kasprzyk-Hordern, B., Ziolek, M. & Nawrocki, J., Catalytic ozonation and methods of enhancing molecular ozone reactions in water treatment. *Applied Catalysis B*. **46**, pp. 639-669, 2003.

[8] Pera-Titus, M., Garcìa-Molina, V., Banos, M. A., Giménez, J. & Esplugas, S., Degradation of chlorophenols by means of advanced oxidation processes: a general review. *Applied Catalysis B*. **47**, pp. 219-256, 2004.

www.witpress.com, ISSN 1743-3541 (on-line)

[9] Watts, R. J., Foget, M. K., Kong, S.-H. & Teel, A. L., Hydrogen peroxide decomposition in model subsurface systems. *Journal of Hazardous Materials*. **B69**, pp. 229-243, 1999.
[10] Watts, R. J., Udell, M. D. & Monsen, R. M., Use of iron minerals in optimizing the peroxide treatment of contaminated soils. *Water Environment Research*. **65**, pp. 839-844, 1993.
[11] Fukushima, M. & Tatsumi, K., Degradation pathways of pentachlorophenol by photo-Fenton systems in the presence of iron(III), humic acid, and hydrogen peroxide. *Environmental Science Technology*. **35**, pp. 1771-1778, 2001.
[12] Kwon, B. G., Lee, D. S., Kang, N. & Yoon, J., Characteristics of p-chlorophenol oxidation by Fenton's reagent. *Water Research*. **33**, pp. 2110-2118, 1999.
[13] Chen, G., Hoag, G. E., Chedda, P., Nadim, F., Woody, B. A. & Dobbs, G. M., The mechanism and applicability of in situ oxidation of trichloroethylene with Fenton's reagent. *Journal of Hazardous Materials*. **B87**, pp. 171-186, 2001.
[14] Hirvonen, A., Trapido, M., Hentunen, J. & Tarhanen, J., Formation of hydroxilated and dimeric intermediates during oxidation of chlorinated phenols in aqueous solution. *Chemosphere*. **41**, pp. 1211-1218, 2000.
[15] Engwall, M. A., Pignatello, J. J. & Grasso, D., Degradation and detoxification of the wood preservative creosote and pentachlorophenol in water by the photo-fenton reaction. *Water Research*. **33**, pp. 1151-1158, 1999.
[16] Tang, W. Z. & Huang, C. P., Effect of chlorine content of chlorinated phenols on their oxidation kinetics by Fenton's reagent. *Chemosphere*. **33**, pp. 1621-1635, 1996.
[17] Oturan, M.A.,Oturan, N., Lahitte, C.& Trevin, S., Production of hydroxyl radicals by electrochemically assisted Fenton's reagent: application to the mineralisation of an organic micropollutant, pentacholophenol. *Journal of Electroanalytical Chemistry*. **507**, pp. 96-102, 2001.
[18] Smith, B. A., Teel, A. L. & Watts, R. J., Identification of the reactive oxygen species responsible for carbon tetrachloride degradation in modified Fenton's systems. *Environmental Science Technology*. **38**, pp. 5465-5469, 2004.
[19] Mecozzi, R., Palma, L. D., Pilone, D. & Cerboni, L., Use of EAF dust as heterogeneous catalyst in Fenton oxidation of PCP contaminated wastewaters. *Journal of Hazardous Materials*. **137(2)**, pp. 886-892, 2006.
[20] Weavers, L. K., Malmstad, N. & Hoffmann, M. R., Kinetics and mechanism of pentachlorophenol degradation by sonification, ozonation and sonolitic ozonation. *Environmental Science Technology*. **34**, pp. 1280-1285, 2000.
[21] Bonez, M. A., Bruning, H., W.H, R., Sudholter, E. J. R., Harmsen, G. H. & Bijsterbosch, J. W., Kinetic and mechanistic aspects of the oxidation of chlorophenols by ozone. *Water Science Technology*. **35(4)**, pp. 65-72, 1997.

Performance measurement in wastewater control – pig farms in Taiwan

C.-K. Hsiao[1] & C.-C. Yang[2]
[1]*Department of Finance, Nan Kai Institute of Technology, Taiwan*
[2]*Department of Applied Economics, National Chung Hsing University, Taiwan*

Abstract

Using a DEA approach this paper assesses the performance of wastewater treatment from pig farms in Taiwan. The results indicate that most pig farms have decreasing returns to scale. The average value of scale efficiency for the sample of pig farms is 0.901, and the pure technical efficiency is 0.821. These efficiency values indicate that most pig farms may improve performance of wastewater treatment through the adjustment of control equipment scale and increasing wastewater treatment efficiency. Moreover, the main cause of scale inefficiency is decreasing returns to scale, which means that increasing investment in pollution control may not provide a corresponding increase of wastewater treatment efficiency. Based on the farm size, it is found that larger pig farms usually have higher values of efficiency. In addition to the farm size, other factors affecting the environmental efficiency are also analyzed and discussed.
Keywords: pig farm, effluent regulation, environmental performance, data envelopment analysis (DEA).

1 Introduction

The hog industry is Taiwan's most important livestock industry. However, it is also the main cause of livestock water pollution. It is estimated that the daily quantity of biochemical oxygen demand (BOD) from wastewater is about 4,223 tons in Taiwan, and of this, livestock wastewater contributes 673 tons (16%) [1].

To regulate the water pollution caused by livestock industry, the Environmental Protection Administration (EPA) of Taiwan has implemented

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effluent standards, which are composed of BOD, chemical oxygen demand (COD), and suspended solids (SS), with maximum levels of 80, 600, and 150mg/l, respectively. To ensure their discharge meets the effluent standards, pig farms must construct wastewater treatment facilities, the so-called Three-Stage Wastewater Treatment System. However, pig farmers argue that the effluent standards are too rigorous to comply with. From the viewpoint of authorities, polluters are required to comply with the regulation. Nevertheless, the effluent standards are not easily met, and it is likely that violations may occur. Consequently, the goals of the regulation may not be achieved, and the regulatory agencies and the farmers are often in conflict.

In this context, it is important to explore the operational efficiencies of wastewater treatment facilities among different pig farms. By analyzing the relationship between inputs and outputs of wastewater control for pig farm sample, this study uses a nonparametric approach to inspect the pig farms' ability and performance in operating wastewater control. Thereby the technical and scale efficiencies on pollution abatement for pig farms can be calculated. A regression analysis is further used to test the relationship between efficiency values and the affecting factors. Then, the policy implications of improving the environmental performance of wastewater control may be derived.

2 Methods

The DEA has been widely used to assess the comparative efficiencies of homogeneous operating units such as banks, hospitals, and farms and so on. These units of assessment are usually termed as decision making units (DMUs), as termed by Charnes *et al.* [2]. By calculating Shephard's distance function [3], a conventional model can be applied to estimate various efficiencies via input or output orientation (e.g., [2, 4]).

Based on Luenberger's benefit functions [5], Chambers *et al.* developed a more generalized directional distance function to modify the traditional model [6]. Consider a single input (x) and a single output (y) production mix as illustrated in Figure 1, where DMU k is inefficient and can be projected onto the efficient bundle c (or bundle e) through an input (or output) orientation. Alternatively, any point between c and e, e.g. bundle d, could be the projected point when the directional distance function is used.

Consider a data set relating to N pig farms. For any individual farm k (k=1,…,N), let y_k denotes its $S\times 1$ output vector, x_k its $M\times 1$ input vector, respectively. For all farms, Y denotes their $S\times N$ output matrix, and X the $M\times N$ input matrix, respectively. Thus a DEA model based on directional distance function is formulated as follows [7]:

$$\begin{aligned} &\underset{\theta,\lambda}{Max}\,\theta \\ &s.t.\quad \begin{aligned} &Y\lambda-(1+\theta)y_k\geq 0 \\ &(1-\theta)x_k-X\lambda\geq 0 \\ &\lambda\geq 0 \end{aligned} \end{aligned} \tag{1}$$

where θ is a scalar, and λ is an $N{\times}1$ vector of intensity variables to ensure convexity of the production set. The calculated θ^* is the value of directional distance function, and the technical efficiency TE_{CRS} is then defined in terms of $1/(1+\theta^*)$ [7]. Model (1) is referred to as a CRS (constant returns to scale) technology. It can be easily modified to account for VRS (variable returns to scale) by adding constraint $I\lambda=1$ to model (1) [8], where I is a $1\times N$ vector of ones, and the pure technical efficiency TE_{VRS} can be calculated by a revised model. The ratio of TE_{CRS} to TE_{VRS} represents the scale efficiency (*SE*):

$$SE = TE_{CRS} \big/ TE_{VRS} \tag{2}$$

when the *SE* value is less then one, indicating a divergence between the efficiency rating of a pig farm under CRS and VRS, and the impact of scale size is caused either by increasing or decreasing returns to scale (IRS or DRS). By using non-increasing returns to scale (NIRS) frontier [8], IRS and DRS can be distinguished. In fact, adding a constraint $I\lambda \leq 1$ to model (1), its corresponding technical efficiencies TE_{NIRS} can then be estimated. As illustrated in Figure 1, the efficient frontiers for CRS, VRS, and NIRS are along the segments *obc*, *abce*, and *obce*, respectively. A pig farm has CRS if the TE_{CRS} value is equal to the TE_{VRS} value. If these two values are not equal but $TE_{NIRS}=TE_{VRS}$, then DRS is identified, otherwise the operation of control equipment is IRS.

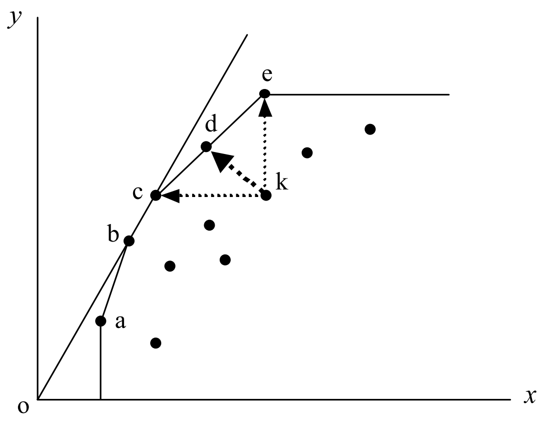

Figure 1: Illustration of IRS, CRS, and DRS.

3 Three-stage wastewater treatment system

The prevailing wastewater control equipment for pig farms in Taiwan is the so-called 'Three-Stage Wastewater Treatment System (TSWTS), which consists primarily of solid/liquid separator, anaerobic fermentation tank, and aerobic fermentation tank. In order to collect wastewater, a raw tank ahead of the solid/liquid separator is required; and after the process of aerobic fermentation, a sediment tank is installed to further collect sludge. A flow diagram of the TSWTS is shown in Figure 2.

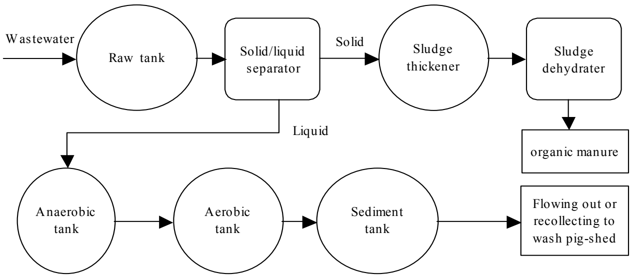

Figure 2: Flow diagram of TSWTS.

4 Data

Based on two projects co-sponsored by the EPA and the Council of Agriculture that were completed in 2003 and 2004, respectively, this paper took 31 pig farms as objects of study. Wastewater was drawn bimonthly from the raw tank and the sediment tank for inspecting the concentrations of BOD, COD, and SS. The differential values between raw and sediment tank are the output data, while the input data include investment in wastewater control equipment, operation and maintenance costs, and work hours for operating. Galanopoulos *et al.* [9] indicated that if inputs can be shared by per head of sow, then the DRS of control equipment in a small farm may be reasonably explained. Hence the input data in this study have been expressed as per head of sow.

Though the difference of before and after wastewater treatment can represent the degree of pollution reduction and to some extent may denote the effectiveness of control equipment operation, the differential value itself does not guarantee the correspondence of regulated standards. Therefore, BOD, COD, and SS are combined in this paper to check the pass ratio of total inspections.

5 Results and discussion

The estimated results of wastewater treatment performance for 31 pig farms are illustrated as in Table 1. There are 4 (12.9%) and 8 (25.8%) pig farms that reach the efficiency frontier under CRS and VRS assumptions, respectively.

According to the returns to scale of wastewater treatment, pig farms can be identified as IRS, CRS, and DRS and the respective numbers are 7 (22.6%), 4 (12.9%), and 20 (64.5%) (Table 2). The average values of TE_{CRS} and TE_{VRS} for pig farms with DRS are lower than those with IRS and CRS. It should be noted that the proportion of pig farms with DRS is about 65%. The large proportion of pig farms identified with DRS and failing to meet the effluent standards implies that other pig farms may have even more difficulty complying with the regulations.

Table 1: Environmental efficiency values and returns to scale.

DMU	TE_{CRS}	TE_{VRS}	SE	RTS*	DMU	TE_{CRS}	TE_{VRS}	SE	RTS
1	0.609	0.685	0.889	IRS	17	0.783	0.980	0.799	DRS
2	0.656	0.690	0.950	IRS	18	0.763	1	0.763	DRS
3	1	1	1	CRS	19	0.591	0.639	0.925	DRS
4	1	1	1	CRS	20	0.593	0.632	0.938	DRS
5	0.703	0.865	0.813	DRS	21	0.578	0.588	0.983	DRS
6	0.578	0.591	0.977	DRS	22	0.990	0.998	0.992	IRS
7	0.642	0.909	0.707	DRS	23	0.753	1	0.753	DRS
8	0.608	0.692	0.879	DRS	24	0.612	0.626	0.979	DRS
9	0.664	0.846	0.785	DRS	25	0.686	0.926	0.741	IRS
10	0.710	0.776	0.915	DRS	26	0.730	0.888	0.822	DRS
11	1	1	1	CRS	27	0.601	0.652	0.922	DRS
12	1	1	1	CRS	28	0.665	0.665	0.999	IRS
13	0.872	1	0.872	IRS	29	0.600	0.617	0.973	DRS
14	0.582	0.595	0.979	DRS	30	0.798	0.912	0.876	DRS
15	0.822	0.842	0.977	DRS	31	0.660	0.824	0.801	IRS
16	0.937	1	0.937	DRS	mean	0.735	0.821	0.901	

* RTS: Returns to scale, IRS: increasing returns to scale, CRS: constant returns to scale, DRS: decreasing returns to scale.

Table 2: Average efficiency values of RTS.

	IRS	CRS	DRS	F value	P value
TE_{CRS}	0.734	1	0.682	14.913	0.000
TE_{VRS}	0.827	1	0.783	3.667	0.039
SE	0.892	1	0.885	3.013	0.065
No. of farm (%)	7 (22.6)	4 (12.9)	20 (64.5)		

To test whether farm size may affect the performance of environmental efficiency, the sample was divided into three groups, i.e. 13 farms (41.9%) with less than 100 sows each, 9 farms (29.0%) with 101~200 sows each, and 9 farms (29.0%) with more than 201 sows each. The results indicate that with larger farms, there are higher values of environmental efficiency (Table 3).

Table 3: Average efficiency values of three different farm size.

No. of sow	≤ 100	101-200	≥ 201	F value	P value
TE_{CRS}	0.652	0.75	0.84	5.729	0.008
TE_{VRS}	0.77	0.821	0.893	1.663	0.208
SE	0.866	0.918	0.936	1.89	0.17
No. of farm (%)	13 (41.9)	9 (29.0)	9 (29.0)		

Furthermore, the relationships of farm size and RTS are analyzed in Table 4. It can be seen that all small farms (less than 100 sows) are DRS, and the larger farms have higher percentages of CRS and IRS. This may imply that the pollution control cost per head of sow is lower for larger farms, which is an advantage for RTS.

Table 4: Contingency table of farm size and RTS.

No. of sow	IRS	CRS	DRS	Chi-Square Test
≤ 100	0 (0)*	0 (0)	13 (100)	Chi-Square=15.06
101-200	3 (33.3)	1 (11.1)	5 (55.6)	P value=0.005
≥ 201	4 (44.4)	3 (33.3)	2 (22.2)	

* Numbers in parentheses are percentage values.

Table 5: Factors affecting control performance using Tobit model.

variable	TE_{VRS}		SE	
	coefficient	t value	coefficient	t value
constant	0.4260 **	2.144	0.8449 **	6.884
no. of sows on farm	0.0002	0.805	0.0003 *	1.86
whether shotes are sold	0.2116 **	2.245	0.0166	0.305
temperature of farm location	-0.0219	-0.567	0.0146	0.605
local characteristics of the farm location	0.0305	0.285	-0.0104	-0.16
quantity of treated wastewater collected	-0.0001	-0.123	0.0014 **	2.433
whether the anaerobic tank is clean	-0.0181	-0.272	0.0071	0.172
education years of operator	0.0809 **	3.000	-0.0064	-0.394
years of experience in operating	0.0068	0.829	-0.0011	-0.225
having training within 5 years	0.0354	0.564	-0.0400	-1.006

** Significant at 5% level;* significant at 10% level.

In addition to the RTS factor, other factors may also affect the performance of wastewater control, e.g. no. of sows on farm, the temperature of the farm location, local characteristics of the farm location, quantity of treated wastewater collected, whether the anaerobic tank is clean, the socio-economic conditions of the wastewater operator, etc. To test the impact of these factors on the efficiency values, a Tobit regression model has been used, and the results are shown as Table 5. Some of variables' signs meet the expectations and are statistically significant at the 5% or 10% level, e.g. the number of sows in a farm, whether the farm sells shotes, and the education years of the operator. The reason that higher level of education is conducive to the effective operation of wastewater control equipment may be because the treatment system is complex and requires education-related knowledge.

6 Conclusions

The average TE_{CRS}, TE_{VRS}, and SE for the pig farm sample are 0.735, 0.821, and 0.901, respectively. These results indicate that an average farm in Taiwan may have 26.5% of divergence for Pareto-efficiency in technical efficiency, 17.9% in pure technical efficiency, and 9.9% in scale efficiency.

By testing the RTS of wastewater control, the 31 pig farms are identified as 7 (22.6%) IRS, 4 (12.9%) CRS, and 20 (64.5%) DRS. Since the inspection ratio meeting regulated standards is only 0.56, and most farms are classified as DRS, it is very difficult for these farms to increase their operational efficiency by simply investing in more control equipment.

Several suggestions can thus be made from the above findings and discussion. First, more effective control equipment or techniques need to be developed to replace the current three-stage system. Second, the average size of pig farm should be further increased. Third, the size of pigs should be adjusted in some farms. Last, pig farms should hire operators with higher education or better experience.

References

[1] Environmental Protection Administration (Taiwan), *Environmental Statistics*, EPA: Taipei, 2006.
[2] Charnes, A., Cooper, W.W., & Rhodes, E., Measuring the efficiency of decision making units. *European Journal of Operations Research*, **2(4)**, pp. 429-444, 1978.
[3] Shephard, R.W., *Theory of Cost and Production Functions*, Princeton University Press: Princeton, 1970.
[4] Banker, R.D., Charnes, A., & Cooper, W.W., Some models for estimating technical and scale inefficiencies in data envelopment analysis. *Management Science*, **30(9)**, pp. 1078-1092, 1984.
[5] Luenberger, D.G., Benefit functions and duality. *Journal of Mathematical Economics*, **21(5)**, pp. 461-481, 1992.
[6] Chambers, R.G., Chung, Y., & Färe, R., Benefit and distance functions. *Journal of Economic Theory*, **70(2)**, pp. 407-419, 1996.

[7] Chung, Y.H., Färe, R., & Grosskopf, S., Productivity and undesirable outputs: a directional distance function approach. *Journal of Environmental Management*, **51(3)**, pp. 229-240, 1997.
[8] Coelli, T.J., Prasada Rao, D.S., & Battese, G.E., *An Introduction to Efficiency and Productivity Analysis*, Kluwer Academic Publishers: Boston, 1998.
[9] Galanopoulos, K., Aggelopoulos, S., Kamenidou, I., & Mattas, K., Assessing the effects of managerial and production practices on the efficiency of commercial pig farming. *Agricultural Systems*, **88(2-3)**, pp. 125-141, 2006.

Section 7
Water markets and policies

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Do markets promote more efficient and higher value water use? Tracing evidence over time in an Australian water market

H. Bjornlund[1,2]
[1]*University of South Australia, Australia*
[2]*University of Lethbridge, Canada*

Abstract

This paper analyses buyers and sellers of water entitlements in Australia based on surveys conducted during three time periods from 1992 to 2006 to identify whether water markets have facilitated a reallocation of water from inefficient, unproductive and low-value users to efficient, productive and high-value users. There is evidence that the entitlement market has an increasing impact on the way water is used and facilitates reallocation. While the early markets mainly activated previously unused water, there is evidence that as the market matures, more actively used water is being sold reducing the productive capacity of the selling property. The evidence suggests that many sellers reduced their irrigated area during the last five years, in response to prices in the seasonal market for water allocations, before eventually selling their entitlement.
Keywords: water trading, water reallocation, community impact, Australia.

1 Introduction

As water scarcity intensifies around the world water markets are increasingly being proposed to facilitate a reallocation of water from inefficient low-value users to efficient high-value users to minimize the overall economic impact of scarcity. The expectation is that the buyers will be able and willing to pay the sellers a price that compensates them for their losses. However markets in water entitlements (the market in which the long term entitlement to receive annual water allocations is traded) have been slow to evolve. Australia is one of a few countries where entitlement markets have been operating for some time. Australia therefore provides an opportunity to investigate whether the market has

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achieved the anticipated outcomes. This paper investigates this using the Goulburn-Murray Irrigation District (GMID) in Victoria as a case study. The author has previously analysed the impacts of water trading during the first five years of trading [1–5] and the results indicate that the anticipated reallocation takes place. It was, however, pointed out that the outcome of these early markets should be seen as indicative only, as activities in the market were limited, with less than 0.5% of the entitlement base traded each year, and the early markets were dominated by unused water with about 50% of the water sold not being used by the sellers. Ten years later these conditions have changed. Market participation has increased [6] and the volume traded has during the last three years been about 2.5% p.a. of the entitlement base [7] and the volume of unused water is likely to have declined. This paper analyses water trading for the three year period 2003-2006 based on surveys of buyers and sellers to identify the extent to which the anticipated reallocation takes place. These findings are then compared with the previous findings from the period 1992-1996. The paper first provides a brief overview of the emergence of water markets in Australia and within the case study area and how trading activities have developed.

2 The introduction of water trading in Australia – an overview

Water trading was first introduced in the south-eastern part of Australia where water resources within the Murray-Darling Basin (MDB) showed the first signs of over allocation with the result that over time a moratorium was placed on the issuing of new water entitlements within most catchments. When demand for water increased from new or expanding water users it could only be met by reallocating existing entitlements. At the same time there was an interest in reallocating water away from areas unsuitable for irrigation and away from inefficient and low-value users.

Due to a general concern in the community about the potential impact of the sale of water entitlements, the entitlement market was slow in emerging. During the late 1970s and early 1980s many water authorities allowed informal transfers between farmers to accommodate the need to reallocate water during periods of severe drought. Formal provisions for seasonal trade in water allocations were first made in South Australia (SA) and New South Wales (NSW) in 1984 and pilot programs in some districts were introduced in Victoria in 1987. Markets for water entitlements first emerged in SA in 1984. This state first felt the pressure for mechanisms to permanently reallocate water to meet new demand from horticulture and viticulture since: i) SA had already in the 1970s reduced irrigators' entitlements according to actual use; ii) horticulture and viticulture had no interest in trading in water allocations as they need long-term supply security for their high-value and capital intensive permanent crops. Trading in water entitlements were introduced by legislation in NSW in 1989 but did not emerge within the irrigation districts until these were privatized in the mid 1990s. In Victoria trading in allocations and entitlements were introduced by legislation in 1989 but entitlement trading did not commence until the necessary regulations were introduced in September 1991.

Two policy initiatives have promoted the adoption of water markets. First, in the early 1990s it became apparent that the MDB had serious water quality problems. This resulted in an audit of water use which found that the Basin was overcommitted for extractive use and that the level of extraction would continue to increase by at least 14% if no action was taken. Such development would result in further detrimental environmental impacts and threaten the future economic use of water. A Cap was therefore placed on water use within each state set at the 1993/94 level of development. As trade activated unused water, and as drought and increased need for environmental water reduced the consumptive pool, less and less water was allocated each season in order to stay within the Cap. This caused a further increase in water market activities, especially in the markets for seasonal allocations, as irrigators with permanent crops were struggling to secure adequate water.

Second, in 1994 the Council of Australian Governments (CoAG) agreed on a new water policy framework calling for the separation of water entitlements from land, more clearly specified entitlements and water trading [8]. Trade in water entitlements was encouraged to ensure that water is used to maximize its contribution to national income and welfare within social, physical and ecological constraints of catchments. This framework was further strengthened in 2004 with the National Water Initiative (NWI) [9] which emphasises that existing water markets and mechanisms are preventing markets from reaching their full potential and that more efficient market mechanisms and more sophisticated market instruments are necessary. The NWI therefore includes a commitment to: i) introduce water entitlements with nationally compatible characteristics defined as a perpetual share of the available pool of water for consumptive use; ii) introduce secure entitlement registers, where third party interests can be registered; and iii) gradually remove all barriers to trade. It has to be noted that the removal of barriers to trade is associated with substantial opposition within irrigation communities [10].

3 The introduction of entitlement trading in Victoria

The first transfers in Victoria were registered in January 1992. To alleviate the concern over the impact of trading in water entitlements, the regulations limited the ability to trade in water in six main ways: i) geographical restrictions on how water could move around; ii) to ease the concern over the impact of trading out of irrigation districts, an annual maximum was set at 2% of the entitlement base at the beginning of the year; iii) trading was not allowed between different classes of water entitlements such as private irrigators pumping their own water from the rivers and irrigators within the irrigation districts; iv) to ease environmental concerns, limits were placed on how much water could be purchased onto each parcel of land depending on the quality of drainage, laser grading and irrigation technique and trading into certain high impact zones were prohibited; v) to ease the concerns of banks, trade was only allowed following

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consent from third parties with an interest in the land to which the water was attached; and vi) to alleviate concern over channel capacity, trade would only be allowed if the water could be delivered by the existing supply infrastructure without reducing existing water users' supply reliability.

These restrictions have been continually revised: i) in 1994, trading between the two major entitlement types was allowed and trade to outside the GMID was introduced. It was initially feared that his would cause a significant export of water as downstream demand from horticulture and viticulture would drive water in that direction; ii) in 1995, some of the geographical restrictions on trade within the GMID were eased; iii) in 1997 further allowances were made for interstate trading with NSW; iv) under the NWI interstate trading with both NSW and SA was finally agreed in 2006 and the barriers to trade out of districts were expanded from 2% to 4% in 2005; and v) in 1995 the regulations as to how much water could be purchased onto a given parcel of land were revised to require that the land on which the water would be applied must be suitable for irrigation. Further revision took place in 1996 when new Salinity and Drainage Guidelines were approved. The major changes were that: i) the limits on how much water could be traded onto a parcel of land were removed and replaced by a limit on how much water could be used. This allowed irrigators to buy more water entitlements to secure against low seasonal allocations; and ii) the need to have suitable land was replaced with the requirement of an irrigation and drainage system consistent with modern practices and suitable for the soil condition and crop type of the farm.

The Victorian government has been reluctant to detach the water entitlement from land ownership. The transfer of water entitlements required that the water authority detach the entitlement from one parcel of land and then attach it to another. The latest policy reform in Victoria (DSE [11]), in compliance with the NWI, finally introduced the separation of land and water rights. To alleviate the community concern that this separation will result in a consolidation of water entitlements into large speculative corporations to the detriment of family farming [10], a rule was introduced that only 10% of any water resource can be owned by the same entity. It was also decided to introduce a new water use right which will require all water users to prove that they are using the water according to best irrigation and drainage practices. To alleviate the concern that trade out of supply systems would leave the remaining water users with a higher maintenance burden, a separate supply capacity entitlement tied to land was introduced. Charges to cover the cost of infrastructure maintenance will be attached to this capacity entitlement which will remain with the land even after the water entitlement has been sold.

4 The development of trade within the GMID

Trading in water entitlements evolved slowly due to a number of factors: i) restrictions on trading discussed in the preceding section; ii) cumbersome administrative procedures; iii) uncertainty associated with the successful completion of transfers; iv) uncertainty about the level of long term allocations yielded by the entitlement; and v) perception of water as an inherent part of the farm [12].

During 2005/06 the total volume of water entitlements within the GMID was 1,793,637 ML. Since the first trade of water entitlement to outside the district took place in 1996/97, some 124,265 ML have been exported out of the area, representing a reduction of 6.5% of the entitlement base. The GMID consists of a number of irrigation districts with very different soil and production characteristics. Export of water out of individual districts has therefore varied substantially. The two districts experiencing the biggest proportional export of water have seen a drop in their entitlement base of some 11.5% [7].

During the first 15 year of entitlement trading (from 1992 to 2006) 308,666 ML, representing 17.2% the entitlement base, have changed hands. Even though entitlement trading has been taken up slowly, there are, by the end of the first 14 years, indications that it is starting to have some impact on the distribution of water entitlements. Annual volumes traded have increased from less than 0.5% of the entitlement base during the first five years to more than 2.5% during the last three years [7]. Similarly the participation rate of farm businesses started out with only 0.5% of them active in the market each year increasing to 2.5% selling and 2% buying during 2003/04, at which time some 8% of farm businesses had sold and 8% had bought water entitlements [6].

5 Data and methodology

This paper is based on surveys conducted during three time periods: i) mail questionnaires from 1992-1994 with 337 usable responses representing 58.5% of all traders; ii) telephone interviews from 1995-1996 of 100 buyers and 100 sellers representing about 30% of all traders; and iii) mail questionnaires from 2003-06 with 157 useful responses representing some 15% of all traders. In all instances the sellers' response rate was lower than the buyers' as many sellers are disenfranchised with the industry and therefore will not return the questionnaire. Also, about 7% of all surveys send to sellers were returned with the person unknown at the address reflecting that these sellers have sold or abandoned their property and have moved in search of a job. The study thus covers the first five years of trading and a three year period after 10 years of market experience. The data was not collected for the purpose of this longitudinal study. Each survey instrument was designed based on the experiences with the preceding instrument and to accommodate the need for more and different data. Consequently, in some instances different questions were asked and longitudinal comparisons were not possible.

6 Has entitlement trading resulted in more efficient, higher value and more productive water use?

To determine whether more efficient water use has resulted from entitlement trading, respondents were asked: i) whether they have the various types of irrigation and drainage infrastructure and to which extend their irrigated land is

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serviced by such infrastructure; ii) what they have done over the last five years and what they intend to do over the next five years to improve their irrigation and drainage infrastructure; iii) whether they have a whole of farm plan for their property and use some kind of technical aid to schedule their irrigation; and iv) questions related to their use of extension services, participation in training events, and membership of professional and community organization. To determine whether water has move to more valuable and productive uses, respondents were asked: i) what they produce on their land; ii) what they have done in the last five years and intend to do in the next five years to change production and buy and sell land or water; iii) the size of their irrigated area and water entitlement; iv) whether they find their property to be long-term viable; and v) how they perceive that their productivity is developing. To aid the answer to both questions a number of demographic questions were asked such as: i) age; ii) family background in farming; iii) expectation of family continuity; iv) schooling; v) further education and vi) dependence on off-farm work.

6.1 Entitlement trading and more efficient water use

During the early period, there are very clear signs that far more buyers have the various irrigation and drainage infrastructure and also have a larger proportion of their irrigated area serviced by such infrastructure (table 1). This is particularly evident when it comes to re-use systems. This is caused by the dominance of the dairy industry as buyers in the early markets (table 2). During 2003-2006 these trends changed as the difference between the proportion of the buyers and sellers having the various irrigation and drainage infrastructure became less pronounced. This is likely to be caused by at least three factors: i) the irrigation industry has generally gone through a period of infrastructure upgrading both on and off-farm; ii) the changing fortune of the dairy industry. Due to drought, declining allocations, deregulation of the industry, and low commodity prices, the dairy industry has been suffering over the last five to ten years. Therefore far more dairy farms are selling their water and getting out of dairying; and iii) new irrigation methods such as centre pivots, sprinklers and drip irrigation has been introduced (20% of buyers reported other irrigation method than gravity) following the emergence of new and higher value industries such as viticulture, olives and vegetables (tomatoes) (13.2% of buyers, table 2). These productions do not benefit from laser grading, re-use system and off-farm drainage as this infrastructure is mainly used a measures to improve the efficiency of gravity irrigation. There is also a significantly larger percentage of buyers having a whole of farm plan and using aid in scheduling their irrigation, indicating more efficient water use.

During the early period it can also be seen (table 1) that many of the buyers of entitlements installed this irrigation and drainage infrastructure subsequent to the purchase of the water, making the difference between buyers and sellers even more pronounced. This trend is also evident during the second period, with a significantly larger percentage of buyers having improved their irrigation and drainage infrastructure the last five years and expecting to do so the next five years. From table 3 it is also apparent that the buyers are younger with more agricultural qualifications and more actively participating in training events and using extension services suggesting that this group are more likely to be actively pursuing more efficient water use and the introduction of more efficient irrigation technology. Finally if we consider that more viable farms with continuity of family ownership are likely to result in more efficient water use, then table 3 clearly shows that the buyers consider their property to be more viable with an increasing production. This is supported by the fact that the buying properties are considerable larger and have considerable larger water entitlements, especially during the last period (table 2).

Table 1: Water use practices and past and future farm changes.

	1994-96				2003-06			
	% of water		% of area		% of respondents		% of area	
	Buyers	sellers	Buyers	Sellers	Buyers	Sellers	Buyers	Sellers
Re-use system	74.7	36.5	74.8	45.4	72.2	49.2	59.4	38.5[2]
Laser grading	80.3	54.4	66.0	64.1	80.0	83.1	63.1	61.4
Surface drains	76.9	70.2	93.1	82.6	77.8	69.8	69.7	60.5
Off-farm drains	55.4	28.6			48.8	58.5	42.0	52.6
Other irrigation system					20.0	12.3	10.9	5.7
Installed it since purchasing the water:								
Re-use system		7.9						
laser grading		10.4						
On-farm surface drains		12.5						
Of the buyers who did not have a re-use system 29% inteded to get one								
							% of respondents	
Do you have a whole of farm plan							78.4	60.3[2]
Do you use aid in scheduling irrigation							18.9	9.0[3]
Within the last five years have you improved your irrg. and drainage							73.9	53.2[1]
Within the last five years have you bought more water right							94.0	12.9[1]
Within the last five years have you bought more land							25.0	9.8[2]
Within the last five years have you changed your irrig. production							42.6	38.4
Within the last five yaers have you reduced your irrigated area							10.2	41.0[1]
Within the last five years have you sold water right							6.9	72.6[1]
In the next five years will you improve your irrigation and drainage							54.5	42.9[2]
In the next five years will you buy additional water right							28.4	10.9[1]
In the next five years will you buy additional land							24.7	10.9[1]
In the next five years will you change your irrigated production							31.0	25.0
In the next five years will you reduce your irrigated area							2.3	14.5[1]
In the next five years will you sell water right							1.1	15.9[1]
Significance level using Pearson Chi-Square [1] 0.0; [2] 0.05; [3] 0.1								

6.2 Entitlement trading and higher valued and more productive water use?

Traditionally water markets within the GMID have been dominated by the dairy industry as the high-value water user in the region. This is evident during all three periods (table 2). However the dominance of dairy has reduced over time with fewer buyers and more sellers being dairy farmers. This reflects, as previously discussed, the hard economic times that the dairy industry has experienced during this period. Still there are a significantly larger percentage of sellers being cattle and sheep farmers. On the buying side there have been and increase in purchases by horticultural producers, but also by cropping and sheep producers and by buyers with no crop which represent new irrigation enterprises as well as golf courses, caravan parks etc. Among the sellers there have been increases in sale by cropping farmers and farmers without irrigation.

Table 2: Land use and farm and entitlement size of buyers and sellers.

	1992-94		1994-96		2003-06	
	% of water				% of respondents	
Production:	Buyers	Sellers	Buyers	Sellers	Buyers	Sellers
Dairy	79.4	6.8	69.0	10.4	36.4	24.1
Cattle	9.8	37.9	18.3	44.5	15.5	31.5[2]
Sheep	1.6	39.9	1.5	26.3	7.2	17.0[3]
Cropping	3.5	6.7	3.7	4.6	7.2	14.8
Horticulture	5.8	2.8	2.2	5.6	13.2	7.4[2]
No Crop	0.0	5.9	5.3	8.6	7.8	19.4
Irrigated area:						
No irrigated land			12.1	16.2	7.9	19.4
20 hectares or less			10.3	33.9	19.1	13.4
21 to 50 hectares			17.8	13.1	14.6	17.9
51 to 100 hectares			28.0	11.0	15.7	19.4
101 or more hectares			31.8	26.3	42.7	29.9
Mean farm size			96.1	83.7	294.3	97.4
Size of water entitlement						
No entitlement			5.6	na	3.4	10.6
50 Ml or less			12.2	24.6	11.2	12.1
51 to 200			40.3	42.1	24.7	28.8
201 to 500			32.6	25.4	31.5	31.8
500 or more			9.4	7.9	29.2	16.7
mean			226.8	199.2	620.0	301.6

Significance level using Pearson Chi-Square [1] 0.01; [2] 0.05; [3] 0.1

While sellers without irrigation during the early years represented entitlement holders which never had used their water and now used the introduction of trade to sell this unused asset, during the last period many of these sellers represent irrigators who have stopped irrigation recently in response to high water market prices both in the allocation and entitlement market (table 1 shows that 41% of sellers have reduced their irrigated area the last five years). Surveys of sellers in the seasonal allocation market indicates that as many as 29% of the sellers do not irrigate at all and instead sell their water each year [13]. Some of these entitlement owners are likely to be tempted to sell their entitlements as the price of these has gone up by some 12.7% per annum over the last 13 years [14].

The facts that the buyers have more viable and productive farms operated by younger and better agriculturally qualified people, more actively involved in improving their production, also suggest that the buying properties are more productive (table 3). The fact that the sellers have a larger proportion with non-agricultural qualifications reflects that many of the selling farms are considered to be hobby farms or rural residences. Further, considering that about 7% of all sellers no longer reside at the address of the property suggests that many are adjusting out of farming. There is evidence that this is part of a longer trend even though previous research [15] suggests that the market for seasonal water allocations seems to delay this process as a large proportion of farmers are using this market to remain on the farm and in the community for the rest of their working life. While buyers and sellers have much the same family background in farming, it is evident that many of the sellers are not commercial farmers but simply live in a rural area while working off-farm. This trend is going to continue since only 24% of buyers and 14% of sellers are certain of family continuity of their farm business while a quarter are uncertain, leaving only a quarter to half of the farm businesses in operation after this generation.

Table 3: Demographic information.

	2003-2006			
	Buyers		Sellers	
Family and farming 2003 to 2006	yes	uncertain	yes	uncertain
Expects family members to take over the farm	24.4	26.7	14.3	23.8
Household member belongs to a community group such as Land Care	48.3		41.5	
Household member belongs to a professional organization such as a farmers association	56.2		50.8	
Household member have off-farm work	53.4		61.5	
% of household income from off-farm work	54.7		52.9	
Viability of farm				
Considers your farm to be long-term viable³	68.2		55.6	
How is your productivity developing¹				
Decreasing	10.3		30.0	
Steady	40.9		38.3	
Increasing	48.8		31.7	
	1994-96		2003-06	
Ageand family tradition in farming	Buyers	Sellers	Buyers	Sellers
40 or younger	29.2	21.0	14.0	10.8
41 to 50	35.8	33.0	30.2	24.6
51 to 60	24.5	21.0	37.2	30.8
61 or older	10.4	25.0	18.6	33.8
Where you parent also in farming	81.3	77.0	72.4	72.3
Qualification and training				
When did you leave school				
Year 10 or earlier			18.6	21.5
Year 11 and 12			81.4	78.5
Have agricultural qualifications			40.4	29.2
Have non-agricultural qualifications			48.3	55.4
Participated in training events the last 12 months			65.2	53.8
Used extension services the last 12 months³			46.6	29.2
Significance level using Pearson Chi-Square ¹ 0.01; ² 0.05; ³ 0.1				

7 Conclusions

There is evidence that the entitlement market has an increasing impact on how water is used and that it moves water from less efficient and lower value producing to more efficient and higher value producing farms. While the early water market mainly activated unused water, there is evidence that as the market matures and activates this unused water, more actively used water is being sold reducing the productive capacity of the selling properties. Entitlement markets therefore seem to have an increasing impact on local farming communities. The operation and continued high level of use of the allocation market have slowed this process down, as the current generation of irrigators struggle to stay on their properties and within their communities. This has a beneficial impact on communities' and individuals' ability to adjust and deal with the rapid pace of change. Finally there is strong evidence that the entitlement market in the medium future will have an increasing impact on water allocation and structural change in many communities as only between a quarter and half of the existing farm businesses will remain in operation after this generation.

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References

[1] Bjornlund, H. *Water Trade Policies as a Component of Environmentally, Socially and Economically Sustainable Water Use in Rural Southeastern Australia.* Thesis submitted in fulfilment of the requirement for the degree of doctor of philosophy, University of South Australia, Adelaide, 1999

[2] Bjornlund, H. Formal and informal water markets – Drivers of sustainable rural communities? *Water Resources Research* 40, W09S07, 2004.

[3] Bjornlund, H. Can water markets assist irrigators managing increased supply risk? Some Australian Experiences. *Water International,* 31(2), 221-232, 2006.

[4] Bjornlund H and McKay, J., Aspects of Water Markets for Developing Countries – Experiences from Australia, Chile and the US. *Journal of Environment and Development Economics* 7(4), 767–793, 2002.

[5] Bjornlund, H. and McKay, J., Are Water Markets Achieving a More Sustainable Water Use. *Proceedings from the X World Water Congress,* Melbourne, March, 2000.

[6] Bjornlund, H., Increased participation in Australian Water Markets. In Lorenzini, G. and Brebbia, C.A. eds. *Sustainable Irrigation Management, Technologies and Policies*. Southampton: WIT Press, 289-302, 2006.

[7] Bjornlund, H., Tradable permits - instruments to manage water scarcity? - Some Australian experiences. *Proceedings from the International Workshop on Tradeable Permits*, Wittenberg, Germany, November , 2006

[8] CoAG, Council of Australian Governments, *Communiqué*, Meeting of CoAG in Hobart 25 February 1994. 1994

[9] CoAG, Council of Australian Governments, *Intergovernmental Agreement on a National Water Initiative*. www.dpmc.gov.au, 2004

[10] Bjornlund, H. Water markets and the environment – what the irrigation community told us. *Proceedings from the Riversymposium*. www.riversymposium.com/index.php?page=GeneralArchive , 2004.

[11] DSE, Department of Sustainability and Environment, *Victorian Government White Paper: Securing our water future together.* Department of Sustainability and Environment, Melbourne, 2004.

[12] Bjornlund, H., Farmer Participation in markets for temporary and permanent water in southeastern Australia. *Agricultural Water Management* 63(1), 57–76, 2003.

[13] Bjornlund, H., *The Monitoring of, and reporting on, water trading within the Goulburn-Murray Irrigation District,* Industry Partner Report of ARC Linkage Grant Water Scarcity and Rural Social Hardship – can water markets alleviate the problems? University of South Australia. For copy henning.bjornlund@unisa.edu.au. 2006

[14] Bjornlund, H. and Rossini, P., tracing evidence of rational investor behaviour in water markets – revisited. *Proceedings from the 13th Annual Conference of the Pacific Rim Real Estate Society*, Fremantle, Western Australia, January. www.business.unisa.edu.au/prres. 2007

[15] Bjornlund, H. The socio-economic structure of irrigation communities – water markets and the structural adjustment process. Journal of *Rural Society* 12(2), 123–145,

The cost of water and water markets in Southern California, USA

W. G. Hamer
Geomatrix Consultants, Inc, Newport Beach, California, USA

Abstract

It is surprising to learn what people pay for water in Southern California. Of course, it depends on what you need the water for, when you need it, and where it comes from. Rates for the least expensive water, for agricultural use, are \$15 (€12) or more per acre-foot (1,230 m^3). The highest-cost wholesale water is desalinated seawater at a cost of \$800 to \$900 per acre-foot (AF). The base treated and untreated water rates for the largest wholesale water purveyor in California, Metropolitan Water District of Southern California, are \$371 and \$478 per AF (€232 and €300 per 1000 m^3). Actual rates paid by water districts for Metropolitan water vary depending on the type of service and the costs to deliver the water to the service location.

Residential water rates in California, which include delivery and service charges, averaged \$905 per AF in 2005 (€567/1000 m^3). The average residential user in 2005 required 0.4 AF (490 m^3) of water for the year. Residential water rates have increased over the last 15 years, from approximately \$20 (€15) per month in 1991 to approximately \$35 (€27) per month in 2005, representing an average annual increase of 3.8%.

As the need for reliable water supplies increases so does the cost. Informal and formal water markets are increasing. Markets for permanent water rights and annual water rights have been incorporated into recent legal judgments for groundwater basins in Southern California. Water markets are helping to reduce over pumping of groundwater basins. They promote better planning for droughts, water conservation, and increased water system reliability.

Keywords: water cost, water markets, water rates, water supply, California water.

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1 Introduction

As major cities have developed in Southern California, the costs that citizens have paid for water have risen steadily. Competition for ever-scarcer low-cost reliable supplies is growing, and as a result, costs are increasing.

Despite appearances to the contrary, there is enough water to meet the needs of Southern California residents and industry. Half of the water used in California is used to water just four crops: irrigated pasture, alfalfa, cotton, and rice! [1]. Overall, agriculture consumed more than 80 % of water used in California in 1990. In 1990, the director of the California Water Resources Center observed: "We've still got plenty of water. It's just misallocated." [2].

The question is how much will adequate, reliable water supplies cost Southern California residents? Water supplies are often discussed in terms of environmental, social, and financial costs. This paper focuses on the financial costs for water in Southern California. The overall range in costs for water is discussed along with information regarding imported and retail water costs. Some historic information is also presented to provide perspective on the changes in costs in recent years. Additionally, recent creative efforts by water suppliers are also discussed, with special emphasis on the water market in the Mojave River drainage, in San Bernardino County, California.

2 General factors affecting the cost of water

The cost of water varies widely, depending on many factors. Just ask anyone in the water supply industry about his or her water costs, and it quickly becomes obvious that the subject is a complex and dynamic one, with costs based on many factors. You're likely to hear a wide range in costs and answers, ranging from bulk water costs to pumping costs, to complaints about their monthly water bill! The more significant factors include the following.

- Potability. Whether or not the water is treated or treatable for potable use is a key factor. Surface waters must be treated for potable use. Reclaimed water, although treated, is not currently accepted for potable use. (There is an aversion to using water that is "Toilet to tap").
- Location. The point of use has a significant impact on the cost of water. Pumping and distribution costs may be a significant factor in what the user pays for water.
- Volume. The volume of water required ranges from bulk water to bottled water.
- Reliability. The reliability of the supply, both on a seasonal and long-term basis has a significant effect on the overall cost.
- Power costs. In most cases, power costs for pumping and distributing water are the most significant component in the rate paid by the user.

These various factors are discussed below as they relate to the types of waters consumed in Southern California.

2.1 The cost of bulk (wholesale) water

Bulk or wholesale water is considerably lower in cost than potable water delivered to individual residential or industrial users. Bulk water supplies are primarily surface waters, including imported aqueduct water and local surface waters. Distribution costs, which may be significant, are much less for bulk water.

The lowest-cost water in Southern California is agricultural water. In the Imperial Valley, and in parts of the Central Valley, farmers pay as little as \$15 per acre-foot (AF) for irrigation water (€9/1000 m^3). As recently as 1990, subsidies resulted in farmers obtaining untreated Central Valley Project water as cheaply as \$3/AF (€1.9/1000 m^3), and from the California State Water Project (SWP) aqueduct as cheaply as \$62/AF (€39/1000 m^3) [3].

Local Groundwater is often pumped for farming use at a cost in the range of \$40 to \$60/AF (€25 to €37/1000 m^3). Because groundwater in many cases does not require treatment to be potable, the only cost to the user is the pumping cost, which is governed primarily by the cost of power.

Imported water supplies vary considerably in cost. Water imported for use in Southern California travels via one of three aqueduct systems: the SWP, the Colorado River Aqueduct (CRA), and the Owens River Aqueduct. These imported water supplies, being surface water, require treatment prior to their use as potable water. The price that agencies pay for regular supplies from these aqueduct systems ranges from \$148 to more than \$570/AF (€93 to €360/1000 m^3) [4, 5].

The SWP also offers supplemental water that is periodically available. This mechanism for obtaining supplemental waters acts as a type of basic "water market". An example of the buying and selling of water in this manner is the California Department of Water Resources' "water bank" where farmers and agencies that do not need their full water entitlement allow the state to sell the available water to cities and water agencies.

In recent years, technological improvements have caused the costs for desalinization of sea water to decline. Recent estimates for the Orange County Water District for a large-scale desalinization plant (60 million gallons or 180 AF/day) are approximately \$900 to \$1200/AF (€560 to €750/1000 m^3). The West and Central Basin Municipal Water District operates a desalinization test program. They estimate the cost of seawater desalinization at approximately \$900/AF. To encourage desalinization, Metropolitan has offered a subsidy of \$250/AF for up to 50,000 AF/year of seawater desalinization. This subsidy would reduce the cost to \$650/AF (€410/1000 m^3); in the range of some other potable water supplies.

Lastly, reclaimed water is becoming an increasing source of supply. Reclaimed municipal wastewaters are being sold for a variety of non-potable uses, primarily landscape irrigation. In the Coachella Valley, reclaimed water is sold by the Desert Water Agency for 50% of the potable water cost, or approximately \$150/AF (€95/1000 m^3). In Orange County, reclaimed water is sold by the Irvine Ranch Water District for 10 to 20% less than potable water [6].

2.2 The cost of retail water

The price that a Southern California homeowner pays for water varies considerably. At a glance, significantly higher rates in one area than in another may seem unreasonable. This may not be the case, however. The setting of retail water rates is a complicated process, based on many factors. The factors affecting water rates in an area include the following.

- Sources of funds used by and available to a utility. Sources may be retail water sales, monthly service fees, and property taxes.
- Geographical factors. These include the distance to any imported water sources, and the size and elevation differences within the service area.
- Rate design. This includes the costs involved in operating the water system and the means to fund them. For example, an older water system may require higher rates to pay for system improvements.
- Reserve funds. Rates may include costs for the utility to maintain a reserve for funding growth or debt service.

Retail water rates are commonly stated in cost per "hundred cubic feet" ($/ccf). In order to compare retail rates with wholesale or bulk water rates, retail rates are expressed below in $/AF. Because all water rates, both wholesale and retail are made up of many complex factors, comparisons should be considered only general in nature. Water rates and consumer connection fees have increased in recent years as the result of increasing demand and to an aging infrastructure and rising construction costs [6].

From 2003 to 2006, the average residential monthly charge, for 1500 cubic feet (42 m^3) of water per month, increased from \$30.33 to \$36.39 (€23,33 to €28). This is a total 17% increase during the three-year period.

3 A water market helps control water costs and use

Over pumping of groundwater in the Mojave River Basin (Basin) in the Mojave Desert of Southern California resulted in the legal adjudication of groundwater rights in the Basin and development of a water market. The water market is helping to reduce groundwater pumping and to provide for importing water to recharge the groundwater in the area. Costs for buying and selling water rights on the water market are increasing as limited water supplies are needed to meet increasing demands.

The Basin is approximately 9900 km^2 in area. Most of the area is very dry, receiving only an average of four to 5 inches (10 to 12 cm) precipitation per year.

Summer temperatures are very high; the mean annual maximum temperature in the Basin is approximately 27 degrees centigrade (^{o}C).

Rapid population growth has resulted in intensive use of the aquifers in the area. The population growth rate from 1960 to 2000 was approximately 5,5%. From 1990 to 2000 the population in the basin has increased from about 243000 to 290000 people and water demand has increased by approximately 5% per year. The increasing population has caused the water use in the Basin to shift from predominantly agricultural use to predominantly municipal use. The percentage of agricultural use continues to decline as the population increases.

In the Basin, groundwater is the primary source of water supply for communities, farms, and industry and groundwater extractions have greatly exceeded the safe or sustainable yield of the aquifer system in the basin. The safe yield is the amount of groundwater that can be produced from a basin or subarea on a long-term basis without a reduction in the amount of water in storage and without adverse effects on the resource. The Safe Yield for the Alto Subarea of the Basin is 69900 AF/year (56,6 mm^3/year). The amount of groundwater that had been pumped from the subarea during the early 1990s (the Base Annual Production, or BAP) was 122400 AF/year (99,2 mm^3/year). The pumpage had been greater than the Safe Yield and the area is severely overdrafted and groundwater levels declining.

For the Basin as a whole, approximately two-thirds of the groundwater consumptive use is for urban uses (municipal and industrial). The remaining one-third is agricultural use.

3.1 The Mojave Basin Judgment

As groundwater levels declined in the Basin, water users sued each other to protect their water rights, and the matter went to Court. In determining the best course of action, the Court had to consider how best to balance the legal rights and needs of agricultural interests in the basin and the water demands of a rapidly growing population. Many of the farms in the area had long histories of pumping groundwater for irrigating crops, including alfalfa, and various fruits and vegetables. The farmers felt that the growing population centers were threatening their water rights and ability to pump groundwater at relatively low cost. The Court had to find a solution that was acceptable to all of the primary water interests, including farmers, land developers, cities, water districts, and small water users. In doing so, the Court had to provide an incentive to farmers to accept the agreement, and an assurance that their long-standing water rights would be protected. Final legal challenges to the legal Judgment were settled in July 2002, including the ability for farmers to sell or lease their water rights if they chose to, and a payment of approximately €385000 ($500000) to agricultural interests.

The final Judgment: (1) Established a Basin Watermaster to oversee groundwater pumping in the Basin and to administer the legal Judgment, (2) Established a Safe Yield for the Basin, which is the amount of groundwater that can be produced from a subarea on a long-term basis without a reduction in the amount of water in storage and without adverse effects on the resource.

(3) Assigned a Base Annual Production (BAP) amount to each major groundwater producer. The BAP is a right to pump a certain amount of water every year in perpetuity. (4) Assigned a Free Production Allowance (FPA) to each major groundwater producer. The FPA is based on the BAP for the producer and the total for the subarea. The FPA is the amount of groundwater that a major producer can pump in a given year without paying a penalty fee to the Mojave Water Agency. (5) Established a class of "minimal users" who pump only minor amounts of groundwater and who are not subject to the Judgment.

The Judgment requires that the FPA for each producer be reduced over time so that the total amount of water pumped in a subarea is consistent with the Safe Yield of the subarea. Eventually, the total FPA for each subarea must approximately equal the safe yield of the subarea. The FPA reduction is controlled by the Watermaster. Each year, the Watermaster reviews the difference between the amount of groundwater pumped in a subarea and the Safe Yield of the subarea. If the amount of production is greater than the Safe Yield, the FPA is reduced by 5% for each major groundwater producer. The reduction in FPA is reviewed every year and the FPA is reduced until a maximum reduction to 60% is reached. Figure 1 summarizes this reduction in FPA for the Alto Subarea. As of 2005, the Alto Subarea FPA had been reduced to 60% of its original BAP. In the future, the FPA may be further reduced. Currently, the FPA is approximately the same as the Safe Yield for the Alto Subarea, helping to bring the subarea demand into balance with the long-term water supply.

The Judgment recognizes that some major producers may pump more groundwater than allowed by the Judgment. For example, this might occur if a water district has a significant increase in the number of people it serves. If a major producer pumps more groundwater than his FPA allows, he can do several things.

1) He can purchase unused FPA from another producer. For example, if a major producer such as a city or water district pumps more water than allowed by his FPA, he can purchase water from a farmer in the same subarea who did not use his entire FPA. A purchase of this type is only good for one year's pumping, and purchases must be made every year that the FPA is exceeded.

2) He can buy a certain amount of BAP from another producer. Once a certain amount of BAP is purchased, it can be used to pump water in perpetuity. This type of purchase costs more, however, it represents a permanent right to pump, not just a right for a single year.

3) He can pay the Watermaster a fee for each AF (1230 m^3) pumped in excess of his FPA. The Watermaster will then take this money and use it to purchase imported SWP aqueduct water and recharge it in the appropriate subarea. For the 2004-2005 water year, the payment was \$262/AF (€164/1000 m^3).

3.2 Water market established

The Judgment established a method for the transfer (buying and selling) of both FPA and BAP within each subarea of the Mojave River Basin. The Watermaster tracks these transfers. When a major producer exceeds his FPA and does not purchase FAP or BAP from another producer, the Watermaster collects the fee required for overproduction [8, 9].

During the ten years since the Judgment was first established, there have been many changes in the water market. As anticipated by the Court, overproduction by some users has resulted in the purchase of FPA and BAP from other users in each subarea. These transfers have been tracked by the Watermaster. The number of FPA and BAP transfers has been largest in the Alto Subarea where the population growth and intensive use of the groundwater aquifers is the greatest. In the 2004-2005 water year, there were more than 240 transfers in the Alto Subarea. For this reason, the discussion below focuses on the Alto Subarea.

Three main conditions have affected the number of transactions and the amount of money paid for both FPA and BAP rights. These conditions, described below, are: 1) the reduction in FPA, 2) a shift in water use from agricultural to municipal and industrial use, and 3) increased stability in the water market as challenges to the Judgment have been settled.

The reduction in FPA as shown on Figure 1, means that the amount of FPA available to be bought and sold through the water market is declining. At the same time that the FPA is being reduced, the future water demand will increase. The recent and future water demands are shown on Figure 2. Figure 2 shows the percentage of agricultural and municipal and industrial (M&I) pumping of groundwater in recent years. In 1996, when the Judgment was first issued, agricultural pumping was 60% of the total amount of groundwater pumped. M&I use was 40%. Also in 1996, approximately 50% of the total FPA for the Alto Subarea was not used.

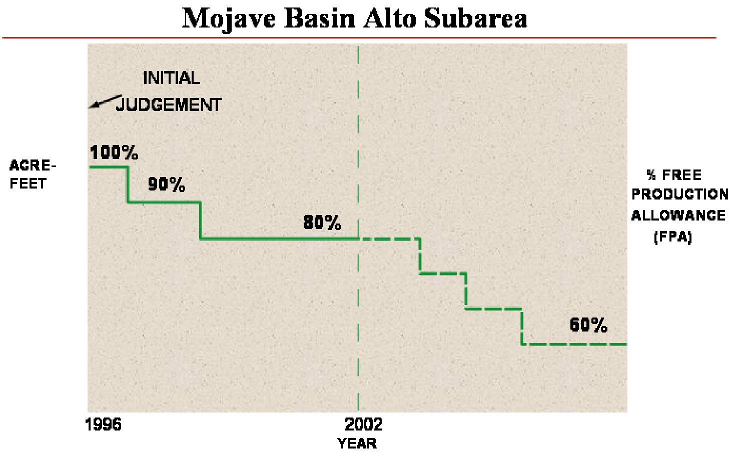

Figure 1: Reduction in free production allowance.

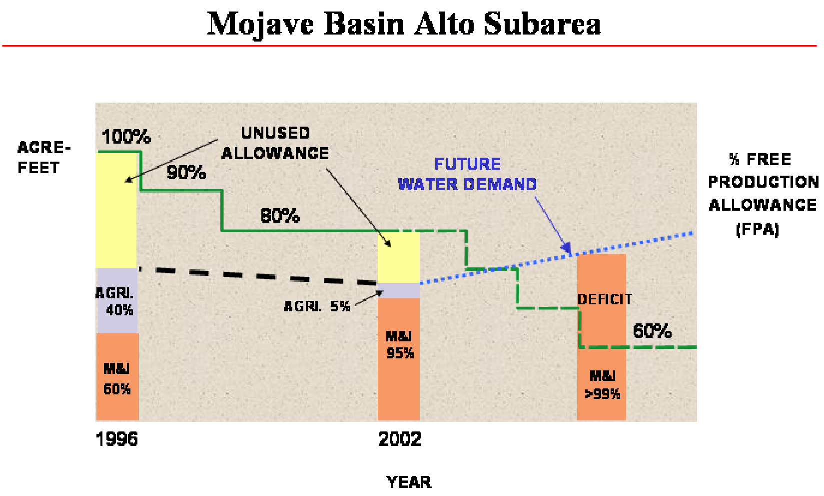

Figure 2: Shift in use and increased demand.

By the year 2002, agricultural pumping had declined to only 5% of the total pumpage Pumping for M&I use had increased significantly, as the result of population growth, to 95% of the total amount of water pumped. The amount of unused FPA has declined to approximately one quarter of the total allowed FPA for the Alto Subarea. The shift in water use from agricultural to M&I is expected to continue into the future as the cities in the area grow, and as farmers reduce the size of their farms and sell their FPA or BAP allowances to other users. In the future, M&I water use will most likely be greater than 99% of the water demand in the Basin. This trend suggests that the Judgment is working as planned, to allow farmers to reduce their groundwater pumping and sell their unused FPA or BAP. This allows farmers to make a profit on their water rights if they would like to.

As shown on Figure 2, since 2002, as the FPA was reduced to 60% of its original amount, the water demand has been greater than the FPA. This results in a deficit, where there is not enough FPA available for the major producers. The reduced availability of FPA results in increased competition for FPA and BAP rights.

Figure 3 shows the cost of both FPA and BAP in 1996 and in 2002. At the time of the initial Judgment (1996), a major producer could buy FPA for approximately \$30/AF (€24,3/1000 m^3) and a BAP right for \$800 to \$1000/AF (€650 to €770/1000 m^3). In 2002, the cost for buying FPA had almost tripled to \$80/AF (€64,8/1000 m^3) and BAP rights increased to \$1350/AF (€850/1000 m^3). In 2005 the cost had increased even more with BAP right as much as \$2500/AF (€1570/1000 m^3).

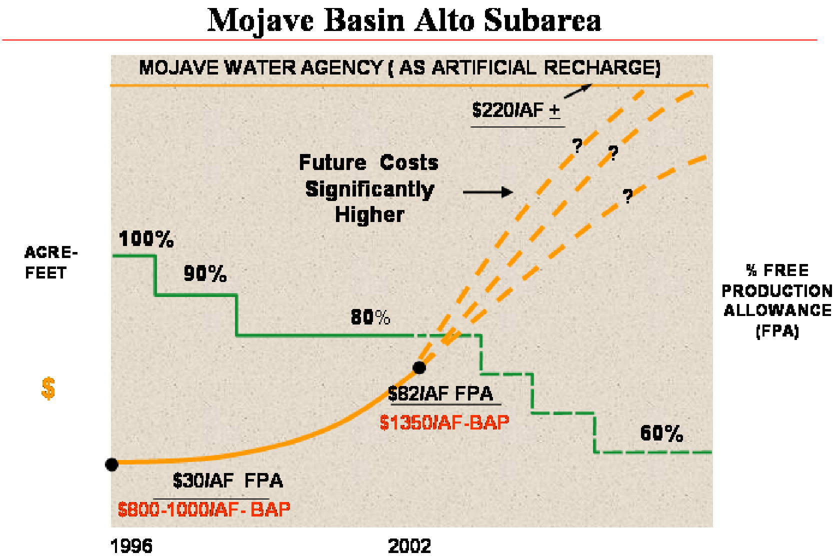

Figure 3: Increasing FPA and BAP prices.

In the past, if a major producer pumps more than his FPA, it has been cheaper to purchase unused FPA from another major producer rather than to pay the Watermaster an even higher fee. As a result, the MWA had only received a very few payments for purchasing water from the California SWP aqueduct and recharging it into the groundwater basin. However in recent years, the Watermaster has been able to collect more money and has begun recharging larger amounts of SWP water.

In the future, however, as shown on Figure 3, the water demand will exceed the available FPA and future costs will be significantly higher. The cost for one major producer to buy FPA from another major producer will increase until it consistently reaches or exceeds the same price as the Watermaster fee. Figure 3 shows the 2002 cost of $220/AF (€180/1000 m^3). At that time, the costs for buying FPA from other producers was less than the Watermaster's fee.

Currently and in the future, when FPA prices are too high, major producers will begin to pay the Watermaster for imported water. At that time, the Watermaster will have the funds to buy imported water and artificial recharge to the Basin will increase. The Mojave Water Agency, who acts on behalf of the Watermaster, has an imported water entitlement of up to 75800 AF/year (94 mm^3 per year) from the SWP. The SWP aqueduct crosses part of the Alto Subarea and water can be purchased, up to the entitlement amount, and recharged to the aquifer. This artificial recharge will be an important source of water supply to the overdrafted and intensively used aquifers in the Basin. The 94 mm^3 of water that may be potentially recharged would increase the Safe Yield of the Basin from approximately 110 mm^3 to more than 200 mm^3 per year, almost doubling the Safe Yield of the basin. The increased availability of water will come at a price, as users must pay more for the right to pump water. As water costs increase, however, the Mojave River Basin water market will work to make water use more efficient; benefiting all water users in the Basin.

4 Conclusions

As the growing population in Southern California places increasing demands on local and imported water resources, the cost of water supplies will continue to increase. In order to maintain reliable and reasonably priced water supplies for their customers, water managers will need to look toward increased integration of imported, surface water, and groundwater supplies, including wastewater reclamation and seawater desalination. Factors such as climate change and population increases in Northern California and other parts of the American Southwest will also place greater demands on available water supplies and the amount of imported water available to Southern California.

Southern Californians have a long history of innovative approaches to meeting water supply challenges; including conjunctive use of surface and groundwaters, sea water intrusion barriers to protect aquifers, and seawater desalination. Creative solutions such as water markets and legal management of scarce resources will continue to increase water use efficiency while providing reliable cost-effective long term water supplies.

References

[1] Hundley, N. Jr., 2001, *The Great Thirst*, University of California Press, pages 465-466.
[2] Hundley, N. Jr., 2001, *The Great Thirst*, University of California Press, page 466.
[3] Hundley, N. Jr., 2001, *The Great Thirst*, University of California Press, page 467.
[4] California Department of Water Resources, 2005, *Management of the California State Water Project*, Bulletin 132-04, September.
[5] Metropolitan Water District of Southern California, 2006, *Regional Urban Water Management Plan*.
[6] Municipal Water District of Orange County, 2001, Water Rates, Water System Operations, and Financial Information, 2001.
[7] Black & Veatch Company, 2006, *2006 California Water Rates Survey*.
[8] Mojave Basin Watermaster, 2006, *Watermaster Annual Report for Water Year, 2004-2005*, April 1, 2006.
[9] Mojave Water Agency, 2002, *Summary of the Judgment after trial*, www.mojavewater.org. and *Regional Water Management Plan Update, Phase 1 Report*, June 2002.

Water transfer from Basilicata to Puglia: a technical, economic and institutional challenge

M. R. Mazzola & G. Scordo
Dipartimento di Ingegneria Idraulica ed Applicazioni Ambientali, Università di Palermo, Italy

Abstract

The necessity of supplying Puglia by transfer of water resources from Basilicata (250 Mm^3/year mainly for civil use but also for agricultural and industrial uses) and another bordering region (Campania for 200 Mm^3/year only for civil use) arises from the scarcity of water resources inside Puglia, where precipitation is very low (400 mm/year), rivers are few and with limited watersheds and costal aquifers are overexploited with salt intrusion. Moreover recent droughts have pointed out the need for revising water supply policy in Puglia, looking for the increment of the transfer of water resources from Basilicata (50 Mm^3/year) and Campania (32 Mm^3/year) and the development of feasibility studies of new supply options from other nearby regions (Abbruzzi and Molise) and/or another country (Albania). In this paper this plan and all these alternatives are described. In Italy the art.17 of the Galli Law, enforced in 1994, states that the deal between two regions for the transfer of water resources has to take into account not only the technical definition of the project, but also the environmental and social impacts, and the institutional assessment for the government of the resources and the management of the infrastructures. The recent application of the Water Framework Directive of the European Community, based on the principles of full cost recovery and polluters pay, introduces in this process the need for defining not only the financial costs of the proposed projects but also the environmental and the resources costs. In the paper the deal between Puglia and Basilicata for the management of existing and proposed infrastructures for the transfer of water resources is also described, which represents the first application in Italy of Galli Law (art.17) and of the principles of the EC Directive.
Keywords: water supply, water transfer, EC Directive, Galli Law.

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1 Present and forecasted water demands

The problem of water supply of Puglia region is not new, owing to the very low precipitation (yearly mean value of 400÷450 mm) and the presence of more than 4,0 million of inhabitants, large cultivated areas and important industrial areas with high water consumption firms.

The present water supply for civil uses in the region is 540 Mm^3/year but the actual consumption is lower, because water losses are very high. A large program for water loss reduction has recently started by the Acquedotto Pugliese SpA, which is the water company with the concession of water and wastewater services till 2018. The objective of this program is the reduction within 5 years of total losses to a level 30÷35%, and the forecasted gross civil demand in year 2032 is 500,8 Mm^3/year and the net demand is 383 Mm^3/year, with per capita consumption of 250 l/d.

As a consequence of lack of alternative water supply, the pressure for irrigation and civil uses has caused the overexploitation of coastal aquifers in Puglia, and one of the most important objective of the new water supply planning is to reduce groundwater withdrawals, implementing water saving irrigation methods, reusing wastewaters and replacing private wells with collective irrigation systems supplied by reservoirs. Actually the extension of collective irrigated districts is 75,517 ha with water demand of 199 Mm^3/year, while the private irrigated areas are extended 286,885 ha with water demand of 362 Mm^3/year. Starting from the present state, for the year 2032 an increment of the extension of collective irrigated districts to 117,911 ha has been stated with water demand of 301 Mm^3/year, while the private irrigated areas do not change, and the total irrigated areas become 404,796 ha with water demand of 663 Mm^3/year.

The present industrial demand for the whole region is 142 Mm^3/year, with a strong concentration in the industrial area of Taranto, with water demand of 79 Mm^3/year, partially supplied by reservoirs. The forecasted demand does not change considerably from the present one.

2 Present water supply systems of Puglia

The Puglia region is supplied mainly by water systems transferring resources from the bordering regions Campania, Basilicata and Molise (Fig. 1), rich in springs and rivers, while in Puglia it exists only the down stream of Ofanto river, whose upper stream is located in Campania and Basilicata. The only water supply sources located in Puglia are the coastal aquifers, which are overexploited as stated before. The three large water systems which transfer resources from the other regions are [5]:

- the Ionico-Sinni;
- the Ofanto-Sele-Calore;
- the Fortore.

The sources of *the Ionico-Sinni water system* (Fig. 1) are located in Basilicata and it supplies civil, agricultural and industrial users of *Basilicata* and *Puglia*, and to a less extent also of *Calabria*. The system is supplied by the flows of the rivers *Sinni*, *Agri*, *Basento* and *Bradano*, regulated in these reservoirs, with the following regulation capacity K (Mm^3), mean inflow I_M (Mm^3/year), inflow with return time of 5 years $I_{0.20}$ (Mm^3/year) and with return time of 20 years $I_{0.05}$ (Mm^3/year) (see table 1).

Table 1: Hydrologic features of Ionico-Sinni water system.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Monte Cotugno	433.0	347.8	247.3	158.8
Pietra del Pertusillo	142.0	279.8	179.6	109.7
Camastra	23.7	117.3	81.0	55.8
San Giuliano	90.2	115.0	42.1	13.7

The comparison among I_M, $I_{0.20}$ and $I_{0.05}$ shows the strong variability of inflows and the possibility of severe droughts.

The Sinni sub-system aqueduct (186 km long; diameter 3000 mm) transports raw water, and the resources for civil uses are treated in the "Parco del Marchese" plant (capacity of 6,0 m^3/s), located in Puglia close to the border with Basilicata. The release in a normal year (1999) has been 337,01 Mm^3, shared in this way among the different users (see table 2).

Table 2: Resources supplied by Sinni water sub-system.

	Civil	Agricultural	Industrial
Basilicata users	12.79	149.92	
Puglia users	113.63	32.38	18.02
Calabria users	3.48	6.79	

The resources regulated in the "Pietra del Pertusillo" reservoir for irrigation are released downstream, while those for civil use are treated in the Missanello plant (capacity of 3,6 m^3/s) located close to the reservoir and supplied by the discharge of a hydroelectric power plant. The Pertusillo sub-system main pipeline is 90 km long (diameters 1900÷2200 mm) till the border with the Puglia region and after divides into two other pipelines supplying central (111 km; diameters 1000÷1900 mm) and southern Puglia (164 km; diameters 1200÷1900 mm). The release in a normal year (1999) has been 154,15 Mm^3, shared in this way among the different users (see table 3).

The release of the *Camastra sub-system* in a normal year (1999) has been 14,02 Mm^3, shared in this way among the different users (see table 4).

The release of the *San Giuliano sub-system* in a normal year (1999) has been 42,90 Mm^3, shared in this way among the different users (see table 5).

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Table 3: Resources supplied by Pertusillo water sub-system.

	Civil	Agricultural	Industrial
Basilicata users	3.15	42.79	
Puglia users	108.21		

Table 4: Resources supplied by Camastra water sub-system.

	Civil	Agricultural	Industrial
Basilicata users	13.15	0.75	0.22

Table 5: Resources supplied by San Giuliano water sub-system.

	Civil	Agricultural	Industrial
Basilicata users	22.90		
Puglia users	20.00		

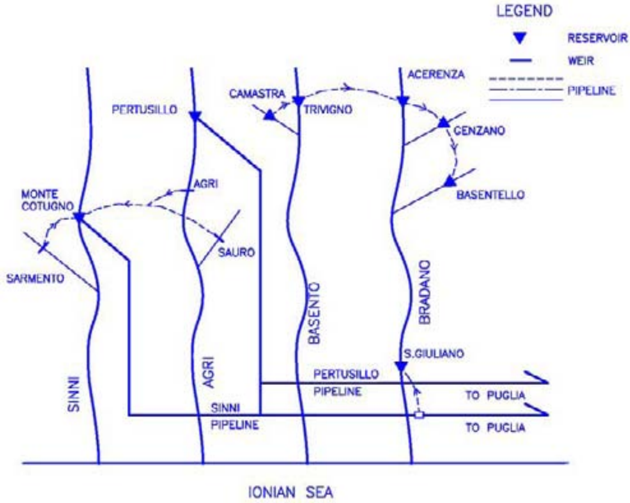

Figure 1: Ionico-Sinni water system.

The sources of the *Ofanto-Sele-Calore water system* (Fig. 2) are located mainly in Campania and also in Basilicata and Puglia and it supplies civil, agricultural and industrial users of Basilicata and Puglia, and to a less extent also of *Campania*. The system is supplied by the spring groups of "*Caposele*" and "*Cassano Irpino*" in *Campania*, and by the flow of the river *Ofanto*, whose watershed is interregional (Campania, Basilicata and Puglia). The mean diverted flow of the two spring groups is 155 Mm^3/year, while the mean flow of the *Ofanto* river diverted by the *Santa Venere* weir is shown in table 6.

Table 6: Resources features of Santa Venere weir.

	I_M	$I_{0.20}$	$I_{0.05}$
Santa Venere	230.4	165.1	112.0

The *Sele-Calore sub-system* aqueduct has been built more than 80 years ago, its principal canal is 244 km long with tunnels more than 100 km long. It supplies only civil users, mainly in Puglia (147,2 Mm^3/year) but also in *Campania* (1,2 Mm^3/year) and Basilicata (6,3 Mm^3/year).

The water resources diverted by *Santa Venere* weir are regulated in these reservoirs (see table 7).

Table 7: Hydrologic features of reservoirs of Ofanto water system.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Rendina (Basilicata)	20.6	55.6	30.6	18.6
M. Capaccioni (Puglia)	50.0	2.3	0.7	0.2
Locone (Puglia)	105.0	11.8	5.2	3.3

The resources diverted in a normal year (1999) have been 90,92 Mm^3, shared in this way among the different users (see table 8).

The resources for civil use from the *Locone reservoir* are treated in the closed homonymous plant (capacity of 1,5 m^3/s) and transported to central Puglia by a pipeline 86 km long and with diameters in the range 2400 ÷ 1600 mm.

The sources of *the Fortore water system* (Fig. 3) are located in Puglia and Molise and it supplies civil agricultural and industrial users in Puglia. The system is supplied by the flow of the Fortore river, regulated in the *Occhito* reservoir (see table 10).

The release of the *Fortore* system in a normal year (1999) has been 190 Mm^3, shared in this way among the different users (see table 9).

Table 8: Resources supplied by Ofanto water sub-system.

	Civil	Agricultural	Industrial
Basilicata users		23.23	3.95
Puglia users	15.00	48.72	

Table 9: Resources supplied by Fortore water sub-system.

	Civil	Agricultural	Industrial
Puglia users	51.0	129.0	100.0

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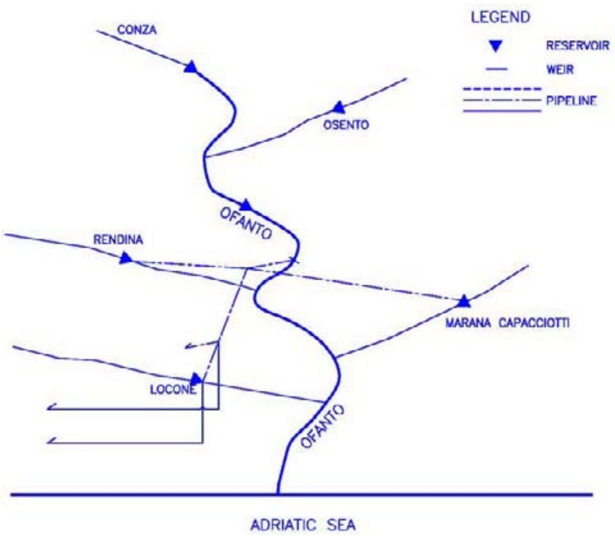

Figure 2: Ofanto water sub-system.

Table 10: Hydrologic features of reservoirs of Fortore water system.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Occhito	247.5	137.3	94.2	60.5

The resources for civil use from the *Occhito* reservoir are treated in the closed *Finocchito plant* (capacity of 2.4 m^3/s) and transported to northern Puglia by a pipeline 40 km long and with diameter of 1600 mm.

3 The development of the water supply systems

The analysis of the demand pattern for the different uses till 2032 and the updating of the hydrologic forecasting recently developed have shown the opportunity of implementing new infrastructures to guarantee supply also during intense drought periods. Some of these infrastructures are under construction or have already been designed and their construction will start in few months, while others infrastructures are in the feasibility study stage.

The Ionico-Sinni water system supply will be increased in few years when the tunnels diverting Basento river streams down the Camastra reservoir and at the Trivigno wear section to the Acerenza and Genzano reservoirs located in the Bradano river watershed will be completed (see table 11).

The optimisation of the operation of these reservoirs, along with the other two located in Bradano watershed (*Serra del Corvo* and *San Giuliano*), will allow not only to widen the irrigation areas along the valley and close to the coast, but also to increase the supply security for the whole system.

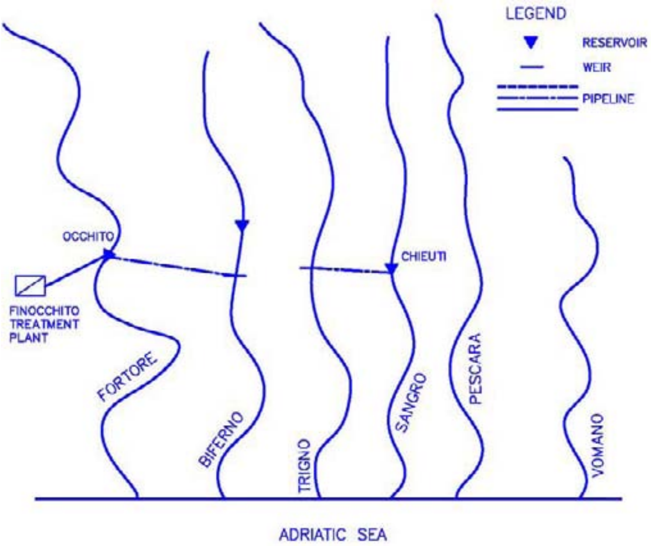

Figure 3: *Fortore water system.*

Table 11: Hydrologic features of Trivigno weir and Acerenza, Genzano and Serra del Corvo reservoirs.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Trivigno	0.0	116.6	80.4	46.6
Acerenza	38.5	19.1	9.5	4.0
Genzano	52.9	5.4	2.3	0.8
Serra del Corvo (Basentello)	28.1	33.1	14.6	6.0

Always in the Ionico-Sinni water system, also the water resources regulated in *Monte Cotugno* reservoir will be increased in few years, along with the supply security, when the construction of the weirs and the tunnels diverting into this reservoir the flows of rivers *Sarmento*, *Agri* at a section downstream of *Pertusillo* reservoir and *Sauro* will be completed. These weirs have (see table 12):

Table 12: Hydrologic features of Sarmento, Afri and Sauro weirs.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Sarmento	0.0	106.7	63.5	40.3
Agri	0.0	107.5	71.3	38.5
Sauro	0.0	46.1	30.1	17.7

Another feasibility study proposed to increase the supply capacity of *Monte Cotugno* reservoir is to release downstream the water resources stored in the *Cogliandrino* reservoir located in the upper watershed of *Sinni* river, and presently diverted to the *Noci* river for hydroelectric energy production at the Castrocucco plant (see table 13).

Table 13: Hydrologic features of Cogliandrino reservoir.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Cogliandrino	10.1	120.1	79.0	42.8

Some hydroelectric power plants in series have been proposed to partially compensate the loss of energy production. In any case the possibility of increasing water transfer from *Monte Cotugno* reservoir to irrigation areas and to *Puglia* is limited to 50 ÷ 60 Mm^3 by the size of existing pipeline, and to overcome this limit it is necessary another pipeline, with strong environmental and social problems.

The Ofanto-Sele Calore water system supply will be increased in the next years by the resources regulated in the *Conza* reservoir located in *Campania* (see table 14).

Table 14: Hydrologic features of Conza reservoir.

	K	I_M	$I_{0.20}$	$I_{0.05}$
Conza	63.0	90.7	62.1	40.2

The new treatment plant will have a capacity of 1,5 m^3/sec, and a yearly volume production of 32 Mm^3 distributed in the central *Puglia* by an already existing 175 km long pipeline, with diameters in the range 2400÷1600 mm.

To increase water supply of *the Fortore water system* supply for northern *Puglia* a feasibility study of the diversion from *Biferno* river in *Molise* to *Occhito* reservoir has been carried out (Fig. 3). It has shown the possibility of transferring 20÷25 Mm^3/year with limited environmental problems.

Always for the same system, other two very complex projects have been proposed. The first one is the diversion of water resources from *Sangro* and *Trigno* rivers, whose watersheds are in *Molise* and *Abruzzo* regions, regulated in the existing *Chieuti* reservoir located in Molise (K = 112,3 Mm^3) to *Finocchito* treatment plant and *Locone* reservoir, by pipelines 163,7 km long and with diameters in the range 3000÷1000 mm. The preliminary feasibility study should define amount of water to divert, costs and environmental effects to verify the possibility of an agreement among *Puglia*, *Molise* and Abruzzo.

The other project is the transfer of 200÷260 Mm^3/year from *Sangro*, *Pescara* and *Vomano* rivers in *Abruzzo* to the *Finocchito* treatment plant by three pipelines 125,7 km, 165,9 km and 191,5 km long in a large part submarine. The project is expansive, also because it is necessary to increase the capacity of water treatment plant and to realize new pipelines to transport resources also to central *Puglia*.

Furthermore the preliminary analysis of potential irrigation demand in northern *Puglia* and of environmental effects are key points for the evaluation of project benefits and costs and the subscription of the agreement between *Puglia* and *Abruzzo*.

Another proposed project for the supply of *Puglia* is a pipeline 212 km long (85 km submarine) transferring 150 Mm^3/year from *Albania* to southern *Puglia*, but also in this case a preliminary feasibility study, with cost-benefit and alternative analyses, should be carried out before going further in the project.

Since many of these projects are long term ones, the *Puglia* administration has analysed the possibility to build three desalination plants to reduce the negative effects of long droughts as that occurred in the 2000÷2002 years. These plants should be located close to the cities of Bari (20 Mm^3/year), *Brindisi* (20 Mm^3/year) and *Taranto* (18 Mm^3/year). The first two treat sea water, while the third treats brackish spring water.

4 Institutional framework of interregional agreements

As stated before the development of interregional water transfer projects involves not only technical aspects but also institutional and economic ones. The institutional framework of interregional water transfers in Italy is stated in the article 17 of the national Law n. 36 enforced in January 1994 [11], concerning the reorganization of civil water services, and also in the latest environmental legislation [4]. The law states that, when in the river basins planning the analysis of water demand-supply balance could imply water transfers among different regions and river basins, the concerned Regions and River Basins Authority promote Program Agreements, which are approved by the National Government. The interregional water transfer infrastructures are defined of national interest and State can financially support investment and operation costs. In each agreement are defined the criteria and procedures for the realization and management of proposed measures or infrastructures, which should respect in any case the principle of environmental, economic and social sustainability. The feasibility studies of measures and infrastructures proposed in the agreement should contain [3]:

- present and forecasted demand-supply analysis for the different uses;
- energetic state balance of water resources before and after the transfer;
- analysis of resources quality state before and after the transfer;
- environmental impact evaluation;
- financial and economic benefit – cost analysis.

In the agreement should be also defined the financial plans for the realization of proposed measures and infrastructures, with the identification of public and private funds, and subjects involved in their realization and management.

The concepts of article 17 of Law 36/94 have been confirmed in the EC Water Framework Directive 2000/60, which has introduce the following principles:

- Integrating water resources management at the hydrographical district scale, taking into account at the same time the qualitative aspects of surface and ground waters to reach a satisfactory level of environmental protection
- Analyzing the characteristics of the hydrographical district, the impact of human activities and developing the economic analysis of water resources uses.

The economic analysis is based on the principles of “polluter pays” and “full cost recovery” of water services, also if each State could subsidize partially the costs for some users, like the agricultural ones, but the social reasons of this policy should be clearly stated in the management plans. Three main components of costs are defined [6, 7],

- Financial costs include the costs of providing technological services and the costs of administering these services; they include operation and maintenance costs, capital maintenance costs, capital cost (principal and interest payment, and return on equity where appropriate)
- Environmental costs represent the costs of damage that water uses impose on the environment and ecosystems and those that use the environment (e.g. recreation)
- Resource costs represent the costs of foregone opportunities, which other uses suffer in the present and in the future due to the depletion of the resource beyond its natural rate of recharge or recovery

The recovery of costs should be fair and transparent and it has to be to be disaggregated into at least industry, households and agriculture. Each country should define water pricing policies which stimulate users within 2010 to consume efficiently water resources contributing to the environmental objectives of the Directive. Within the same date civil, agricultural and industrial users have to be “fairly” charged of the full costs of water services, but the member States could be take into account the social, environmental and economic aspects of recovery, as well as geographic and climate regional conditions.

T he enforcement framework of these concepts has been recently developed by the European Commission [9], and there is a vast literature on the different aspects of this subject [2, 10, 12, 13].

5 Institutional and economics aspects of the Puglia-Basilicata Program Agreement

According to article 17 of Law 36/94 and following the principles of EC Directive 2000/60, of which at that time a draft was already known, the regions Puglia and Basilicata and the Ministry for Public Works signed on the fifth of August 1999 the Program Agreement concerning the management of the water resources transferred from Basilicata to Puglia by the Ionico-Sinni water system [1]. This agreement is the first application in Italy of this legislation and it is probably the most complete example not only for the system technical complexity but also because the infrastructures already built have strong environmental and social impacts.

The agreement is based on the cooperation between the two regions and its objectives are stated in this way:

- definition of the water resources balance and planning of the system till 2015;
- identification of existing infrastructures of common interest forming the raw water system;
- establishment of permanent cooperative executive body for the planning of actions and monitoring of the Program Agreement;
- reform of the existing authorities and public water agencies;
- definition of raw water tariff, including financial, environmental and resource costs;
- implementation of actions of water demand control, reducing losses and wastewater reuse in the different uses and of environmental control of coastal areas for retrogradation and aquifer salt intrusion;
- identification of measures and infrastructures to complete and to improve the existing water system;
- development of feasibility study to increment system water supply and transfer to Puglia;
- definition of guidelines for water resources management and priority of reduction measures for the different users during droughts.

The technical aspects of this agreement have been described in the previous sections of this presentation. In this section we focus our attention on the problem of institutional cooperation, reform of authorities and agencies and raw water tariff, looking at the actual state of application of the Agreement.

The article 5 of the Agreement provides the institution of the Authority of the Government of Water Resources, made up by the Minister of Public Works and the Presidents of the Regions Puglia and Basilicata or their delegates, with the participation of the Secretary of River Basin Authorities competent for the water system. The Authority is chaired alternately by one of the President of the two regions and among the other tasks at the end of February in each year it verifies the resources stored in the reservoirs in order to define the releases for the year for the different users and eventually the demand reduction measures to face shortages. The Authority has been constituted after the signature of the Agreement and it worked really well during the long and severe drought of years 2001-2002, smoothing the negative effects of shortages on final consumers.

In the article 7 of the Agreement the two Regions undertake the engagement of reforming the organization of the existing River Basins Authorities reducing the number to only two Authorities whose boundaries are consistent with water system boundaries. These reforms have been carried out and now the Basilicata River Basin Authority includes the Ionico-Sinni water system, while the Puglia River Basin Authority includes the Ofanto water sub-system.

In the article 13 of the Agreement and in the quoted Attachment 5 the two Region and the Ministry agree on the reform of existing water public corporations (Acquedotto Pugliese for civil services and EIPLI for raw water services) in three new stock companies, two for the civil water services in the regions and one for the management of common raw water system. At the present time the two companies for the civil water services have been started, that is Acquedotto Pugliese S.p.A. in Puglia and Acquedotto Lucano S.p.A. in Basilicata. Instead for the management of raw water system the reform of EIPLI has not been carried out by the Ministry of Agriculture, and in the meantime the Basilicata Region has started a new stock company (Acqua S.p.A) to manage and to develop the whole regional system supplying raw waters.

The shareholding of this society is open to other bordering regions, and in any case the control will remain public because the main function of this society is the minimization of the financial costs of raw water system and moreover the society could cover the needs of technical assistance of River Basin Authorities and Regions for the definition and implementation of water resources policies. To avoid the possible distortions of the natural monopoly in which this society operates, its structure should be very light and well qualified, and market competition is introduced by contracting-out the operation and maintenance services and realizing the new infrastructures by contracts for public works and project finance when it is convenient.

Also if in the Attachment 3 of the Agreement, which defines the water resources government policies and the programmatic guidelines, is underlined the opportunity of reforming the existing public subjects charged of irrigation and industrial water services, that is Land Reclamation Agencies and Industrial Development Agencies, nothing has been done in practice to implement their efficiency. The unsuccessful reforms of these subjects and EIPLI and the possible clash of competences of EIPLI and Acqua S.p.A are weak points in the process.

The article 15 of the Agreement has stated that the total production costs of raw water had to be established by the Authority of the Government of Water Resources within the end of year 2000. These costs have to take into account:

- financial costs;
- environmental costs;
- resources costs.

As stated in the article 6 of the Agreement a working group has been charged to propose how to apply these concepts. It produced in July 2002 a very valuable document from the theoretical point of view, but with limited applicability for lack of information and uncertainties.

As a matter of fact information on present operation and maintenance costs are incomplete and partially unreliable, and the estimated value of 0.021 €/m^3 may be significantly different from the real one; consequently the definition of these costs has been postponed.

Environmental costs have been estimated as a percentage of the following costs:

- maintenance of watersheds upstream the reservoirs (50%);
- maintenance of hydrographical networks upstream the reservoirs (50%);
- tertiary level in the wastewater treatment plants upstream the reservoirs (30% of the total treatment costs);
- quantitative and qualitative monitoring (60%);

- maintenance of hydrographical networks downstream the reservoirs (20%);
- defence from hydrogeological risks downstream the reservoirs (50%);
- protection from erosion of Ionian coast (60%).

The estimated unit environmental cost is 0,055 €/m^3.

The document introduces the energy recovery costs for Basilicata as a measure of resources costs. Owing to the lack of specific information, as first approximation of the actual one in the document the estimated unit electric energy recovery cost is 0,024 €/m^3, that is equal to the double of yearly irrigation pumping costs in Basilicata, whose value is defined in the fifth paragraph of article 15 of the Agreement, to take into account also the other pumping costs in Basilicata and the loss of potential production of hydroelectric energy.

In the absence of any indication, the document proposes also to estimate the other resource costs as a percentage of 10÷20% of the sum of financial, environmental and energy recovery ones.

The document tackles also the problem of cost sharing among the different users, and proposes to apply mean tariff to civil users, to increment tariff for industrial ones and to reduce it for agricultural users. The proposed subsidies for rural sector are justified for its weakness and the social and environmental benefits induced by irrigation, especially in order to contrast the desertification process in these areas.

The discussion about document proposals has been complicated and long, and only the 25th May 2004 the Authority of the Government of Water Resources has approved a document on water raw tariff which states the following.

- Provisional mean raw water tariff excluding financial costs for years 2003 and 2004 is 0,055 €/m^3 and for year 2005 is 0,075 €/m^3 and it is applied to the water resources assigned to each region; for the years 2000÷2002 the environmental costs of 35 M€ are covered by national funds.
- Each region can differentiate tariffs among different users within the region, and eventually reducing them by regional funds in order to subsidize sectors, like the rural one.
- Document on raw water tariff should be revised in order to reduce estimate uncertainties, and within the end of July 2004 the financial costs should be fixed.

Probably till the end of year 2005 raw water tariff will be not applied for irrigation by both regions, and this is a weak point in the process. In order to respect the EC Directive principle of "fairly" charging of the full costs of water services to each user, it is worth wishing for the application of this principle at least for the financial costs.

6 Conclusions

The need of transferring most of water resources consumed by Puglia from other regions arose till the past century, and many complex water supply systems have been realized. The management of this system is not simple and technical, economic and institutional aspects have to be faced.

The physical and hydrological features of Ionico-Sinni, Ofanto-Sele-Calore and Fortore water systems have been described in the presentation along with the proposed infrastructures to increment availability of water resources and supply security in Puglia. The management of Ionico-Sinni water system is the more complex one, owing to the technical, environmental and social problems connected with the existing storage and transport infrastructures, and the presence of civil, industrial and agricultural users of Puglia and Basilicata.

To solve these problems the two Regions and the National Ministry of Public Works have signed in 1999 a Program Agreement, that is the first application of national regulations on this field. The Agreement activities are also one of the first applications in Italy of the EC Water Framework Directive 2000/60, which introduces economic aspects in the water resources management of a river basin and the need of planning at hydrographical district scale, extending the concept of integrated river basin management.

The experience of this Agreement shows that it works really well in the coordination of the Regions in the water resources management by the institution of the Authority of the Government of Water Resources, with strong benefits especially during severe shortages and more efficient planning of new infrastructures. Positive effects have been partially the reform of the existing water public corporations for civil services, but till now it has failed in reforming the raw water supply public corporation EIPLI, and the existing public subjects charged of irrigation and industrial water services, that is Land Reclamation Agencies and Industrial Development Agencies. The possible clash of competences of EIPLI and of new society Acqua S.p.A is another weak point in this process. The definition of the raw water tariff has been a long and complicated process and only the 25th May 2004 the Authority of the Government of Water Resources has approved a document on provisional water raw tariff for the period 2000-2005. The weak point is that has been postponed the application at the rural sector of the EC Directive principle of "fairly" charging of the full costs of water services to each user.

References

[1] "Accordo di Programma fra Basilicata, Puglia e il Ministero dei Lavori Pubblici (Art.17 L.36/94)", Rome, 1999, in Codice della Difesa del Suolo e delle Risorse Idriche, Collana Ed. di Studi e Ricerche della Autorità Interregionale di Bacino della Basilicata, vol. 1, Potenza, Italy, May 2002

[2] Agencie de l'eau Seine-Normandie, "Proceedings of the Second international workshop on implementing economic analysis in the Water Framework Directive Opening the Black Box ", Paris; February 2005

[3] Decreto Presidente Consiglio Ministri 4 marzo 1996 "Disposizioni in materia di risorse idriche", Rome, Italy, 1996

[4] Decreto Legislativo 3 aprile 2006, n.152 "Norme in materia ambientale", Rome, Italy, 2006

[5] Di Santo A., Ranieri M., "Studi propedeutici all'Accordo di Programma Puglia-Basilicata per la gestione delle risorse idriche, L'Acqua, 2000(2), pp.65÷76

[6] European Commission, " Communication from the Commission to the Council, the European Parliament and the economic and social committee concerning the pricing policies for enhancing the sustainability of water resources", COM (2000)477 final, Brussels, Belgium, October 2000

[7] European Commission, "Commission Staff Working Paper. Water pricing policies in theory and practice. Annex E to the Communication COM (2000)477 final, SEC (2000) 1238, Brussels, Belgium, October 2000

[8] European Commission, " Water Framework Directive 2000/60/CE, Brussels, Belgium, October 2000

[9] European Commission, Working Group 2.6 WATECO, "Economics and the Environment. The implementation challenge of the water framework directive. A guidance document n°1", Luxemburg, 2003

[10] Gibbons D. C., "The economic value of water" Resources for the future, Washington D.C., USA, 1986

[11] Legge 5 gennaio 1994, n.36 "Disposizioni in materia di risorse idriche", Rome, Italy,1994

[12] Pearce D.W., Mourato S., Atkinson G., "Recent development in environmental cost-benefit analysis", Draft, OECD, Paris, France, September 2004

[13] Young R. A., "Measuring economic benefits for water investments and policies", World Bank, technical paper n.338, Washington D.C., USA, 1996

Section 8
Pollution control

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Pollution policies and market approaches in the Olifants River, South Africa

W. L. Nieuwoudt
University of KwaZulu-Natal, South Africa

Abstract

Approximately 3.4 million people live in the Olifants River Catchment in South Africa and a considerable proportion of South Africa's mining, power generation and agricultural activities are concentrated here. Environmental pollution caused by mining activity is a problem. Two pollution policies are proposed; tradable pollution permits and pollution offsets in the river. The catchment surface is fractured by mining activities, and water is drained into underground aquifers which then seep into streams. A main problem is the effluent leakage from old disused mines. Mines have been permitted to release nutrients into the streams during periods of high flow, which is called the "controlled release scheme". During the past few years, river flow was low and sufficient dilution of nutrients was not possible. Mines and power stations had to invest in desalination plants at considerable cost. It is recommended that polluters should pay a discharge tax which is not the case at present. It is further proposed that tradable pollution permits be adopted which are subject to a rule that discharges in the river are only allowed when flow is sufficiently high and that trades may only occur within certain parameters. Apart from a pollution trading program it is suggested that bio-diversity offsets be created to provide incentives for cooperation amongst stakeholders. The problem with the defunct mines is that they leak pollutants all the time including during the period when river flow is low. DWAF (Department of Water Affairs and Forestry) has accepted ownership of these mines but they may not have the technology (which is expensive) to desalinate the effluent. In an offsetting arrangement, incentives can be provided to existing mines to desalinate water from these defunct mines by allowing them to discharge a given amount in the Olifants when the water flow is sufficiently high. The above arrangement will cost the taxpayer nothing while discharge during low flow periods is reduced. A discussion was held with stakeholders of the Olifants River Forum during 2006 and support was received for some of these policy options.
Keywords: water pollution, South Africa, off-sets, tradable pollution permits.

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1 Introduction

In this study environmental pollution is studied in the Olifants River, South Africa with a view to suggest policy options. The Olifants River rises to the east of Johannesburg and flows north-east through the provinces of Mpulanga and Limpopo into Mozambique. Approximately 3.4 million people live in the Olifants River Catchment and a considerable proportion of South Africa's mining, power generation and agricultural activities are concentrated here (McCartney *et al.* [1]). The catchment also encompasses important tourist destinations (such as the Kruger National Park). It is estimated that activities within the Olifants Water Management Area generate 6% of the GDP of South Africa.

The Loskop Dam in this Catchment is the centre of the coal mining and power generation industries (Eskom) in South Africa. These industries generate saline effluent, part of which is discharged into the river system. According to Van Stryp [2] pollution is bad while several mining operations are currently technically breaking the law due to the Department of Water Affairs and forestry's (DWAF's) lack of capacity to enforce quality standards (Lodewijks [3]). Water quality deteriorates if the level of Loskop Dam falls and with lower flow in the river the dilution capacity of the system is compromised. According to Coetzee [4] the main problem in the Loskop Dam is the effluent leak from old disused mines.

Mines act as a collector of groundwater. The catchment surface is fractured from mining, runoff decreases and water is drained into underground aquifers which then seeps into streams (Lodewijks [3]). The Klein Olifants River is an example of pollution by contaminated underground water that originates from mines. Mines pollute water due to the reaction of water with minerals. During the wet period in 1995/6 many mines filled up with water, and started spilling. Desalination plants had to be built because coalmines need to get rid of this water. The quality of the water originating from coalmines is a critical factor (Lodewijks [3]) while mines near Witbank are accused of polluting the underground water Pretorius [5].

Water quality affects agricultural crops such as tobacco and citrus negatively in the Loskop area (Pretorius [5]). This has a negative impact on export of some agricultural products that are chemically tested. Prinsloo [6] also considers algae a problem in this area as sieves are clocked.

2 Controlled release scheme

Presently pollution levels from mines can be brought to the required level by using the assimilative capacity of streams/rivers. A "controlled release scheme" is currently in place that controls the releases of effluent into rivers and dams. During high flow periods, when the assimilative capacity of the system is high, discharges are possible. Golder Africa Associates monitors the discharge scheme. Although this discharge system is the cheaper method, during low river flow sufficient dilution of nutrients is not possible. If the mines had not put in a desalination plant, they would not have been able to continue with operations as no discharge was possible during the recent period of low flow of the Olifants River (Lodewijks [3]).

This controlled release scheme is dependent on stream flow. During high stream flow the release of pollutants may not exceed required quality levels but during low flows assimilative capacity will be too low to absorb pollutants. The challenge of this approach would be the low flow periods that can be of a long duration in South Africa. For instance during the period 2001 to 2006 it was too dry to release any pollutants in the Olifants catchment (Lodewijks [3]). It is an open question whether buying water use entitlements from agriculture and/or transfers from other catchments can be used to increase the assimilative capacity of streams in dry periods. The cost and availability of sufficient water at the required time may cause such an approach non-viable.

3 Economic theoretical considerations

It is suggested in this contribution that economic measures may be used to complement the Controlled Release Scheme of DWAF. Two economic policies are suggested, namely transferable pollution permits and environmental offsets. Transferable pollution permits are a well known mechanism but the problem is that the above market differs from the traditional pollution markets and that rules and safeguards need to be adopted. DWAF so far had concerns with transferable pollution permits. Environmental offsets are also well known in wetland conservation but so far this is a relatively unknown tool in river management. As these techniques are suggested in the recommendations, some theoretical considerations are given.

3.1 Transferable pollution permits

The optimum discharge tax is conceptually indicated by the intersection of two functions. The first function shows as more is polluted the marginal cost of damage increases (marginal cost of one additional unit of pollution released). The second function shows that as more pollution is eliminated the marginal cost of elimination increases (marginal cost of one additional unit of pollution controlled). Marginal cost functions are opportunity cost functions which are by definition subjective and not observable. It is thus not possible to calculate an optimum discharge tax using econometric tools to a high degree of accuracy. The optimum discharge tax will also vary along the river as is the case with water prices in different water markets along a river, making estimation of the optimum tax impossible.

3.2 Wetland offsets

The concept of wetland offsets will be introduced briefly to show that this arrangement has a scientific foundation and that it could be adopted to provide incentives to stakeholders to reduce pollution in the Olifants River Catchment.

A market for bio-diversity credits has developed in 20 states in the USA where wetlands have been constructed by some developers who then sell an offset right to others who want to drain wetlands (Randall and Taylor [7]). The authority can require the developer to make onsite offsets while in some instances it might be more beneficial to require the offset to be implemented offsite. The concept of "no net loss" in section 404 of the Clean Water Act allows individuals who wish to drain wetlands in one location to mitigate the loss by enhancing wetlands elsewhere within the same hydrological or ecological region. This trading arrangement has been proposed in South Africa for biodiversity offsets by De Wit [8] and Dickens [9]. For more information on these trading schemes the reader is referred to Randall and Taylor [7], Bjornlund [10], De Wit [8] and Dickens [9].

3.3 Isolation paradox explains the need for institutions in off-set arrangements

The creation of bio-diversity offsets for a river creates the incentive for cooperation amongst stakeholders which may be mines, developers, environmental groups, farmers and public land agencies. For many kinds of ecosystems (wetlands and rivers), protection of bio-diversity requires large areas (scale effect) of contiguous habitat. This is the classic isolation paradox. Supporting institutions need to be created to facilitate cooperation. Situations are often unique but it is proposed that opportunities for enhancing the environment be sought through a partnership between government and stakeholders. Due to the importance of creating institutions to promote offsets this isolation paradox is further discussed.

The isolation paradox explains that institutions are important to deal with environmental problems. Such institutions may be biodiversity offsets in wetlands and rivers. Instruments that make coordinated action in rivers beneficial may be rewarding as it may involve many independent individuals and include a vast area (scale effect).

In order to justify the creation of institutions to deal with biodiversity problems it is important to understand why institutions are needed. Economists have traditionally diagnosed environmental problems as market failures. The markets do not transmit appropriate incentives needed to achieve efficiency. Some have called for government to tax or regulate externalities. Others have argued that allocative inefficiency is caused by incomplete property rights and therefore privatization is the appropriate policy response. The latter group contend that government failure is more pervasive. The merits of these approaches will not be debated here.

The insistence on individual action or none at all can leave every one isolated and ineffective. This class of issues are called isolation paradoxes. Some economists contend that the law has evolved over time to deal with the isolation paradox problem without government interference. Examples are downstream fishermen in England formed an association and have taken upstream polluters to task while class action court cases in the USA are common in environmental pollution. For instance citizens of LA have claimed compensation from air polluters in a class action court case. No single citizen in LA may have enough funds to take the polluter(s) to court and even if the person has the funds he/she does not have the incentive as others will free ride on the outcome.

American law therefore created the institution of class action cases through which many victims can enlist and take offenders to court. This example also explains the scale effect that a large amount of money is involved. In biodiversity trading this scale effect is also important as large areas may be involved and the cooperation of many individuals is important. The institution of bio-diversity trading in water as discussed in this paper will provide the required cooperation to address the problem. Bio-diversity trading in water or offsets as will be discussed will provide the required cooperation to address the problem.

4 Policy options that can be used to reduce pollution in rivers

Markets can be used to provide incentives to stakeholders to reduce pollution. Various options are available that can be used in a complementary fashion.

4.1 Discharge taxes

In terms of Chapter 3 of the National Water Act (NWA) No. 36, the water needs for the effective functioning of aquatic ecosystems must be protected. Ecological sustainability refers to water (quantity and quality) required to protect the aquatic ecosystems of the water resources and ensure their sustainability. Waste is defined in terms of Section 1 (1) (xxiii) of the NWA. The calculation of charges will be based on the registered discharge waste load of salinity and phosphorus, as representing the two most widespread water quality problems in South Africa. The salt load will be estimated using electrical conductivity. Phosphorus (as the limiting nutrient for freshwater eutrophication) will be estimated using soluble phosphorus (phosphate) (DWAF [11]).

DWAF is developing a Waste Discharge Charge System aimed at incentivising polluters to reduce discharge levels. This "polluter pays principle" should become operative in 2008 (Havenga [12]). This system will distinguish between point and non-point sources. At present, discharges in the catchment are not taxed. It is recommended that polluters should pay a discharge tax which must be enforced as they use water from the river in a similar way as abstracting users of water who pay water rates.

4.2 Tradable discharge permits

In a permit discharge-trading market the market price of permits will be determined by the intersection of the functions discussed in section 3.1. It may not be necessary to attempt to calculate an optimum discharge tax. In a water market the market discovers the optimum price of water and participants in the market face the opportunity cost of this price. It is recommended that the same principle should be followed in discharges of pollution and that the optimum price be discovered in a pollution trading market. If polluters have to pay a discharge tax then this will reduce the market price.

Lodewijks [13] recommended a discharge permit trading system but the following problems have been raised by DWAF and others (Lodewijk [3]). Discharges are only possible when river flow is sufficient, while the following must be considered; spatial location of mines relative to one another, and the river network which will drain the effluent into the dams. DWAF had concerns about trading monopolies and that it may affect small stakeholders negatively.

It is important that DWAF's concerns and other concerns be considered and possibly be incorporated as potential recommended rules of such a trading program. Any market has rules, for instance the Stock Exchange has opening and closing hours. A market for discharge permits will also have rules. As pollutants can only be discharged in rivers during high flow times, it is important that this rule be adopted in a permit-trading program.

Another rule in a permit-trading program may be that trade may only take place within well-defined reaches of the river. A water market has similar constraints to minimise externalities. In a water market, trade can only take place from up stream to down stream while in a pollution permit trading program, trade should go the other way as down stream trade increases the concentration of the pollutant at a down stream point.

The Olifants River Forum Stakeholder Workshop near Witbank was attended during 2006 (for list of stakeholders see Olifants River Forum [14]). Gunter [15], one of the representatives of the mines who participated, indicated that mines are definitely interested in discharge permits but thought that it may not be possible in future to obtain them from DWAF. The alternative of building desalination plants is expensive. The cost of the plant near Witbank visited is about R300 million (US$ 42 million) (Gunter [15]). Not all this cost is fixed as the reservoir where the pollutants solidify, fill up after 15 years after which time a new site must be established and the old one is thus abandoned.

The rule that discharge is only possible during high flow is also adopted in the Hunter River Salinity Trading Program in Australia HRSTS [16]. Reason for the adoption of the Australian program is because of conflict between primary producers (livestock and irrigation farmers) and mining. Credits in the Australian program are initially allocated free to license holders based on environmental performance. Two hundred credits are auctioned every two years to replace those retired. New credits have a lifespan of 10 years and a total of 1000 credits are permitted. Auction proceeds are used to pay scheme operating cost (environmental and compliance monitoring cost). Targets are set at 900 micro siemens/cm but it may vary along reaches. Options for industry are to purchase more credits and/or to implement cleaner technologies.

4.3 Offsets can provide incentives to mines to reduce pollution in the Olifants River from abandoned and defunct mines

According to Coetzee [4] the pollution in the Loskop dam in the Olifants River is serious. He further is of the opinion that the main source is the leakage from abandoned old mines (pre-1956) during low flow periods. DWAF has accepted ownership of these abandoned mines. Before the promulgation of Water Act of 1956 an agreement was reached between DWAF and the Chamber of Mines that the liabilities with respect to water pollution of all mines that had ceased production before 1956 would lie with DWAF (Lodewijks [13]).

In an offsetting arrangement incentives can be provided to existing mines to desalinate these defunct mines and in return the existing mines could be provided a concession to discharge a given amount in the Olifants River when the water flow is sufficiently high.

The problem with the defunct mines is that they leak pollutants all the time including during the period when river flow is low. The negative environmental impact is reduced with this off-set arrangement as the pollution during low flow periods is reduced and pollutant is discharged when flow is sufficiently high. Lodewijks [3] supports such an approach. It is recommended that this approach or other offsets be further discussed between DWAF and the mines as other offset arrangements may be decided on. The mines have the technology to desalinate polluted water and have already invested hundreds of millions Rand in this. DWAF may not have the technology while a major part of the significant investment is of a fixed nature. The above arrangement will cost the taxpayer nothing and will promote a more desirable outcome.

4.4 Offsets to mitigate negative environmental impacts of dams

The promotion of water markets in South Africa will reduce the pressure on the construction of new dams. However, the demand for increased storable water is great in South Africa due to increased urbanization and demand from the mining sector. For instance, it is estimated that urban demand will double in the Lower Olifants River Catchment during the next decade (McCartney *et al.* [1]). South Africa has a fast growing urban population which is entirely different from countries such as the USA and Australia as well as Europe. Environmentalists in these countries are concerned about the environmental impact of dams. It appears that in China where urbanization is also high, dams built over-riding local opposition. It is suggested if dams are contemplated in South Africa and if impacts are negative in sensitive ecological areas that offsets be considered to mitigate negative environmental impacts. It may be possible to negotiate with the builders of a dam to eradicate alien vegetation over a stretch in the river or to make other offsets in return for waiving opposition to the construction. If offsets are, however, seen to have scientific international foundation then it is possible to strengthen their institutions and to inform stakeholders that such arrangements are possible. If stakeholders are not aware that these offsets are possible then many developments may not take place because of the opposition to such developments.

Several (potential) offsets in rivers in South Africa will be discussed. Two of the offsets are in the Olifants River Catchment (De Hoop dam and Flag Boshielo dam), while the agreement between the builders of a dam and environmentalists in KwaZulu-Natal can be seen as an offset arrangement. It appears as if these arrangements have taken place in a voluntary bargaining way between stakeholders.

(a) De Hoop Dam located on the Steelpoort River
The building of a dam in the Steelpoort River namely the De Hoop Dam has been approved subject to a final environmental audit (Havenga [12]). The Kruger Park have been opposed to the building of the dam initially. Management in the Kruger Park now seems more supportive of the project given that the minimum river flow is such an important variable to them and that the dam may play a role in augmenting flows particularly in dry periods. Gyedu-Ababio [17] indicated that the Kruger Park might waive concerns about the building of the De Hoop Dam in the Steelpoort River if the Park gets an allocation (say 5%) of the dam's capacity. This is not an official offer and it is not known whether it is intended as a serious statement but as a potential off-set such an arrangement should be pursued.

(b) Flag Boshielo Dam
Raising of the wall of the Flag Boshielo dam increases yield by 18 million cubic meters but eight farms were inundated as a result. As part of an off-set the canal infrastructure of Previous Disadvantage Individual (PDI) farmers downstream of the dam is being upgraded as part of the deal.

(c) Newcastle Dam
The town of Newcastle in KwaZulu-Natal is building a dam for drinking water. It has been established that 18 ha will be damaged (flooded) by construction of the dam. In exchange for flooding 18 ha of a provincial reserve, the proponent purchased more than 1000 ha of the catchment area and set aside funds to manage the remaining area to control invasive plants. The 1000 ha will be handed over to KZN wildlife for conservation (De Wit [8]).

4.5 Privatising the eradication of alien vegetation and offsets of wetland

Mines have bio-diversity action plans in the Olifants River Catchment, for instance a wetlands mitigation program is used whereby a previously destroyed wetland can be rehabilitated in exchange for a concession elsewhere (Lodewijks [3]). Mines intend to eradicate 2500 ha of alien vegetation that will yield 5 million cubic meters of water at a cost of R24.4 million or R4.9 per cubic meter. This appears to be the cheapest (best value) option for harvesting water (Rossouw [18]). There are other plans to obtain 13 million cubic meters of water from eradicating alien vegetation at a cost of R117 million (Rossouw [18]). These private ventures should be encouraged as they have positive social spin-offs.

5 Conclusions

Pollution is an example where markets fail to transmit appropriate incentives needed to achieve efficiency. There are two schools of thought, regulation through the government or a market approach. Neither approach may be effective as the insistence on individual action or none at all can leave every one isolated and ineffective. This class of issues are called isolation paradoxes. This coordination could be provided through biodiversity offsets which could be facilitated through the promotion of the necessary institutions.

The study is based on information collected in the Olifants River Catchment of South Africa during October 2006. The significant mining activity and power generation in the Cathment has polluted this river. Discussions were held with the main stakeholders. It is expected that polluters will be required to pay a discharge tax which is not the case at present. In addition to such a tax it is recommended that tradable pollution permits and bio-diversity offsets be adopted. Tradable pollution permits is an internationally well-known concept but it needs to be adapted for pollution in rivers as discharges should only be undertaken when river flow is sufficient to allow for dilution of chemicals. A main cause of pollution is the leakage into the river from old defunct mines during periods of low river flow. The Government has accepted ownership of the mines but they do not have the expertise or the funds to stop this pollution. It is recommended that the current mines be given an incentive in an offset arrangement. In such an arrangement the mines could be given concessions to discharge a given quantity of pollutant during high flow periods if they reduce the pollution from old defunct mines. Pollution is thus reduced during the low flow period. These offsets are also recommended to mitigate the negative environmental impacts of dams. South Africa has a vast growing urban population and the demand for potable water to supply urban needs will increase.

References

[1] McCartney, MP, DK Yawson, TF Magaglula and J Seshoka. *Hydrology and Water Resource development in the Olifants River Catchment.* Working Paper 76. International Water Management Institute, 2004.

[2] Van Stryp, J. Personal Communication. 10 October 2006. CEO Loskop Irrigation Board. Groblersdal. Tel 013 2623992/5.

[3] Lodewijks, H. Personal Communication. 13 October 2006. Anglo Coal and vice Chairman Olifant River Forum.

[4] Coetzee, J. Personal Communication. 26 October 2006. Mpumalanga Tourism and park Agency. Tel 082 9287543.

[5] Pretorius, K. Personal Communication. 10 October 2006. Dwaf Regional Office. Groblersdal. Tel 013 2626839, Cell 082 8075654

[6] Prinsloo, B K. Personal Communication. 10 October 2006. Farmer on Loskop Scheme.

[7] Randall, A and MA Taylor. Incentive based solutions to agricultural environmental problems: Recent developments in theory and practice. *Journal of Agricultural Economics*, **32(2)** pp. 221-234, 2000.

[8] De Wit, M Provincial guideline on biodiversity offsets. Department of environmental Affairs and Development Planning. Provincial Government of the Western Cape, 2006.

[9] Dickens, C Personal Communication. August 2006. INR. University KwaZulu-Natal. Pietermaritzburg.

www.witpress.com, ISSN 1743-3541 (on-line)

[10] Bjornlund, H *Market Experiences with Natural and Environmental Resources (other than water): Lessons for the Next Generation of Water Market Policies,* ARC-SPIRT, 2003.

[11] DWAF. *A draft paper on the development of a water resource classification system (WRCS): Draft discussion document*, version 8, Pretoria, 22pp, 2006.

[12] Havenga, B. Personal Communication. 9 October 2006. Dwaf. Pretoria

[13] Lodewijks, H. *The application of transferable permits for the control of saline effluent from coal mines and power stations in the Loskop Dam Catchment, MBA thesis, UNISA,* 2002.

[14] Olifants River Forum, *Map of Olifants River Water Management Area and key issues*. Co-ordinator Marianne Nieuwoudt, Tel 017 634 7208 cell 082 459 1021, 2006.

[15] Gunter, P. Personal Communication. 26 October 2006. Olifants River Forum. Witbank.

[16] HRSTS. Hunter River Salinity Trading Scheme. http://www.environment.nsw.gov.au/licensing/hrsts/index.htm undated

[17] Gyedu-Ababio, T Personal Communication 13 October 2006. Sanparks Palaborwa.

[18] Rossouw, O Personal Communication. 10 October 2006. CEO Lebalelo Water User Association.

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Best available techniques for oil spill containment and clean-up in the Mediterranean Sea

F. Cumo[1], F. Gugliermetti[1] & G. Guidi[2]
[1]*Department of Technical Physics, Sapienza University of Rome, Italy*
[2]*ENEA, Department of Fusion and Nuclear Technologies, Italy*

Abstract

An oil spill is the accidental or intentional release of petroleum products into the environment as a result of human activity (drilling, manufacturing, storing, transporting, waste management). Oil spills are unfortunately common events in many parts of the world. A survey has been undertaken on Best Available Techniques (BATs) used both for oil spill prevention and oil spill response. Preventing oil spill is definitely the best strategy for avoiding potential damage to human health and the environment. Nevertheless, when a spill occurs, the best approach for containing and controlling the spill is to take action quickly and in a well-organized and efficient way. The aim of the paper is the proposal of evaluation criteria aiming at an optimal BAT determination when an oil spill occurs. The criteria are referred to BATs for oil spill containment and to BATs for clean up of marine habitat, with particular regard to the item of the preservation of biodiversity.
Keywords: BAT, oil spill, clean up.

1 Introduction

This work has been carried out in the framework of a research activity, concerning the marine environment defence and the loss of biodiversity as a consequence of spills in maritime oil transport, in cooperation with the Italian Ministry of Environment. An oil spill is the accidental or intentional release of petroleum products into the environment as a result of human activities (drilling, manufacturing, storing, transporting, waste management) [1]. This kind of event is not unusual and happens all over the world. Even if oil spills are actually just a

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small percent of the total world oil pollution problem, they represent the most visible form of it. Oil spills give rise to many problems throughout the world. The impact on the ecosystem in an area can be severe as well as the impact on economic activities. The cleaning-up of oil spills is a very difficult and expensive activity.

2 Oil spill prevention techniques

Preventing oil spill is the best strategy for avoiding potential damage to human health and to the environment. Clean up is time consuming and expensive, but clean-up costs are minor compared to the negative impact on an organization's brand and image. Repetitive oil spills, even minor ones, lead to increased scrutiny from environmental regulators. Prevention measures are intended to avoid the release of oil into the environment and are based upon:

- early warning systems;
- satellite monitoring systems.

An early warning system is able to detect spills and leaks of oil-on-water. It can provide 24-hour real-time detection of even small quantities of hydrocarbons and is reliable in rough weather or night conditions, when traditional inspection techniques are ineffective. Early detection enables responding authorities to take immediate corrective actions to stop and contain a spill, thereby offering an effective means of minimizing the environmental and financial impact of a spill event [2].

Earth observation satellites turn out to be more and more often an essential prevention and control tool, especially if it is employed together with standard observation techniques. By means of satellites a global coverage is possible instead of the more local coverage of an aircraft. In order to prevent and control oil spills it could be useful a satellite monitoring system, based upon the use of one of the available channels of Galileo, the new European Satellite Navigation System. It will allow a quick and efficient surveillance and detection of oil spills that today is unthinkable. The use of this technique could discourage illegal oil releases and could allow an accurate localization of particularly heightened risk areas and routes.

Among the prevention techniques should be also taken into account what foreseen in the so-called "Erika packages" [3]. Actually, following the Erika accident, the European Commission prepared measures to increase maritime safety. In March 2000 the Commission adopted a first set of proposals (Erika I package), followed by a second one in December 2000 (Erika II package). The Erika I package reinforces the inspection regime and raises the quality requirements for the classification of societies. It also set a timetable for phasing-out single-hull oil tankers. As a matter of fact double-hull tankers offer better protection for the environment, in the event of an accident. The Erika II package establishes a European Maritime Safety Agency, responsible for improving enforcement of the EU rules on maritime safety. Moreover, the package establishes a surveillance and information system to improve vessel monitoring in EU water, proposes a mechanism to improve compensation for victims of oil

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spills and raises the upper limits on the amounts payable in the event of major oil spills in EU waters.

3 Oil spill containment techniques

When oil falls into water, it tends to spread and cover as much of the available water surface as possible. The size of the oil-covered surface will depend on factors such as the quantity of oil, its viscosity and the weather conditions [4]. It is critical to contain the spill as quickly as possible, in order to minimize danger to persons, environment and property. Containment techniques are used to limit the spread of oil and to allow for its recovery, removal or dispersal and are based upon mechanical containment.

The most common type of equipment, used to control the spread of oil, is the booms. They are used for concentrating oil in thicker surface layers, making its recovery easier as well as for keeping oil out of sensitive areas or for diverting oil into collection areas.

There are many kinds of booms but all boom types are greatly affected by the conditions at sea: the higher the waves swell, the less effective booms become. Normally booms don't operate well when waves are higher than one meter or currents are moving faster than one knot per hour. New technologies, such as submergence plane booms and entrainment inhibitors, will allow booms to operate at higher speeds, while retaining more oil.

4 Oil spill clean-up techniques

Once an oil spill has been contained, efforts to remove the oil from the water can begin. The main types of techniques currently available are:

- booms;
- skimmers;
- sorbents;
- dispersants;
- in-situ burning;
- bioremediation.

When used in recovering oil, booms are often supported by a horizontal arm, extending directly off one or both sides of a vessel.

Skimmers are devices used to recover floating oil from water surface. They may be self-propelled and may be used from shore or operated from vessels [5].

Sorbents are insoluble materials or mixtures of materials used to soak up liquids through the mechanism of absorption, adsorption or both. Absorbents allow oil to penetrate into pore spaces in the material they are made of, while adsorbents attract oil to their surfaces but do not allow it to penetrate into the material.

Dispersants are a group of chemicals designed to be sprayed onto oil slicks in order to accelerate the process of natural dispersion [6]. They are applied to the water surface to break up surface oil slicks and facilitate the movement of oil particles into the water column.

Spraying dispersants may be the only means of removing oil from the sea surface, particularly when mechanical recovery is not possible. Dispersants are most effective when applied immediately following a spill, before the lightest components in the oil have evaporated. Dispersants used today are much less toxic than those used in the past.

In-situ burning involves the ignition and controlled combustion of oil; it is typically used in conjunction with mechanical recovery on open water. Fire resistant booms are often used to collect and concentrate the oil into a slick that is thick enough to burn. This technique has the potential to remove large amounts of oil from the water surface. There are a number of problems limiting the viability of this technique such as, for example, the generation of large quantities of smoke, the formation and possible sinking of extremely viscous and dense residues and safety concerns.

Oil, like many natural substances, will biodegrade over a period of time into simple compounds. The term bioremediation indicates a series of processes used to accelerate natural biodegradation [7]. Bioremediation agents are almost always applied to residual oil on shorelines, for long-term cleanup situations. Usually, heavy oil is first removed before bioremediation is undertaken. Unfortunately the practical use of bioremediation is restricted. In particular, bioremediation should not be used on oil on the sea surface, since any materials added are likely to be rapidly diluted and lost from the slick. Natural biodegradation can be most usefully accelerated when bioremediation is used on land.

Many different problems arise during the clean-up phase. For example, booms are helpful in spill containment in good weather conditions but they are not so effective in rough sea; after the skimmers have finished their work, there is still a small quantity of oil left which needs to be dispersed using for example chemical dispersants. Extra pollution is generated when using dispersants: although they are efficient in breaking up the oil slick, they contribute to the accumulation of oil products on the sea bed, thus contaminating the food chain of marine life.

An emerging method for oil spill clean-up is based on a magnetic separation technique using the material “CleanMag”. The material has been developed by Prof. George Nicolaides at the Technology Education Institute of Piraeus in Greece. It is a nanocomposite magnetic material which is oleophillic and porous and has an apparent density lower than the water one. Because of its oleophillic character, the oil is quickly absorbed upon contact with the material at a weight ratio that can go up to 1:9. This material can be sprayed in granular form over the spill and can be collected from boats equipped with magnetic collection means [8].

5 Proposed criteria for BAT selection

Many techniques are used in order to reduce oil spill consequences. They can be essentially classified into two macro-areas:

- containment techniques;
- clean-up techniques.

The first ones are substantially booms. The second ones encompass:

- booms, used in recovering oil;
- skimmers;
- sorbents;
- dispersants.

It is necessary to take into account also alternative techniques, such as:

- in-situ burning;
- bioremediation.

They have to be included rightfully in clean-up techniques. There are also emerging techniques and among those takes on a special relevance the one based upon the use of magnetic material, capable of absorbing oil. When an oil spill occurs and a technique is required to be chosen, which turns out to be suitable in the context, it would be useful that simple criteria should be readily available in order to carry out the most appropriate choice.

It is possible to identify three series of criteria that have to be applied in sequence; this means that techniques should above all comply with the criteria of the first type, after that they should meet the second type criteria and ultimately they should satisfy the third type ones:

1) main criteria;
2) technical criteria;
3) economic-environmental criteria.

Factors such as:

- the time of intervention;
- the typology of spilled oil;
- conditions at sea;

fall within the main criteria.

A very important factor when choosing the best available techniques, to face an oil spill, is the time of intervention. For example, in-situ burning should be used immediately after the actual spill, before the lighter volatile and flammable fraction has evaporated. Anyway in order to have a complete combustion, the entire oil slick should be covered with flammable substances, but this is often too difficult to carry out.

Other important factor is the typology of spilled oil that is its physical-chemical characteristics. The main oil typologies are:

- light oils;
- medium oils;
- heavy oils.

Some techniques, such as those based upon the use of dispersants have small effect on heavy crude oils. Actually heavy crude oils do not disperse as well as light and medium oils.

A main factor to take into account is represented by the conditions at sea where the spill occurred: calm sea, choppy sea, water covered with snow or ice. For example when spill occurs in water containing a layer or chunks of ice, in situ burning can often remove much more oil than conventional techniques. On the contrary, in severe sea conditions, the oil will be submerged by breaking waves, preventing direct contact between the dispersant and the oil.

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For this first group of criteria it is possible to draw a table showing the aforementioned criteria and the main containment and clean-up BATs. When a BAT satisfies each criterion an X is marked in the corresponding box.

Table 1: Main criteria.

CRITERIA		**BAT**						
		Booms	Skimmers	Sorbents	Dispersants	In-situ burning	Bioremediation	Clean-Mag
Time of intervention	Prompt	X	X	X	X	X		X
	Next	X	X	X			X	X
Typology of spilled oil	Light	X	X	X	X	X	X	X
	Medium	X	X	X	X	X	X	X
	Heavy	X	X	X		X	X	X
Conditions at sea	Calm	X	X	X	X	X		X
	Choppy							X
	Icy	X	X			X		X

Afterwards, technical criteria have been identified. They take into account purely technical characteristics of the BAT such as:

- actual availability;
- feasibility;
- compatibility with other techniques.

It is noteworthy the actual availability of each technique: as a matter of fact it is necessary to verify if the technique is available in the area where oil spill occurred or if it must reach the area from considerable distance.

Another important characteristic is the technique's feasibility, in terms of logistics and other operational aspects. It is essential to check the presence of suitable trained operators and facilities in support of the technique, indispensable to the technique's carrying out.

Finally should not be neglected the compatibility of the technique, under examination, with the other ones used in the context of the operations implemented to face out the oil spill.

At last should be considered the economic-environmental criteria. They take into account costs and the impact of BATs on human health and the environment. It is possible to count among these criteria the following ones:

- the proximity of built-up areas;
- the presence of economic activities, such as fishery and tourism;
- the presence of environmental protected areas;
- the presence of submerged archaeological sites;

- the loss of biodiversity;
- the cost of the technique.

Those criteria should be subdivided according to the following table 2:

Table 2: Economic-environmental criteria.

ECONOMIC-ENVIRONMENTAL CRITERIA		
IMPACT ON HUMAN HEALTH AND ON ENVIRONMENT	Impact on human health	Proximity of built-up areas
		Presence of economic activities
	Impact on the environment	Presence of environmental protected areas
		Presence of submerged archaeological sites
		Loss of biodiversity
COSTS	Costs	

As far as concerns the proximity of built-up areas, these areas plainly require the use of less aggressive techniques that don't give rise to pollution, for example the atmospheric one. The closeness of built-up areas therefore excludes the possibility to employ in situ burning, because it produces huge quantities of black smoke. Clouds of black smoke may result in an oily rain, falling on such areas and leading to temporary mass evacuation.

The presence of economic activities, such as fishery and tourism, should be taken into account. In this case it would be better to use techniques acting quickly, subsequently preventing the oil spill to reach shorelines. A good technique, well satisfying this criterion, is the one based upon the use of a magnetic material, capable of absorbing oil quickly.

In some areas, and in particular in Mediterranean Sea, the possible presence of submerged archaeological sites should be taken into account. In this case are not recommended aggressive techniques, such as the one based on dispersants or in situ burning that could cause irreparable damage to these sites.

Another significant factor is the presence of marine protected areas. The same considerations, done about the previous one, apply to this criterion.

Loss of biodiversity, in the territory where oil spill happened, is the last item of the human health and environment section. Biodiversity is defined as "the variability among living organisms from all sources including, inter alia, terrestrial, marine and other aquatic systems and the ecological complexes of which they are a part" (Earth Summit – Rio de Janeiro, 1992); this includes diversity within species, between species and of ecosystems.

Loss of biodiversity has deep implications for development. Biological resources are renewable and output can be increased under appropriate management. Natural habitats which can maintain productivity without significant management have the ability to provide means for human survival.

Conservation of biodiversity seeks to maintain the human life support system provided by nature and the living resources essential for development. Biodiversity may, therefore, be seen as an indicator of environmental health. Success in maintaining biodiversity must take into account both spatial and temporal factors. It is not, however, possible to ensure a constant level of biodiversity at a particular location over time. This item should be taken into account in all kind of impact analyses.

Therefore also in this case is not recommended the use of aggressive techniques, while it is suggested the use of mechanical containment techniques and the use of techniques such as the one based on absorbing magnetic materials.

In the end the criterion based upon the cost of the technique should not be neglected. This item includes not only the material cost of the technique, but also for example the cost for training of operators. Conditions being equal, should be clearly preferred the cheaper technique.

6 Conclusions

The present schematisation of BATs, used for prevention and containment of oil spills, is gathered from available international literature and from technical experience of Nature Protection Direction of Italian Ministry of Environment. The application of other innovative technique is under investigation and may lead to a further review of the schematisation as well as future improvements of the described BATs.

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References

[1] Bilardo, U. & Mureddu G., *Traffico petrolifero e sostenibilità ambientale*, Unione Petrolifera, 2004

[2] Assilzadeh, H. & Mansor S.B., Early warning system for oil spill using SAR images. *Paper presented at 22nd Asian Conference on Remote Sensing – Singapore, 5 – 9 November 2001*

[3] European Commission, http://ec.europa.eu/transport/maritime/safety /index_en.htm

[4] Mechanical Containment and Recovery of Oil Following A Spill; U.S. Environmental Protection Agency http://www.epa.gov/oilspill/pdfs /chap2.pdf

[5] Clean-up technique; International Tanker Owners Pollution Federation Limited, http://www.itopf.org/containment.html

[6] The use of chemical dispersants to treat oil spills; Technical Information Paper; International Tanker Owners Pollution Federation Limited, http://www.itopf.com/tip4.pdf
[7] Alternative Countermeasures for Oil Spills; U.S. Environmental Protection Agency www.epa.gov/oilspill/pdfs/chap3.pdf
[8] http://www.teipir.gr/cleanmag

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Calculation of the loads of chronic pollution from roadways runoffs

J. Hurtevent, M. Gigleux, M. Despréaux, P. Esnault, F. Caquel, D. Grange, L. Calovi & C. Gardet
Sétra, Ministry of Transports, ASF (South France Motorways), France

Abstract

Motorway infrastructures may cause a chronic pollution of water resources. Wear residues from vehicles, coatings, road equipment as well as the emissions from road vehicles induce an accumulation of particles upon the carriageway which are swept from the roads each time it rains. Protection of the water resource, defined as the common heritage of our nation (Art. L210.1 Code de l'Environnement), requires appropriate measures. To comply, first it is necessary to know about the loads of pollutant which are deposited upon the carriageway to assess the works suited for the protection of the water resource. The National Technical and Scientific Network of the Ministry of Equipment have exploited the measurement provided at nine equipped locations.

The goal of this operation was to obtain correlations between the actual loads in the runoff, concerning various pollutants (suspended matter, PAH, COD, lead, zinc, cadmium, copper and hydrocarbons), and various parameters (annual precipitation, maximal intensity of the rain event, traffic, number of days of dry weather, etc.). At present, two relations might be applied:

- a pollution/traffic law linking the total traffic and the maximal annual global load of pollution collected in the runoff;
- a law linking the maximal compound fraction of annual pollution swept along, and the height of water of the associated event.

These two laws make it possible to quantify the chronic pollution that therefore is liable to reach the water resources and then to adapt the strategy of water resource protection according to the sensitivity of the aquatic environment.
Keywords: runoff, infrastructure impact assessment, copper, cadmium, suspended matter.

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1 Context

The examining authorities of the impact assessment of the road infrastructures' projects are very attentive about chronic pollution due to roadway runoffs: sometimes the decrees of authorization are taken without considering the real impact on the water resource. The data which are still used as a reference date back more than 25 years: at this time gasoline contained lead, only a few vehicles used gas oil, engines were less powerful, were more prone to oil and water leaks and generated more pollutants (lead, hydrocarbons) particularly harmful to the environment. Today lead has almost totally disappeared from fuel: the measured values are, in the majority of the cases, under the levels decreed safe for drinking water [15]. It is not taken account in this paper. Theoretically the precious metals (platinum, iridium, rhodium …) used as catalysts in the mufflers, would have also had to be considered as metal elements. Nevertheless, regarding the latest technologies (the new monolithic catalysts reduce this platinum emission of a factor 100 to 1000) [8], the contents likely to be reached are extremely slight (under detection limit) and will not be considered in this analysis. With this purpose, the results of the long duration measures carried out in the period 1995-2005 on various motorways distributed through the whole national territory have been exploited. It appears that it was not possible to obtain, from these measures, regional values. On the other hand "Pollution-Traffic" laws could be established. Moreover, to take into account the fact that an important part of the pollution emitted is not taken up by the storm water drainage systems, but projected into the nearby surroundings, a distinction has been made between open site (no obstruction for the dispersion by air) and limited site (the pollution accumulate more on the road because of obstructions for the dispersion by air).

2 Methodological process

The data taken into account in the former document [1] came from various studies realized up until 1987. At that date several experimental sites had been equipped so as to characterize the chronic pollution resulting from road exploitation. These sites placed on the interurban motorways (A1, A4, A6, A61, A2, A11, A10, A26) had been distributed through the whole French territory. These sites have been reactivated and new sites were placed so as to update the data, taking into account the evolutions concerning the traffic, the composition and the quality of the fuels, and the improvement of the workings of vehicles.

A relationship between global pollution load and traffic is underlined and may be retained. The traffic taken into account is the AADT (annual average daily traffic).

In other aspects the studies have shown that the polluting loads could differ for the same traffic according to the presence, or not, of obstructions for dispersion by air. Chronic pollution mentioned in this paragraph concerns: link sections, toll plazas, interchanges, and other areas.

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2.1 Definition of open and limited sites

An open site corresponds to an infrastructure, the outskirts of which do not oppose the dispersion by air of the polluting load. A limited site corresponds to an infrastructure, the outskirts of which limit the dispersion by air of the polluting load. These screens that limit dispersion have a minimal length of 100 m, a height equal to or more than 1,50 m, and are placed on each side of the infrastructure, face to face. They are defined thus: noise barrier, earth bank, retaining wall, safety devices, cutting slopes. Plantations (hedges, trees) are not considered as “screens”. The impermeable surface for the loads corresponds to the whole soil surface covered with hydraulic or bituminous concrete, or with double surface dressing, or geomembrane. The surfaces to take into account are those: of the carriageway, of the coated verges or pavements, of the central reservation, of the lay-by, and of the toll platform.

2.1.1 Annual polluting loads drained by the runoff

Link sections:

The Unitarian annual polluting loads to take in account are, for the carriageways, not constituted with porous asphalt, as follows.

Table 1: Unitarian annual polluting loads Cu for one impermeable hectare and 1000 veh/day.

Unitarian annual polluting loads Cu For one impermeable hectare and 1000 veh/day	Suspended matter kg	COD kg	Zn kg	Co kg	Cd g	Hydrocarbons g	PAH g
Open site	40	40	0,4	0,02	2	600	0,08
Limited site	60	60	0,2	0,02	1	900	0,15

For global traffic under 10 000 vehicles per day, the annual polluting loads are calculated proportionally: to the global traffic and to the impermeable surface

$$\mathbf{Ca} = \mathbf{Cu} \times \frac{\mathbf{T}}{\mathbf{1\,000}} \times \mathbf{S}$$

Figure 1: Equation 1: annual load for traffic under 10 000 veh/day.

T = AADT (Annual average daily traffic)
S = impermeable surface (ha)
Cu = Unitarian annual polluting loads for one impermeable hectare and 1000 veh/day

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For traffic over 10000 veh/day: the observation shows that beyond 10000 veh/day the increasing of the polluting load weakens. The annual load is given as follows:

$$\mathbf{Ca} = \left[\mathbf{(10 \times Cu) + Cs(\frac{T - 10\ 000}{1\,000})} \right] \mathbf{S}$$

Figure 2: Equation 2: annual load for traffic over 10 000 veh/day.

Cs = Specific annual polluting loads for one impermeable hectare and 1000 veh/day over 10 000 veh/day

The specific annual polluting loads to take in account are, as follows.

Table 2: Specific annual polluting loads for one impermeable hectare and 1000 veh/day over 10 000 veh/day, for open and limited sites.

Specific annual polluting loads for one impermeable hectare and 1000 veh/day over 10 000 veh/day	Suspended matter kg	COD kg	Zn kg	Co kg	Cd g	Hydrocarbons g	PAH g
Cs	10	4	0,0125	0,011	0,3	400	0,05

During an impact assessment, the calculation of the annual polluting load must be carried out retaining the following traffic:
- for new infrastructures: the traffic foreseen 15 years after the startup
- for existent infrastructure: the traffic foreseen 10 years after the installations of protection of the water resource.

2.2 Particular cases

2.2.1 Toll plaza
The annual polluting load of a toll station is determined according to its visiting traffic and to its total surface between the link sections. It is convenient to retain the characteristic values of a limited site.

2.2.2 The interchanges
The annual polluting loads on interchanges are calculated from: the traffic on the interchange and the impermeable surface of the interchange.

2.2.3 The areas
The annual polluting loads for a rest, or a service area, depend on: the traffic of the link section, which serves the area and the impermeable surface. For a two-

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way area the global traffic is taken in account. For a one-way area, only the traffic of the circulation direction which feeds the area is taken in account. If the traffic data according to circulation directions are not available, the traffic attributed to one direction is equal to half the global traffic. The annual global polluting loads are equal to 1/10 of annual global polluting loads established for link section in limited site. Waste water and water associated with the washing services offered on the area produce a polluting load that must be estimated in more of the chronic polluting loads and treated in accordance with the regulation in force. Very often, the polluting loads linked to the services are quite a lot higher than those characterizing the chronic pollution.

2.3 Maximal impact of runoff

Experimentation has shown that the maximal impacts are by a summer rain in a low water period. The winter polluting loads are therefore not taken in account. The measures from experimental sites have likewise shown that the peak event is directly linked to the volumes of rain that generate this peak event. The relation is established as follows:

$$\mathbf{Fr} = \mathbf{2{,}3} \times \mathbf{h}$$

Figure 3: Equation 3: maximal fraction of annual polluting loads swept.

h = precipitations (m)

The runoff impact is due to its concentration and the capacity of the water resource to endure a concentration increasing which does not impair its use or its vocation.

The runoff quality must be compatible with the aims and measures defined in the impact assessment, namely:
- uses of water resource (drinking water supply, pisciculture, bathing place);
- the qualitative aims of national laws and decrees;
- the sensibility of the aquatic ecosystems;
- the aims of Framework Directive in the field of water policy [10, 12–14].

Table 3: Quality aims of Rhône drainage basin.

Aims/ Maximal concentration	1A	1B	2	3
Suspended matter (mg/l)	25	25	70	150
COD (mg/l)	20	25	40	80
Co (µg/l)		5	1000	
Cd (µg/l)	2	5	5	

1A corresponds to a quality that allows any use of surface water.

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$$Ce = \frac{Fr\ Ca\ (1-t)}{10\ S\ h} \quad \text{or} \quad Ce = \frac{2{,}3\ Ca\ (1-t)}{10\ S}$$

Figure 4: Equation 4: runoff concentration during a peak event (mg/l).

Ca = annual loads (kg)

Finally, during the peak event, the maximal concentration depends only on the traffic.

2.4 Average concentration of storm water runoff

The pollution drained by the rain is characterized by chronic phenomena and sharp ones constituting a peak event that appears once a year (maximal impact notion introduced in the former paragraph). This average concentration (Cm) is calculated as follows:

$$Cm = \frac{Ca\ (1-t)}{9\ S\ H}$$

Figure 5: Equation 5: average concentration (mg/l).

S = impermeable surface (ha) surface imperméabilisée en ha
H = annual precipitations (m)
t = efficiency of water protection works

In the specific hydrographic areas (annual average precipitation < 500 mm), no observation could be realized. The Precipitation value retained for the concentrations calculations is then 500 mm.

3 Road runoff evolutions

It can be interesting to compare the new results with the old data:
This example shows a project of 3 impermeable hectares:

Table 4: Concentration during peak event with former data.

Concentration during peak event	Suspended matter mg/l	COD mg/l	Zn mg/l	Co mg/l	Cd µg/l	Hydrocarbons µg/l	PAH µg/l
5000 veh/day	150	140	1,4	0,1	7,1	2700	0,35
35000 veh/day	1050	980	9,6	0,7	50,1	19000	2,45

Table 5: Concentration during peak event with new data.

Concentration during peak event	Suspended matter mg/l	COD mg/l	Zn mg/l	Co mg/l	Cd µg/l	Hydrocarbons µg/l	PAH µg/l
5000 veh/day	46	46	0,46	0,023	2,3	690	0,092
35000 veh/day	149,5	115	0,92	0,15	6,3	3680	0,4715

As we can see, the polluting loads have dropped since 1987 with a factor from 2 to 10. However, it is noted that pollution, in particular in term of COD, remains largely above the sensitivity of certain aquatic environments. The levels of cadmium and suspended matter are also above the acceptable thresholds. However, the most risk is with respect to the COD: indeed, the COD is the pollution most difficult to treat by the traditional works.

4 Conclusion

The new data thus confirm that pollution has significantly decreased since the year 1980. The motorway levels remain, in particular, dangerous because of the presence of metal element traces, and of suspended matter suitable to asphyxiate the ecosystems. The motorway runoffs thus require to be treated starting from 3000 vehicles per day traffic. This new method however makes it possible to obtain a technical answer adapted to the risk incurred to the aquatic environment. Moreover, the new qualitative aims related to the transcription of the DCE will impose a finer knowledge of the priority substances present in the motorway runoffs. This is why, the sites given up since 2005 will be given back in service in the next years: on the one hand, in order to follow the evolution of the quality of the runoffs, and in addition, in order to identify a finer relationship between the pollution and the climate.

References

[1] Sétra (1995) L’eau et la route volume 2: L’élaboration du projet.

[2] LRPC Nancy, SETRA. (1995). Suivi de la qualité des eaux de ruissellement de chaussées. Autoroute A31 Metz-Nancy.

[3] LRPC Nancy, SETRA. (1996). Mesure de l’efficacité d’un système de traitement des eaux de ruissellement de chaussées. Site expérimental A31 Metz Sud.

[4] Sétra (1997) L’eau et la route volume 7: dispositifs de traitement des eaux pluviales.

[5] SCETAUROUTE, ASFA (1998). Synthèse des études sur la composition des eaux de ruissellement des chaussées. Rapport ASFA n°98-7-2-10.

[6] M. LEGRET, LCPC (2001). Pollution et impacts d'eaux de ruissellement de chaussées. Collection études et recherches des LPC. Route CR27, p. 109.
[7] Sétra (2004) Nomenclature de la loi sur l'eau: application aux infrastructures routières.
[8] Cete de l'Est/Sétra (2005) Recherche de platinoïdes dans les bassins de traitement des eaux pluviales routières.
[9] ASFA, (1999) Synthèse de l'efficacité des ouvrages de traitement des eaux pluviales routières
[10] Directive 2000/60/EC of the European Parliament and of the Council of 23 October 2000, establishing a framework for Community action in the field of water policy [Official Journal L 327 of 22.12.2001].
[11] Loi sur l'eau du 3 janvier 1992 (transposée dans le code de l'environnement: Articles L211-1 et suivants)
[12] Décret no 93-742 du 29 mars 1993 relatif aux procédures d'autorisation et de déclaration prévues par l'article 10 de la loi no 92-3 du 3 janvier 1992 sur l'eau
[13] Décret n° 2005-378 du 20 avril 2005 relatif au programme national d'action contre la pollution des milieux aquatiques par certaines substances dangereuses
[14] Arrêté ministériel du 20 avril 2005 pris en application du décret du 20 avril 2005 relatif au programme national d'action contre la pollution des milieux aquatiques par certaines substances dangereuses
[15] Décret n° 2001-1220 du 20 décembre 2001 relatif aux eaux destinées à la consommation humaine, à l'exclusion des eaux minérales naturelles

Water pollution legislation in Iran vs England and Wales

G. Asadollahfardi
Department of Civil Engineering, Faculty of Engineering,
Tarbiyat Moalem University, Tehran, Iran

Abstract

In order to control water pollution in a more effective way, there is a need to set up legislation which will cover all aspects relating to water pollution. The second step would be the implementation and management of that legislation in order to achieve the most effective result. Despite three decades of existing legislation and management for water pollution control in Iran, the results of implementation are still not effective. Hence, it is vital to identify the problems which cause improper improvement in surface and ground water quality. First a review of some historical trends of water pollution legislation and management is presented, which have been in progress since the start of legislation in Iran, as well as in England and Wales. Then a comparison of current legislation in both countries is made and also the implementation of the legislation is explained. The results indicate an imbalance in current laws. They point out the existing flaws in the system used in Iran and suggest a few ways for improving it.
Keywords: legislation, uniform emission standard, water quality objective standard, water pollution in Iran.

1 Introduction

Adequate legislation is the primary requirement for the control of surface and ground water pollution and related organizations must enforce and administer the law. However, depending on factors such as the legal system, degree of development, climate and the traditions of a country, the type of organisation may differ from one country to another (Ellis [4]). A very important point to note is that adequate legislation without effective administration is not constructive. Independence of any such organization is necessary and the administrator should

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have freedom to operate according to the clauses in the legal guidelines. In general, the main tasks of any organisation undertaking water pollution control are as follows:

1-0 Apro-active approach to prevent pollution arising by:
1-1 Legislation
1-2 Action on the ground.
2-0 Enforcement by prosecution “Stop-Notices” ordering work to be done.

Generally, there are two philosophies regarding the surface water pollution. In the first approach, which is known as the Uniform Emission Standard (UES), a common quality standard is set for all river users without any regard for the dilution possibility or the existing quality of the receiving streams or their future uses.

The second approach called the Water Quality Objective (WQO) Standard, regards the present and future use of a river, which has an active role. Modern technology is used to assess the natural ability of the river, how it assimilates and, to varying degrees, provides self- purifying effluents without creating untoward deterioration effects. Each polluter is then advised accordingly on the level of the river pollution and the absorption level of that particular stretch of the river.

Between the above two philosophies, the first method is more commonly used, because of its easy use of operation. The second method seems to be more effective and accurate. However, a uniform emission standard does not permit the optimisation of the resources associated with a river system unless, in the unlikely event, of aiming for the best possible technology approach (Ellis [4]).

Legislation for surface and ground water pollution control has been practised for three decades in Iran but the enforcement has not been so effective. As a result, there have not been any serious improvements in surface and groundwater quality in Iran. It is obvious that comparing two laws in two different countries with different cultural views, technological background and social development is a very difficult task, but the author tried to find out the deficiency on the existing law and managment in Iran relating to that in England. The issue here focused on the ineffectiveness of the existing law, implementation and the ways to improve the existing situation.

2 The provisions of law on water pollution control in Iran

Apart from the North of Iran (near the Caspian Sea), the rest of the country has an arid or semi-arid hydrological climate. Due to the limited water resources, there is a serious water shortage especially in dry seasons. Therefore, protecting surface water and ground water aquifers is especially important. As the volume of the surface water is low, the capacity of the acceptable polluted water has also to be in a smaller volume in order to be suitable for the general well being of the people, this makes the protection even more critical. Furthermore, due to the special attention given to the industrialization of the country, the polluted water from industrial plants and agricultural lands should not be allowed to endanger the quality of the limited water resources.

It seems that a brief explanation of how legislation is made in Iran and how it is operated is necessary, hence a short summary is provided below.
There are two common ways in which this can be done:

2.1 A minister or a director puts forward a motion regarding his own ministry or organisation to the cabinet. It is discussed in the cabinet and once it is approved by the majority, it is sent to parliament. The parliament refers the bill to the experts committee, whose duties at an early stage are to take an overall view and examine the bill. Once approved by this committee it is returned to the parliament to be voted on as a whole. It is once again returned to the experts committee to be examined in detail. After the experts committee approve the bill, it is forwarded for a second time to parliament to be voted on. When the majority of members pass the bill, it is then sent to the Guardian Council (a powerful council consisting of 6 religious experts and 6 law experts) to be examined as to whether it conforms to the constitution and Islamic laws or not. If there are no flaws in it, it is returned to the parliament to become law and finally is returned to the cabinet to be implemented.

2.2 Step two is rather similar to step one, the only difference is in the way the motion is put- forward. Fifteen or more members of parliament can approve the motion and the rest is exactly the same as step 1.

For the first time, in 1956, the Water Pollution Control under Article 12, section C, (Hunting and Fishing Act) was passed into law by the Parliament and amended in 1967. The Environmental Protection Law consists of 22 sections and nine provisions and was passed in 1974 some of them are as follows.

Section 1: Protection and improvement of environment and prevention of any pollution and any activity, which may cause an imbalance in environment, also all actions related to wild animals, are under the responsibility of the Departement of Environment (DOE). This is limited to the boundaries of the Iranian water system.

Section 2: The DOE is referred to as an organisation, which is directly answerable to the presidential office. It also has legal entity and is financially independent. The organisation functions under the DOE higher committee. The president of Iran is the head of the higher committee.

Section 5: Appointment of any person to hold office, as the chairman shall be made by the president of Iran.

The constitutional arrangement of the DOE is shown in Figure 1. Suggestion of the standards for prevention of pollution of water, air and soil and distribution of domestic and industrial solid waste and in general parameters which have adverse effects on the environment is the duty of the DOE, and also implementation of educational programmes for the promotion of public opinion for protection and improvement of environment is the duty of the DOE.

According to item 6 of the board approved guidance on prevention of water pollution regulation (1994), the DOE has been made responsible to monitor and classify all of the surface and groundwater, but they still use the Fixed Emission Standard method to control water pollution because there is not yet complete information and control upon all surface and ground water quality in order to establish the degree absorbency of pollution into these waters.

Section 11: The DOE in accordance with regulations and codes of practice, relating to section ten, identify factories which cause pollution to Environment. All owners of the factories responsible for the pollution are given an enforcement notice with relevant reasons. They are then obliged to remove the pollution source in a specified period or stop their activities. Any person who fails to comply with any requirement imposed by an enforcement notice by the DOE is prevented from any other activities. If any person concerned questions the enforcement notices ordered by the DOE, they can appeal the case to the public court in that area. The court will then immediately study the case and if it finds any plaintiff they will immediately quash the enforcement notice and the plaintiff is allowed to continue normal activities. The decision of the court is final. For sources and parameters, which have imminent danger for the Environment, the chairman of the DOE has the right of stopping the activity of the polluter without any written notification.

Section 12: Owners, or the responsible persons in the factories and workshops, which were mentioned in section 11, must stop their activities whenever they receive the order of the DOE. Continuation of work or activity depends on the permission of the DOE. If any person fails to comply with the order of the DOE or the decision of the relevant court, they may be sentenced to imprisonment from 61 days to one year or pay a fine.

Protection of seas and rivers at the boundaries of the country from any oil pollution is the duty of Road and Transportation Minstry. In this regards, there is a law which was passed in 1975 and it consists of nineteen sections and eight provisions. Some of the important sections are detailed below:

Section 2: "The pollution of rivers at the boundaries, internal waters and seas in Iran from oil or any oil based polluters, caused by ships, drilling platforms, artificial islands and from piping facilities, or oil resources in the sea or on the land are illegal. Any person polluting the water can be sentenced to jail from six months to two years and penalised to pay a fine.

Section 11: The enforcing authority for the above section is the Shipping and Harbour Organisation, which is under the supervision of Ministry of Roads and Transportation.

3 Summary of the water pollution control legislation in England and Wales

In the United Kingdom, most rivers are relatively small and therefore they can only accommodate limited amounts of pollutant load (Robinson [11]). Therefore, the main reason for the development of the water pollution control legislation in the UK, which was established before other industrial countries, such as the USA, was the decrease in river quality caused by the intensification of industrialisation during the Industrial Revolution and the resulting epidemic diseases associated with it. Early legislation was based on having regard to riparian rights. Land owners traditionally possess rights relating to the quality of the water in the rivers or streams and so on, which is received by the land owner in his section of that river or stream, under common law. The land owner may use the water, but cannot diminish its quality or quantity (Jenkins [5]).

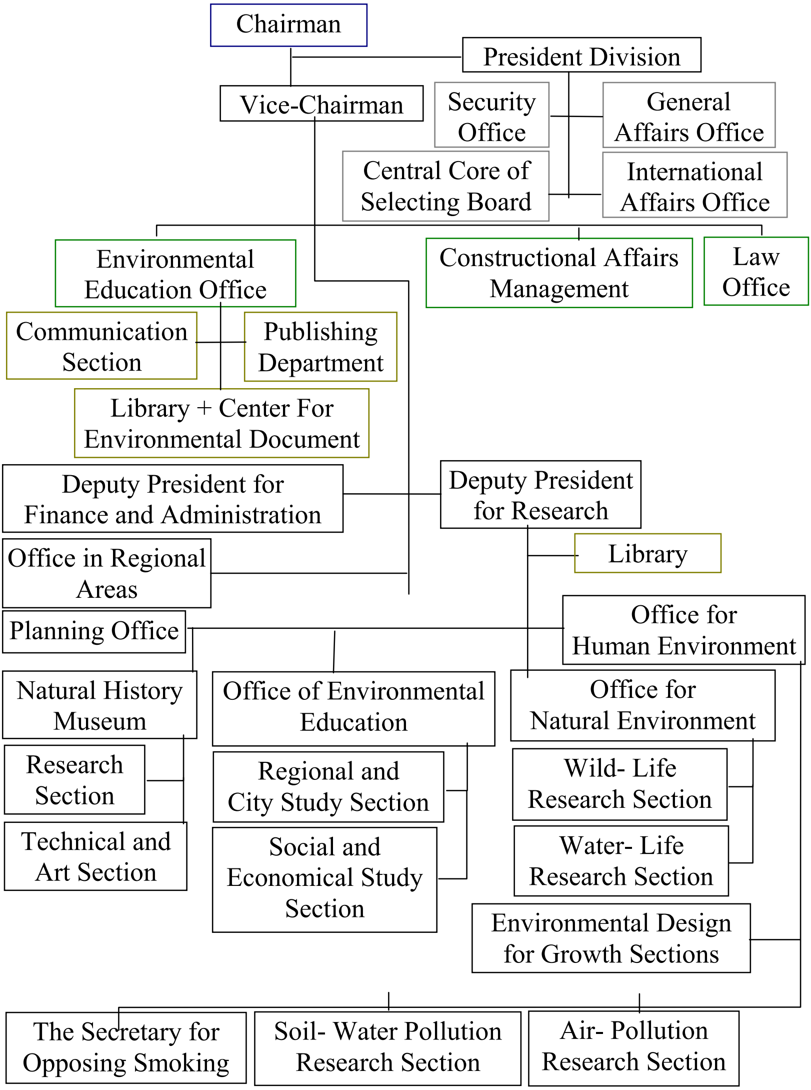

Figure 1: The constitutional chart representing the DOE in Iran (DOE, 1994).

In 1865 and 1868 the Royal Commission recommended the modern water pollution control legislation on rivers. The 1868 recommendation of this Commission regarded the standards of purity for effluent discharge into watercourses. The Royal Commission attempted to incorporate their recommendations on three occasions initially in 1872, then in 1873 and finally in

1875. In none of these three attempts, was success achieved. In 1876, some of the Commission's recommendations were passed by Parliament. One of the Acts passed during this period was the River Pollution Prevention Act (1876). The Act is concerned with the control of sewage discharges into rivers from domestic and industrial sources. At that time, there were two factors which prevented suitable effective pollution control. The first factor was the division of responsibilities between the different authorities made responsible for different aspects by the Act, and the second one was the involvement of some personalities that made it difficult to operate. Despite these weaknesses, this River Pollution Control Act operated through the mid twentieth century (Ellis [4]).

The River Authorities were formed under the water resources Act 1963 in 1964, which empowered conservation, redistribution and proper use of water resources in their areas (Jenkins [5]). The Regional Water Authorities were formed under the water Act 1973, in 1974 and were responsible for the management of most aspects of the water cycle, except when the water supply was the responsibility of the statutory water companies, and when certain powers remained related in certain Government Ministers and agencies. These include water supply, wastewater treatment, and operations on the river catchment basis and finally the administration of legislation to control water pollution (Howarth [6]). Another Act on the control of pollution Act (COPA) 1974 was passed by the Parliament in 1974 which was a controlling act consisting of four parts as stated below

a) Control over solid wastes;
b) Control over water pollution; (to a limited extent)
c) Control over noise pollution; and
d) Control over air pollution.

Apart from item (b), others were largely implemented since 1976 (Doughty [2]]), but the implementation of part (b) was postponed to 1983. Most provisions came into force over the period of July 1984 to January 1985 (Matthews [8]). This Act replaced those of the rivers which were the Prevention of Pollution Act 1951 and 1961, Estuaries and Tidal Waters Act 1960 and the Water Resources Act 1963 (Matthews [8]). Between 1974 and 1989 the water industry in the UK was composed of ten Water Authorities which were responsible for water supply and sewerage services as well as several other water related responsibilities and 29 Statutory Water Companies (SWCs) that only provided water supply services. In the UK, about one third of consumers have received water supply services from the SWCs but received their sewerage services from one of the Water Authorities. The SWCs were subject to statutory controls on profits, dividends, borrowings and statutory limitations on the number of votes. The water and sewerage companies were formed by privatisation under the same regulatory framework. The water bill was making its way through parliament after the passing of the Water Act 1989. In April 1990, England and Wales's ten water and sewerage companies were privatised. Both abstraction of water from rivers and aquifers, and discharges of unclean water to clean water-courses were controlled by the National River Authority which has now been replaced by the Environment Agency. Office of Water Service (OFWAT) is not a government department but an independent body which is set up to implement and administer the responsibilities of the Director General of Water Services, as provided for in the water Act 1989 (Macrory [9]). OFWAT is also a watchdog body, whose mandate is to protect the interests of the customers of the privatised water and sewerage companies efficiently. It also controls prices charged to customers and aims to ensure that the companies operate effectively (Croner's Environmental Management, 1991). OFWAT operates through its central headquarters (based in Birmingham) and it has other offices in eight other cities through England and Wales. Its main tasks are as follows:

a) control of prices;
b) standard of service; and
c) consumer protection:

In England and Wales, regulation of quality has been tackled by subjecting water companies to a wide range of quality regulations and constraints. These include:

i) drinking water quality regulations;
ii) pollution controls;
iii) customer service standards; and
iv) customer service standards, which include codes of practice, and are monitored by OFWAT.

The legal controls over the pollution of water were provided under part III of the Water Resources Act (1991) which came into force in December 1991. According to section 84 of this Act the following should be considered.

1) It shall be the duty of the Secretary of State and of the [Agency] to exercise the powers conferred on him or it by or under the water pollution provisions of this Act [other than the preceding provisions of Chapter one (Quality Objectives of Water Resources Act (1991) and sections 104 and 192 the chapter] in such manner as ensures, so far as it is practicable by the exercise of those powers to do so, that the water quality objectives specified for any waters in

a) a notice under section 83 of this chapter; or
b) are achieved at all times

2) It shall be the duty of the [Agency], for the purposes of the carrying out of its functions under the water pollutions of this Act-

a) to monitor the extent of pollution in controlled waters; and
b) to consult, in such cases as it may consider appropriate, with river purification authorities in Scotland (Duxbury and Morton 2000)

The NRA was responsible for monitoring, controlling and seeking to prevent pollution discharges (Howarth [7]). It had the power to prosecute dischargers who had committed an offence. Under section 85 of the Water Resource Act 1991, it is an offence to cause or knowingly permit the following acts:

a) discharge of poisonous, noxious or polluting matter, or any solid waste matter to entre any controlled waters;
b) discharge of matters other than trade or sewage effluent, to enter controlled waters by being discharged from a drain or sewer;
c) trade or sewage effluent to be discharged:

i) Into any controlled waters; or
ii) from land in England or Wales, through a pipe, into the sea outside the seaward limits of the controlled waters.
d) Trade or sewage effluent to be discharged from a building or from any fixed plant or into any land, or into any waters of a lake or pond which are not inland fresh waters; and
e) any matter which enters any inland fresh waters so as to tend either directly or in combination with other matters which or another person to impede the proffer flow of the waters in a manner leading, or causes or permits to enter those waters and is likely to lead to a substantial aggravation of
i) pollution due to other causes or
ii) the consequences of such pollution (Duxbury and Morton [3]).

The Environment Act (1995) consists of 218 sections and 24 schedules. According to this law implementation of some chapters and sections of the water industry 1991, water resources 1991, Water Act 1989 and some other parts of Acts relating to protection of environment are under the control of an integrated duty called the Environment Agency (the EA). The National Rivers Authority and the London Water Regulation Authority are abolished (Duxbury and Morton [3]).

4 Comparison of current water pollution legislation of Iran with England and Wales

There are some similarities and differences apparent in the two systems which can be seen in Tables 1 and 2. The comparison made between the two systems is reached through a detailed study of different sections and provisions of the two laws. Some of the similarities and differences are as follow:

In section 110 (1) of Environment Act 1995, the EA support any authorized person who would perform his power or duties in accordance with this section but in Iranian Environmental protection law, there is no such section which would support any authorised person to exercise his powers or duties in Iranian law.

Table 1: Principal system for control of water pollution in Iran.

Principal Act	**The relevant ministries**	**Enforcing authorities**
Hunting and Fishing Act 1956	The president of Iran	The DOE
Hunting and Fishing amendment Act 1967	The president of Iran	The DOE
Protection and improvement of Environment Act 1974	The president of Iran	The DOE
Protection and improvement of Environment amendment Act 1991	The president of Iran	The DOE
Protection of seas and rivers at the boundaries from oil pollution Act 1975	Minster of Road and Transport	The Harbours and Shipping Organisation

Table 2: Principal system for control of water pollution in England and Wales.

Principal Act	The relevant ministries	Enforcing authorities
The Control of Pollution Act 1974	The Secretary of State	The EA
The Control of Pollution Act 1989 (Amendment)	The Secretary of State	The EA
The Water Act 1989	The Secretary of State	The EA (some sections)
The Water resource Act 1991	The Secretary of State	The EA (some parts)
The Water Industry Act 1991	The Secretary of State	The EA (some sections)
The Environment Act 1995	The Secretary of State	The EA
The Pollution prevention and control Act 1999	The Secretary of State	The EA

According to section 82 and 83 of Water Resources Act 1991, the EA classifies the quality of water and determines the objective of using the controlled water and comprehensive effluent standards, environment quality standards and action plan for preventation of pollution but at present the DOE only exercise Fixed Emission Standard (DOE [10]) that is not effective. Having Environment Quality Standard needs a complete network of sampling sites and analysing collecting data.

The Ministry of Roads and Transportation and the enforcement authority in the Shipping and Harbour Organization is responsible for oil pollution. Hence, there is not an integrated management system in control of water pollutoion in Iran.

Regarding section 108 of Environment Act (1995), which is called the powers of enforcing authorities and persons authorised by them, there are 16 subsections. According to the section any authorised person by the EA at any reasonable time (or, in an emergency, at any time and if need be, by force), can inspect any premises which he has reason to believe it is necessary for him to enter; or if there is any serious pollution to environment authorised person can enter to the place for examination or checking (Duxbury and Morton [3]). But in the Iranian Law if the authorised person from the DOE wants to enter the place for checking the level of pollution, the owner can prevent the entry, unless the authorised person has permission from the public prosecutor's office. This causes delay in stopping the source of pollution.

The comparison made so far between the legislation of the two countries has only been detailed information between sections and provisions of the two laws, so more practical real life situations are as follow.

- England has several centuries of historical institutional developments; the Islamic republic of Iran has relatively short history of political evolution, which has experienced very drastic changes.
- England has a nationwide common effluent regulatory standard and water specific pollution control actions to attain target ambient water quality while Iran

has a fixed emission standard which is not appropriate and effective for all part of the country.

- According to Environmental Law (DOE [10]), establishing any types of development project (Economic, Industrially, etc), government and private sectors must consider protection of the environment, but in practice priority of development is more significant than environment protection
- Public is not aware of the significance of the protection of the environment, even sometimes in a judiciary system, despite the existing law for the protection of the environment, courts don't issue reasonable judgement for the people that don't comply with the legislation and regulation.
- Implementing laws needs sufficient tools, appropriate technology, human recourses, institutional capability and contribution and interaction between related ministries such as the Ministries of Industry, Power and Jihad-agriculture, which England has, but the DOE does not have the appropriate capabilities.

Recently there is closer contribution and interaction among the DOE and mentioned ministries, but as mentioned previously agricultural and industrial developments are the priorities in Iran. Therefore the following are suggested as first steps:

1-It is necessary to strengthen the capacity of the DOE in order to implement the water quality monitoring, the environmental impact assessment for the new developed industrial and agricultural sectors and executing laws, regulations and standards in all part of the country.

2-Close cooperation and interaction between the DOE and the related ministries is necessary to improve waterquality.

3-To design a timetable for the polluters (both governmental and private sector) in which they can reach the effluent to a standard level with a step-by-step programme.

4-Public awareness and environmental situation should be one of the main projects implemented in Iran through the close cooperation with media and NGO.

It is quite a good aim to strengthen the capacity of the DOE. This requires a huge budget and human resources, proper technology, and suitable institutional organization. Of course this project can be carried out as a pilot project in a limited area and if the result is successful, then it may be extended and duplicated in other parts of the country.

5 Conclusion

As a whole the existing water pollution law in Iran needs reviewing and may also require amending in order to reach the standards that will tackle the environmental pollution issues.

In comparisons made between the legislation of England and Wales and Iran regarding water pollution the following points may be necessary in improving the current law and Management of the Iranian system.

-Political will is the most important element; it can help improve the implementing of the legislation and environmental situation.

-To strengthen capacity of the DOE (human resource, technology, institutional organization, etc) so that the DOE will be able to enforce the law properly.
-Cooperation and interaction between the DOE, Minstry of Housing, Ministry of Culture, Ministry of Education, Radio and Television Organization and Private sectors is vital.
-Public awareness and education are other elements which need a master plan study and greater effort. The subsection regarding public awareness needs to be expanded and explained more thoroughly and also the public training needs to be more widespread using mass media.
-The current control of water pollution lies with the DOE and the Ministry of Road and Transportation. It seems that if it was possible to give the control to one organisation (the DoE) it would improve the management of the water pollution.
-To strengthen the DOE in all parts of the country for implementing water quality monitoring and analysing data, so the results of interpretation of data can help decision makers to make a suitable decision for reduction of water pollution, and it can help for the development of local standards.
-The offence section of the law needs to be more vivid and more detailed in order to make aware the offenders so that they can be punished accordingly; this will also help greatly the management of the water pollution.
-There is no legal position for Non-governmental Organisations. It is suggested to add a section relating to NGO to Iranian water pollution legislation.
-It would be necessary to review effluent standards and to prepare comprehensive environment quality standards for watercourses. The environment quality standards should be set for the receiving watercourse and not the discharge itself.

References

[1] *Croner's Environmental Management* "Water Pollution Control and Water Supply" Kingston upon Thames: Croner, October, (1991).
[2] Doughty, G. M. COPA: "An Act of Remembrance?" *The Public Health Engineer.* 10, 178- 179, (1982).
[3] Duxbury, R. M. C. & Morton, S. G. C. "*Blackston's Statues on Environmental Law"* Blackstone, London, U.K., Third Edition, (2000).
[4] Ellis, K. V. "*Surface Water Pollution and its Control"* Houndmills, Macmillan, UK, (1989).
[5] Jenkins, W. O. "Decision Support System in River Basin Management" *PhD Thesis*, Imperial College, University of London, (1988).
[6] Howarth W. "*Water Pollution Law"* show and sons, London, UK, (1988).
[7] Howarth W. "The Law of the National Rivers Authority" The National Rivers Authority and the Centre for Law in Rural Areas, University College of Wales, Aberystwyth, (1990).
[8] Matthews, P. J. COPA II: "Interpretation and Application" *The Public Health Engineer*, 14, 38-42, (1987).
[9] Macrory R. "Water Act" Sweet and Maxwell, London, (1989).

[10] The Department of Environment “Laws and regulations relating to protection of Environment” *The DOE*, (in Farsi), (2000).
[11] Robinson, R. “The Control of Pollution Act 1974. Implication for River Quality Management” *Journal of Institute of Water Engineering and Scientists,* 34(2), 129-144, (1980).

Preventing waste oil from polluting river and coastal sea environments by a novel technology

M. Sumimoto[1] & T. Azuma[2]
[1]*Sumimoto Scientific Institute Co. Ltd, Japan*
[2]*JDR University, Japan*

Abstract

Every day, a huge amount of waste lubricating oil (LO) is generated worldwide. It is feared that some of this waste oil is discarded or slips into the environment. We developed a novel technology that enables semi-permanent use of LO without generating any waste, compensating for lost oil and reducing LO consumption. This technology keeps LO clean much like our kidneys clean our blood; the technology is therefore called a kidney system. The kidney system has been working well in many marine and cogeneration diesel engines, hydraulic machines, and machine workshops. The authors have also applied the kidney system to various environmental problems by custom remodelling it for each specific application. Focusing on preventing pollution of coastal sea, this paper reports the authors' rationale for this research and the waste oil situation both in Japan and abroad and gives an outline of the technology we developed. Furthermore, a case in which the authors succeeded in preventing waste oil from polluting rivers and neighbouring coastal sea is presented. The waste oil was generated in air compressors of a fertilizer plant as a mixture of LO and cooling water; i.e. emulsion of oil in water (O/W).
Keywords: no waste oil, semi-permanent use of engine oil, emulsion of oil in water, waste oil from compressors, fertilizer plants, slip of oil, pollution of river and coastal sea.

1 Introduction

The authors have been engaged in developing a system of technologies for provision of clean fuel oil, clean lubricating oil (LO) [1], and clean water [2].

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The clean LO technology can keep LO permanently clean in machines and has been working well in many engines and hydraulic machines [1]. Practical application of this clean LO technology has proved very useful for saving natural resources and protecting the environment by: 1) enabling semi-permanent use of LO without creating waste oil; 2) prolonging life of machines by reducing wear; and 3) preserving efficiency of machines.

This paper reports the authors' R&D philosophy and the current situation of waste oil in Japan and abroad, and provides an outline of the clean LO technology we developed. In addition, a case in which the technology was used to prevent possible pollution of rivers and neighboring coastal sea by LO-polluted waste water from air compressors at a Japanese fertilizer plant is presented.

2 Research stance

In 1994, the authors began to share data and ideas that were considered quite ahead of the time because they focused on developing technologies for semipermanent use of natural resources and products rather than for recycling.

Their philosophy went as follows: by representing natural resources with water, this may be seen as borrowing water from our future grandchildren and hence we should return the borrowed water to our grandchildren (i.e. to nature) in clean condition. Thus this philosophy was called "Thought of Borrowing Water."

The authors believe that this philosophy and its related technologies should be known and used all over the world to ensure the future coexistence of humankind and nature. With this in mind, the authors are trying to verify the technology, write as many papers as possible, and construct a theoretical base of the technology, because any well-refined theory should not only incorporate all data within it but also predict what more can be done using the technology.

3 Waste oil in Japan and the USA

Tables 1 and 2 show total oil demand and waste in Japan and in the USA in 1991 [3,4]. Waste oil accounted for as much as 50% of the total demand in both countries. It seems that at least some of this waste oil was thrown out into the environment. Furthermore, the situation today remains about the same as that reported in 1991 in both countries as well as in some other countries [4].

Figure 1 shows waste oil flowing down a seaside cliff into sea. One newspaper reported that a number of drums of waste oil had been discarded into a hole dug for waste disposal. Some years later, namely in 1997, the waste oil was discovered exuding from the cliff [5]. According to another newspaper report, 55,000 drums of waste oil were left in a mountain area [6]. It is not known who left the waste oil there, and it was feared that the waste oil might permeate into soil and pollute underground water. Eventually, the local government disposed of the waste oil at a cost of JPY240 million (about €1.7 million).

Table 1: Waste oil in Japan.

LO demand	2470 ML
Waste LO	1270 ML (51.4%)

Table 2: Waste oil in U.S. (1991).

LO demand	9850 ML
Waste LO	5200 ML (53%)

Figure 1: Waste oil flowing down a cliff into coastal sea.

4 Outline of technology for preventing waste oil

4.1 Principle of new LO system

Our new technology for preserving clean oil acts in the same way as our kidneys keep our blood clean and therefore it is referred to as an LO kidney system. The key element of this system is a newly developed filter. Human kidneys comprise 1~2 million filtration structures called nephrons [7], and so by analogy our newly developed filter is called a nephron filter.

Figure 2 shows comparison between conventional filters and the nephron filter. LO flows between sheets of filter paper in the nephron filter. Sludge particles sit on the filter paper and are removed while LO flows between the filter papers. Very small particles measuring 0.001~1 μm in diameter are considered to obey Brownian motion [8]. Therefore the nephron filter is considered to remove particles as small as 0.001 μm in diameter. Field data show that the nephron filter almost completely removes sludge particles larger than 1 μm as well as most fine particles measuring smaller than 1 μm, which conventional systems cannot remove.

Figure 3 shows the LO kidney system in marine and cogeneration diesel engines. Here, it comprises a nephron filter and small centrifugal separator of very simple structure. The conventional system removes sludge by large centrifugal purifiers with highly complicated structures and is not able to remove fine sludge particles. The amount of sludge therefore increases with running time using conventional LO systems and eventually LO must be changed with fresh oil in the conventional LO system.

Figure 2: Comparison of filter.

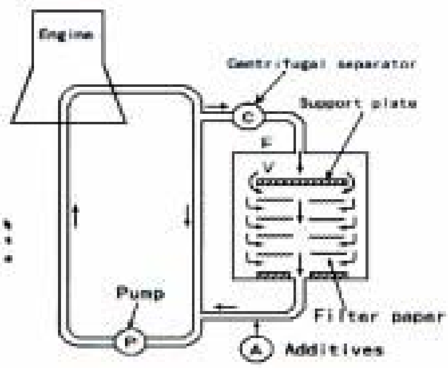


Figure 3: LO kidney system.

4.2 Fundamental features of LO kidney system

The kidney system has 2 features that conventional LO cleaning systems do not possess: 1) It can remove the majority of sludge particles smaller than 1 μm as well as completely removing larger particles; and 2) it completely removes oxidized LO. All of the remarkable results of the kidney system derive from these 2 unique features.

Figure 4 shows that the kidney system almost completely removes oxidized LO [1] because no absorption of infrared (IR) radiation is observed near 1,710 waves/cm. When there is oxidized LO, strong IR absorption occurs at 1,710 waves/cm as shown by the broken line in fig. 4.

Figure 5 shows the results of a comparison ship test of 3 cases: 1) no LO cleaning; 2) conventional LO system; and 3) kidney system [1]. In general, normal pentane (n-p) insoluble is used as index for contamination of LO. As shown in fig. 5, n-p insoluble was kept under 0.1 mass% for an extended period in the case of the kidney system, while it increased with running time in the other 2 cases. These results verify that the kidney system can keep LO clean for a long time due to the 2 fundamental properties mentioned above.

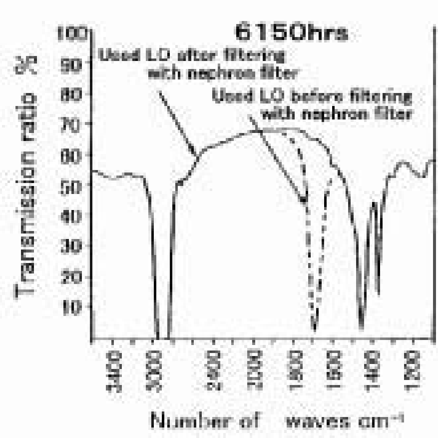


Figure 4: IR analysis of used oil.

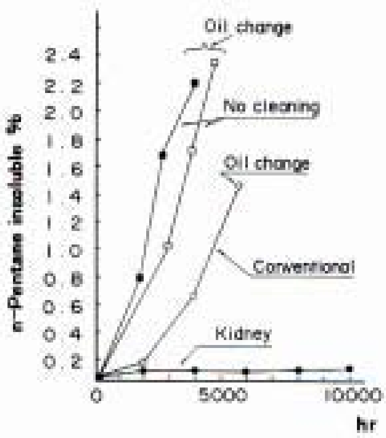


Figure 5: Field test comparing LO systems.

5 Preventing pollution of river and coastal sea

5.1 Generation of waste oil at a fertilizer plant

In fertilizer plants there is a process in which air is separated into oxygen and nitrogen. Air is therefore compressed by air compressors and cooled. Cooling after compression results in condensation of humidity and water generation. Some LO in the compressor mixes into this generated water and thereby produces emulsion of oil in water (O/W).

Water content of the emulsion reaches maximum value of about 99% during Japan's rainy season (June~July) and minimum value of about 70% during the low-humidity winter season from December to February.

In the past, O/W emulsion was generated and largely carried off-site by waste disposal companies. However, few restrictions were enforced regarding how and where these companies disposed the oil. It seems that a part of the O/W emulsion was drained into rivers and coastal sea. For this reason, a system for recycling and semi-permanent use of compressor LO was planned so that no waste emulsion was required to be taken away from the plant.

5.2 System for recycling waste oil

The recycling system was designed based on the following principles:

1) Water in the O/W emulsion should be evaporated by action of steam from steam boilers at the plant.
2) Residual LO should be cleaned by LO kidney system, namely by centrifugal separator and nephron filter.

Figure 6 shows the schematic process of the recycling system, and fig. 7 shows the recycling unit. Each process is made in 1 unit in sequence. As shown, the system comprises 3 processes described as follows.

[Process 1]

During this process as much as 90% of water in the emulsion is evaporated. The process works as follows:

1) Waste emulsion is supplied into the tank as shown in fig. 6.;
2) Waste emulsion is heated by steam at around 100~110°C;
3) Air is supplied into the tank so that waste W/O emulsion may be well mixed and to allow uniform heating;
4) Water is evaporated until water content decreases below 10%;
5) At the end of this process, waste water becomes waste oil including about 10% water.

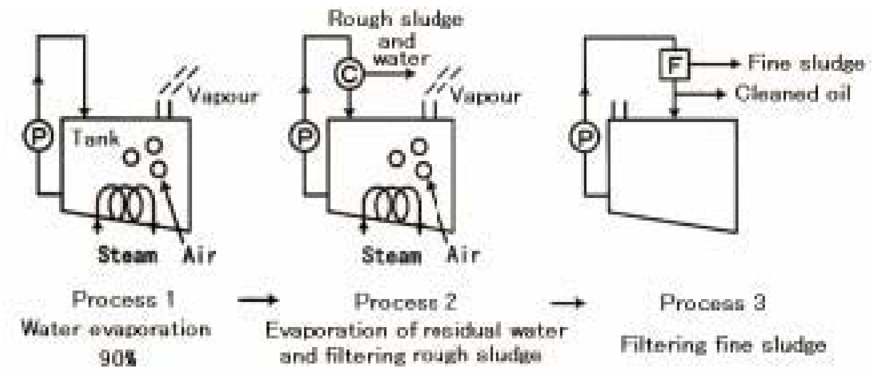


Figure 6: Schematic view of recycling process.

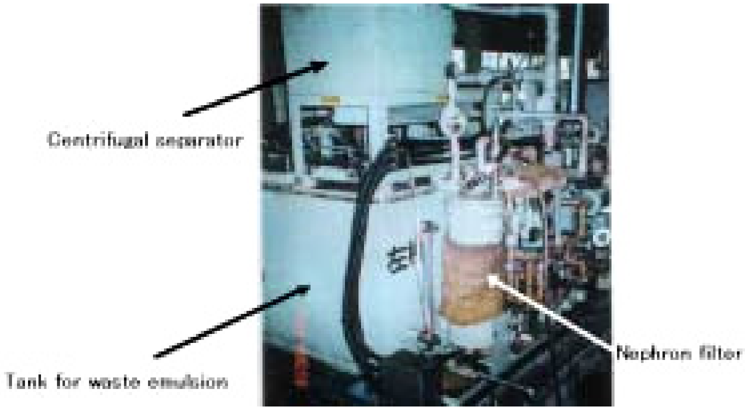


Figure 7: Recycling unit.

[Process 2]
The purpose of this process is to evaporate almost all remaining water and to remove rough sludge by centrifugal separator. The process works as follows:

1) waste oil is heated to around 110~120°C and almost all remaining water is evaporated;
2) air is supplied as in process 1;
3) the centrifugal separator removes rough sludge together with water.

[Process 3]
The purpose of this process is to remove fine sludge from waste LO by nephron filter to produce recycled LO.

5.3 Recycling result

Figure 8 shows a schematic view of the change from fresh LO to recycled LO.

Figure 9 shows: 1) fresh LO; 2) waste water (waste emulsion); 3) sample after process 1; 4) sample after process 2; and 5) recycled LO.

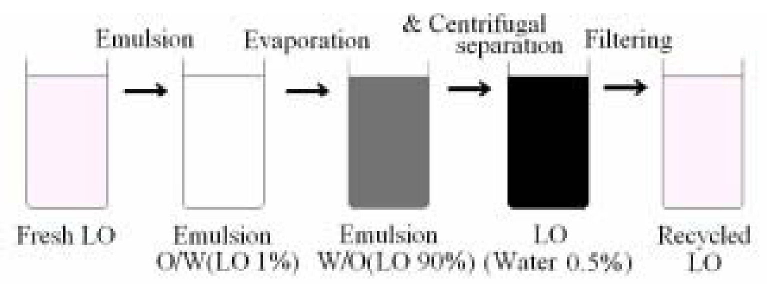
Figure 9 (1) shows fresh LO of NAS grade 9.

Figure 9 (2) is waste water of O/W emulsion with 99% water content from air compressors of the fertilizer plant. It has milky appearance with 1% oil content in water.

Figure 9 (3) shows heated and condensed waste water after process 1, in which steam heats waste water to around 100~110°C. At this point, it is emulsion of water in oil with 10% water content. It is opaque and light brown in color.

Figure 9 (4) shows a sample after process 2 in which almost all water is evaporated and rough sludge removed by centrifugal separator. It is black due to the presence of many fine sludge particles.

Figure 9 (5) shows finally recycled LO after process 3, in which fine sludge particles are removed by nephron filter. As shown, the recycled LO is transparent and cleaner than fresh LO: recycled LO is of NAS grade 8 in comparison with NAS grade 9 of fresh LO.



(1) (2) (3) (4) (5)

Figure 8: Schematic view of change.

Clean	Milky white	Light brown	Black	Transparent
(1)	(2)	(3)	(4)	(5)

Figure 9: Results of recycling process.

Table 3: Analysis of compressor oil.

Item	Fresh oil	Waste oil After Process 1	Recycled oil
Density g/cm	0.881	0.925	0.894
Flush point ℃	285	—	275
Kinematic viscosity cSt , 40℃	149.2	—	150.2
Total acid number mgKOH/g	0.17	0.60	0.18
Base number mgKOH/g	1.50	0.45	1.30
Water %	0	12.5	0.02
Sludge mg/100mL	0	35	10

6 Discussion

6.1 LO recycling

Table 3 compares: 1) fresh LO; 2) waste emulsion after process 1, in which more than 90% water is evaporated; and 3) recycled LO. Waste LO differs from fresh LO in all measured items; it contains 12.5% water because process 1 evaporates 90% water. It includes as much as 35 mg/100 mL sludge that may have been generated in the air compressors.

On the other hand, recycled LO is almost the same as fresh LO, except for sludge 10 mg/100 mL. According to the present standard, the nephron filter is renewed when recycled LO contains sludge 12 mg/100 mL. Immediately after filter replacement, there is no sludge at all and recycled LO appears cleaner than fresh LO.

The results of the field test verify that the recycling system shown in figs. 6 and 7 enables complete recycling of waste O/W emulsion from air compressors at the fertilizer plant. The success of the system has led to the following results:

1) All water in waste O/W emulsion is evaporated by the extra steam available in the plant.
2) No waste water polluted with oil is exhausted out of the plant, alleviating fears that polluted waste water could be thrown out into the environment, especially into nearby rivers and coastal sea.
3) Recycling of waste LO reduced LO consumption to about 1/10.

6.2 Closed plant system with no waste water and no waste oil

The authors solved the problem of possible pollution of rivers and neighboring coastal sea with waste oil from compressors at a Japanese fertilizer plant. Since that time, laws for environment protection have been tightened and the situation is better than in former times.

The danger remains, however, that pollution of rivers and coastal sea may occur at any time by spillage or slick of oil. LO is widely used at many plants and workshops, and oil-containing waste water from these facilities could flow into rivers and coastal sea.

To solve such problems, systems that do not produce any waste oil and waste water are desirable. To this end, the authors constructed a plant that produces no waste oil and waste water in Thailand. This was realized by combining LO kidney system and water kidney system.

Water kidney system utilizes rain water and recycles waste water from the plant. An artificial river built around the plant biologically cleans the waste water. Biological cleaning means that various plants, aquatic fauna, and bacteria in the river clean the waste water without any chemical processing.

6.3 Reduction of oil consumption expected by UK government

The science of tribology was started according to a report by a committee sponsored by the UK Government in 1964 [9]. In this report, the committee expected that LO lifetime should be lengthened and LO consumption reduced by about 20%. The authors have succeeded in lengthening LO life semi-permanently and in reducing LO consumption to about 1/5~1/10 in engines and other machines.

7 Conclusion

The authors have successfully developed a technology called a kidney system that enables semi-permanent use of lubricating oil without producing any waste oil. It also enables to reduce oil consumption to 1/5～1/10 in comparison with the conventional system. By applying the kidney system to recycle waste O/W emulsion from air compressors at a fertilizer plant in Japan, the following results were obtained:

1) Waste LO was separated from waste emulsion and completely recycled.
2) Thus, the recycling system has been preventing possible pollution of the neighbouring rivers and coastal sea since 1996.
3) Recycling of LO has greatly reduced LO consumption to about 1/10 at the fertilizer plant.

References

[1] Morio, S. & Tadanori, A., Reduction of operation and maintenance cost by technology for clean engine, *Pro. of the 7th International Symposium on Marine Engineering,* Tokyo, October 24th to 28th, 2005, 33-3, Paper 56.
[2] Somchet Koomsorn et al, A No Waste Water Plant with Minimum City Water Supply Utilizing and Recycling Rain Water, *6th International Conference on The Environmental Management of Enclosed Coastal Seas (EMECS),* 2003, November 18~21, 2003, Bangkok, Thailand, Abstract, pp141.
[3] Investigation Report on Recycling Petroleum, Japan Institute of Applied Energy (March 1992), pp62-63.
[4] Investigation Report on Recycling Lubricating Oil, Japan Lubricating Oil Society (March 1997), pp72-92.
[5] The Asahi Shinbun (Japanese newspaper), Wednesday, October 22, 1997, Evening edition.
[6] The Yomiuri Shinbun (Japanese newspaper), Wednesday, November 11, 1998, Evening edition.
[7] Taizo, S., Kyoji, T. & Toshiyuki, Y., *Physiology for university students*, Nonkoh-dou (September 1995), pp204.
[8] Douglas, H. Everett, *Science of Colloid* (in Japanese), Kagaku, Dojin (1992), pp5.
[9] Kazuo, K. *The world of friction*, Iwanami Sinsyo in Japan (1994), pp169.

Section 9
Irrigation problems

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Simulation at regional level of irrigated wheat and tomato in a Mediterranean environment

M. Rinaldi & R. Ubaldo
CRA – Istituto Sperimentale Agronomico, Italy

Abstract

In Mediterranean countries, where water-limited conditions are frequent, it is important to identify soil and crop management which optimize resources transformation and maximize farmers' return. In this research a seasonal merged with a spatial analysis was simulated with AEGIS/WIN, a Geographic Information Systems (GIS) interface from the DSSAT crop simulation package. The case-study refers to a 1000 km^2 area (Southern Italy), characterized by 481 soil samples collected at a regular grid. Durum wheat and processing tomato have been simulated punctual-based using soil and long-term weather data (45 years). The two crops have been compared in the following management scenarios: rainfed and three automatic irrigation levels based on soil water content thresholds. Averages and standard deviations of commercial yield (grain and fruit), seasonal irrigation amount, number and profitability were evaluated as model output. GIS allowed one to visualise the output variables in the soil polygons. The wheat productivity was increased by irrigation of 19% and no difference occurred among automatic irrigation thresholds. In tomato the irrigation increased the yield by 3 times with respect to rainfed, with no difference among irrigation scenarios. Net return of wheat was higher in rainfed than irrigated scenarios also for the cost of water and the low price of the product. For tomato CAW 50% resulted in the most profitable scenario. The "soil x climate" interaction (rainfall, temperature and soil water holding capability) influenced the spatial response at regional level, allowing us to identify the area more productive for wheat and tomato.
Keywords: simulation model, durum wheat, processing tomato, irrigation scenarios, soil water content, net return.

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1 Introduction

Water resource allocation at district or basin scale, and environmental impact assessment of agriculture activities are typical issues needing of a decision support system where biophysical processes and human interactions such as adaptive changes of agricultural practice have to be modelled. Crops simulation models should appreciate soil-climate-crop interactions, giving suggestion to stakeholders for a better water allocation, also from an economic point of view. Crop models usually need site specific characteristics such as weather, physical and chemical parameters of soil, water management, and agronomic practices [1] as input data. Applicability of these models can be extended to much broader spatial scales by combining them with a Geographic Information System (GIS) [2–5].

Experiences of application of DSSAT software at spatial scale are reported by Hoongenboom and Thornton [4] that applied GIS to bean, maize and sorghum crop models. Calixte *et al* [6] developed an Agricultural and Environmental Geographic Information System (AEGIS), which combined DSSAT crop models with GIS to assess the impact of different agricultural practices of Puerto Rico. Georgiev *et al* [7], Heinemann *et al* [8], Batchelor *et al* [9] and Nijbroek *et al* [10] reported further applications of DSSAT at spatial scale, especially for water requirement estimation.

In previous studies the crop models embedded in DSSAT software have been calibrated and validated for Southern Italy conditions [11–13]: it revealed to be a good tool in simulation of field crops in several soil and climatic conditions; large number of users and the upgrade with user-friendly interface and new applications are further reasons to choose DSSAT software. The seasonal analysis [14] has an economic module that allows an economic evaluation useful to compare management scenarios and geographical areas also from an economic point of view.

In this paper we reported the results of a seasonal and spatial simulation of CERES-Wheat and CROPGRO models for two important field crops in Southern Italy in a large area. The objective of this paper is to apply a GIS-based crop model to compare irrigation strategies in durum wheat and processing tomato, to predict crop yield and economic profitability.

2 Material and methods

The “Capitanata” is a plain of about 4000 km^2 in South-Eastern Italy, mainly cropped with durum wheat, tomato, sugar beet, olive and grape orchards. Irrigation is managed by a local authority “Consorzio per la Bonifica della Capitanata” of Foggia that distributes irrigation water on demand and at low pressure (2-3 bar) at a large part of the plain (1800 km^2). A part of this plain (about 1000 km^2) has been characterized from pedological and climatic point of view. A large number of soil samples (481) were collected at 0–20 and 20–40 cm depth and 115 soil profiles were examined up to 2.5 m depth. The main chemical and physical characteristics were recorded (texture, hydrological characteristics, nitrogen and phosphorus content, organic matter, bulk density, etc.). Daily climatic data (maximum and minimum temperature, solar radiation and rainfall) derived by eight meteorological stations located in the area and managed by the above reported "Consorzio" (Fig. 1).

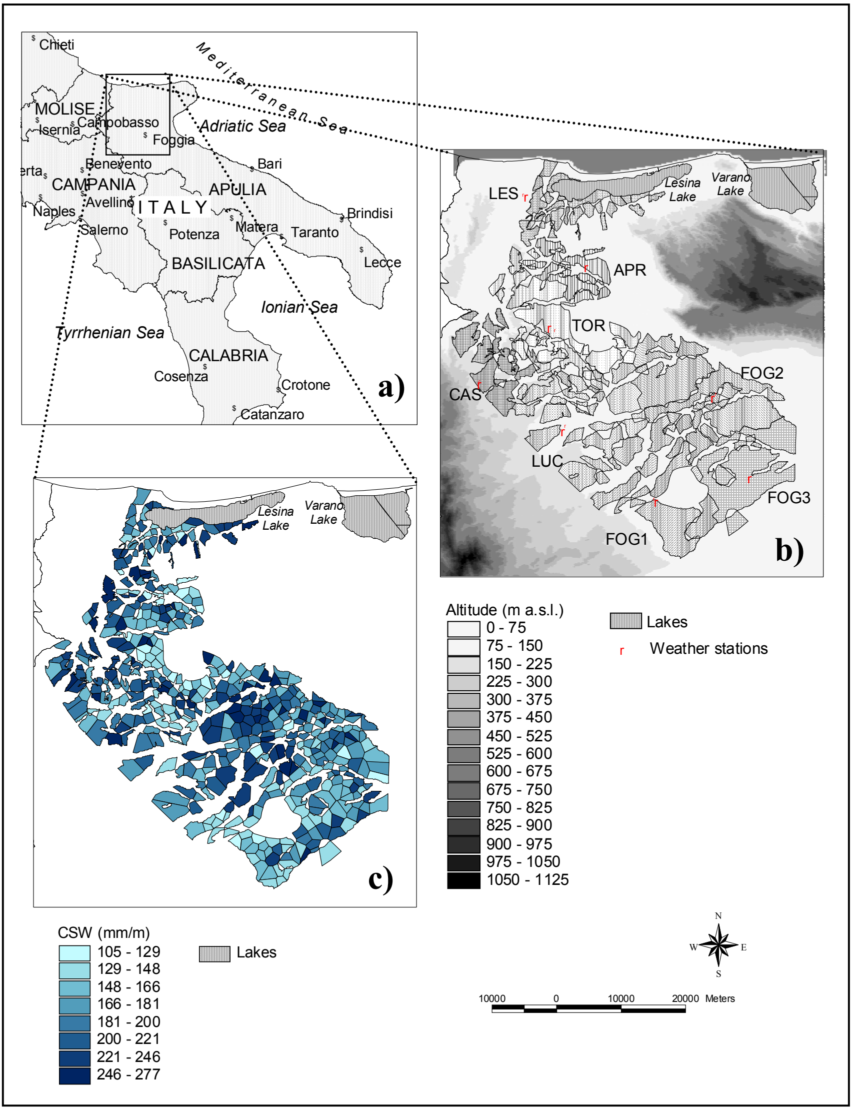

Figure 1: a) Localization of the test area in Southern Italy; b) clustering of the 481 soils round the 8 climatic stations and altitude map of the area as background; c) crop soil water availability (mm m^{-1}) for each soil polygon.

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CERES–Wheat model, embedded in DSSAT program [15], previously calibrated and validated for durum wheat (cv. Simeto) in the test area [12, 16] was used in a seasonal (44 cropping cycles, from 1955 to 1999) and spatial analysis comparing the following irrigation scenarios:

1. Rainfed;
2. Automatic irrigation starting at 10% of crop available water (CAW) in the 0.3 m soil depth (IRR10), with water amount refilling up to field capacity, until head emission stage; a sprinkler irrigation method was used;
3. Conditions as above at 30% (IRR30);
4. except at 50% (IRR50).

The use of low thresholds to start automatic irrigation derive by the fact that in the test area durum wheat is usually not-irrigated or irrigated occasionally (1-2 applications) in the spring. Durum wheat management was simulated with fixed sowing date (15th November), fertilisation with 100 kg ha^{-1} of ammonium phosphate pre-sowing and 100 kg ha^{-1} of ammonium nitrate at 1st March. Harvest date was simulated by the model at crop maturity.

CROPGRO model, embedded in DSSAT program, has been calibrated and validated in the test area for a processing, self pruning, globe shape, tomato variety (PS 1296 [13]). The simulation was run for the same years and location as before described for wheat, and similarly for the irrigation scenarios, except for thresholds of soil crop available water to start automatic irrigation, fixed for tomato to 30 (IRR30), 50 (IRR50) and 70% (IRR70).

Tomato crop, according to local management, was simulated with fixed sowing date (30th April), fertilization with 100 kg ha^{-1} of ammonium phosphate pre-sowing and 100 kg ha^{-1} of ammonium nitrate at fruit formation (30th May). Harvest date was simulated by the model at crop maturity.

The 481 referenced points have been converted in polygons using the Thiessen methods (threshold value = 5) and overlaying these polygons with a soil map with pedological characteristics [17]. The interface with a GIS program, AEGIS/WIN, allowed to run the model in the 481 polygons and to display the output of the model using map visualization [18]. The total of run was 21164 for wheat and 21645 for tomato and each polygons represent the average of 44 (45 for tomato) yearly values.

The economic evaluation was performed using seasonal analysis tool, and Net Return (NR) was calculated with prices and costs reported in Table 1. Yield (t ha^{-1}) and net return (€ ha^{-1}) for each polygon (soil-climate interaction) were mapped to visualize spatial variability for both crops.

3 Results and discussion

The climatic stations are located in plain area: CAS only is placed on a smooth hilly (177 m a.s.l.), while LES is very close to homonym lake and Adriatic sea coast. The coldest and rainiest place is CAS, the warmest are FOG2, FOG3 and LUC; the less rainy locations are FOG1 and FOG3.

Table 1: Prices and costs of durum wheat and processing tomato field crops in Southern Italy (referred to 2006).

Crop	Operation/product and harvest product	Unit	Cost/price
Durum wheat	Grain price	(€ t^{-1})	160,00
	Base production cost	(€ ha^{-1})	380,00
	Water irrigation cost	(€ mm^{-1})	0,70
	Irrigation application cost	(€/application)	50,00
Processing tomato	Fresh fruit price	(€ t^{-1})	60,00
	Base production cost	(€ ha^{-1})	5500,00
	Water irrigation cost	(€ mm^{-1})	1,00
	Irrigation application cost	(€/application)	15,00

The main difference of 481 polygons derived by the soil texture that influenced the hydrological characteristics (wilting point and field capacity). The crop soil water (CSW, difference between Field capacity and Wilting point), expressed in mm m^{-1} of soil depth, is mapped in Fig. 1. A distribution of soils with greater CSW (mainly clay soils) was noticed in the central part of the tested area, close to FOG2 and LUC weather stations, while sandy soils were located in the inner part (FOG1) and close to TOR and APR stations (Fig. 1).

3.1 Wheat

Wheat is usually not irrigated in the test area but, in the farms were irrigation sprinkler equipment is available, 1-3 irrigation supplies at sowing and at boot stage are frequent to increase and stabilize grain yield.

The simulation of wheat cropped without irrigation (rainfed scenario) produced an overall mean of 3.0 t of grain yield ha^{-1} (Table 2), with a large variability ranging, at single run simulation level, from 0.3 to 6.4 t ha^{-1}. These simulated values by CERES-Wheat model are not so different by local long-term average. In general, the areas more productive resulted the central and northern parts, the less yielding the southern one (FOG1 and FOG3) (Fig. 2).

The application of automatic irrigation reduced the variability of grain yield (on average coefficient of variation decreased from 43 to 33%) and increased the grain yield on average of 19% respect to rainfed scenario (+ 0.58 t ha^{-1}) (Table 2). The effect of irrigation was not different among the three threshold levels of CAW, showing a parity of yield, despite a difference of seasonal irrigation volume was observed.

The explanation of this result comes from the examination of water balance component: the only one that changed was the crop available soil water at harvest, meaning that the latest irrigation supplies were not used completely by the crop and remained in the soil. This is a normal risk in irrigation practice, when rain events following the irrigation, make vain the irrigation supply.

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The re-initializing of soil condition at every starting date of simulation (two days before sowing) did not allow to one consider this beneficial effect of the previous year.

Table 2: Yield, irrigation and economic results (averages ± standard deviations) of the CERES-Wheat and CROPGRO models, referred to the 481 soils and to the 44 (45 for tomato) simulated years.

Durum wheat				
Scenario	Grain yield ($t\ ha^{-1}$)	Seasonal irrigation volume (mm)	Irrigation applications (n.)	Net return (€ ha^{-1})
Rainfed	3.03 ± 1.30	-	-	105 ± 207
IRR10	3.57 ± 1.18	138.5 ± 72.4	2 ± 1	-11 ± 172
IRR30	3.64 ± 1.18	183.0 ± 73.1	3 ± 1	-94 ± 171
IRR50	3.62 ± 1.18	208.8 ± 71.9	5 ± 2	-193 ± 188
Processing tomato				
Scenario	Fresh fruit yield ($t\ ha^{-1}$)	Seasonal irrigation volume (mm)	Irrigation applications (n.)	Net return (€ ha^{-1})
Rainfed	54.3 ± 39.3	-	-	-2374 ± 2357
IRR30	187.1 ± 28.1	298.6 ± 76.8	6 ± 2	5214 ± 1685
IRR50	198.0 ± 25.9	342.8 ± 79.5	9 ± 2	5778 ± 1556
IRR70	197.4 ± 27.0	398.2 ± 76.7	15 ± 4	5597 ± 1621

Economic evaluation pointed out the low profitability and the high economic risk of durum wheat crop both in the rainfed scenario and in the irrigated ones. The overall mean of expected net return (NR) resulted negative in the irrigated thesis, decreasing inversely with the CAW threshold values (Table 2). Moreover, a large variability of NR was observed, mainly in the rainfed scenario, more affected by the yearly variability of rainfall. Negative NR values were registered in about 50% of the soils for the irrigation scheduling IRR10; the number of soils for which irrigation resulted disadvantageous in economical terms increased for IRR30 and concerned the whole test area for IRR50 (Fig. 3). So the results of simulation showed the little increase in grain yield and stability due to irrigation but also highlighted the high incidence of irrigation cost on the global crop management cost. Nevertheless, negative values of NR were observed also in more than 50 soils in the rainfed scenario, mainly located in the southern part of the area, resulted the less productive.

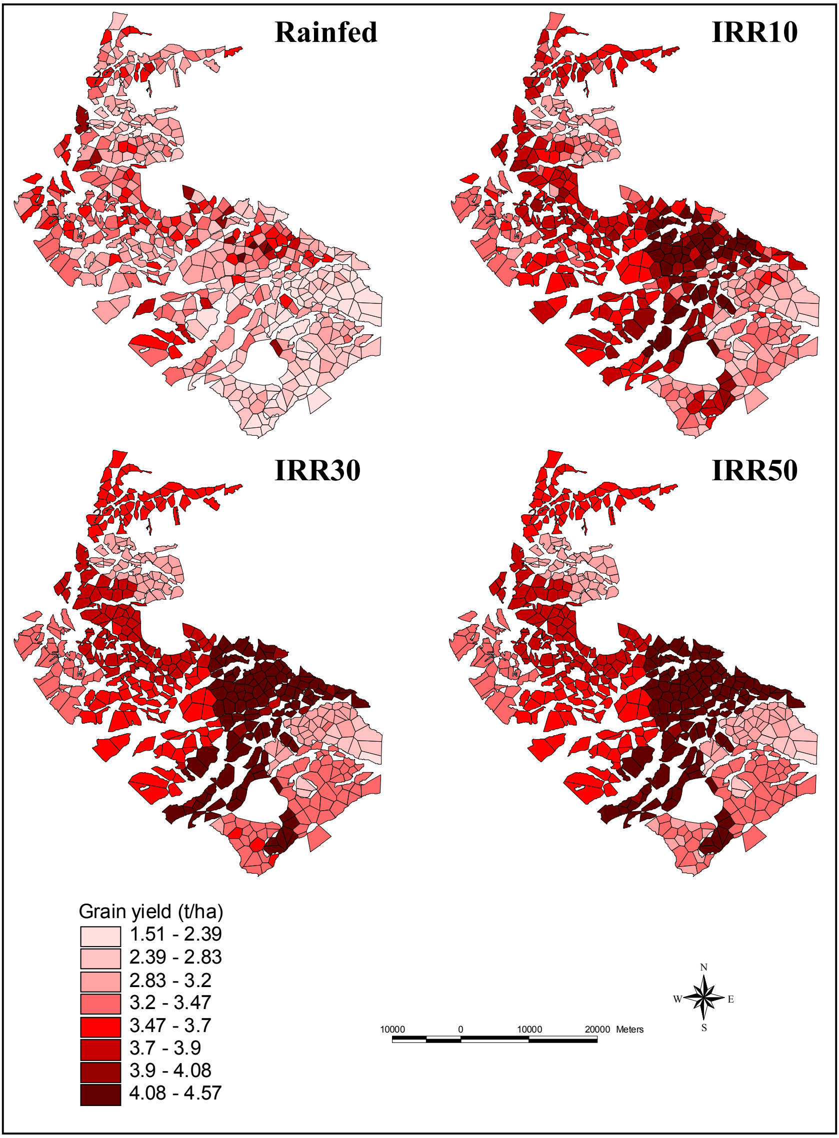

Figure 2: Grain yield (t ha^{-1}) of durum wheat simulated by CERES-Wheat model in the four irrigation scenarios and mapped for the 481 soil polygons.

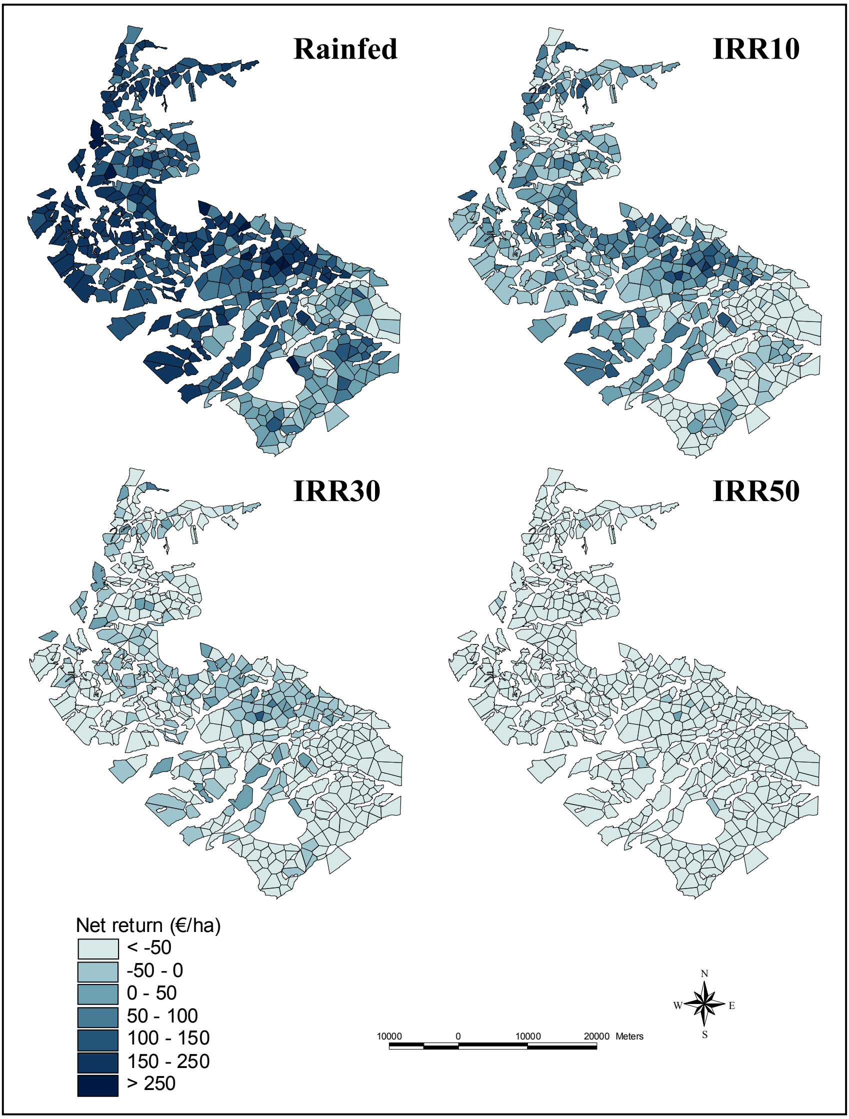

Figure 3: Net return (€ ha^{-1}) of durum wheat simulated by CERES-Wheat model in the four irrigation scenarios and mapped for the 481 soil polygons.

3.2 Tomato

Tomato crop is usually irrigated in test area, with sprinkler and, more widely, with drip irrigation methods; seasonal irrigation volumes range between 300 and 500 mm. In this simulation activity, the seasonal irrigation volume ranged from

300 to 400 mm in the three irrigation scenarios (Table 2), but we choose low CAW thresholds with the specific aim to reduce water application.

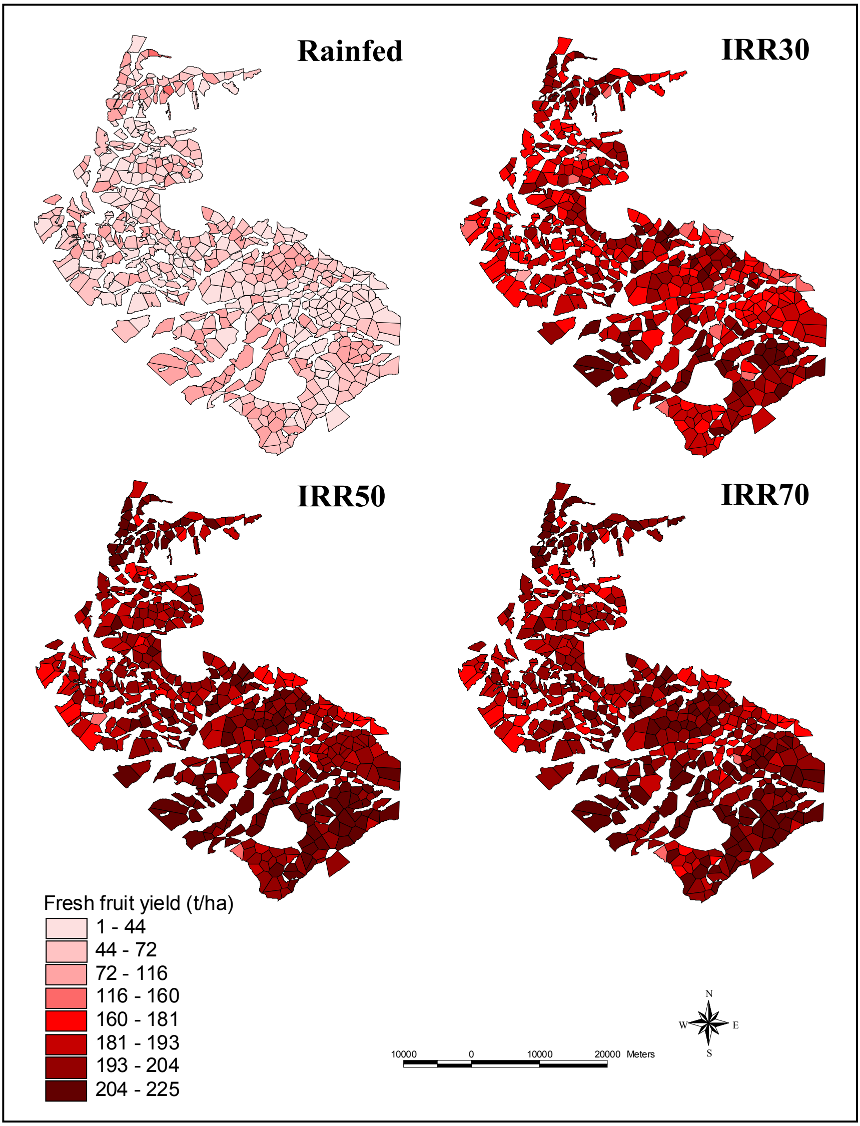

Figure 4: Fresh fruit yield (t ha^{-1}) of processing tomato simulated by CROPGRO model in the four irrigation scenarios and mapped for the 481 soil polygons.

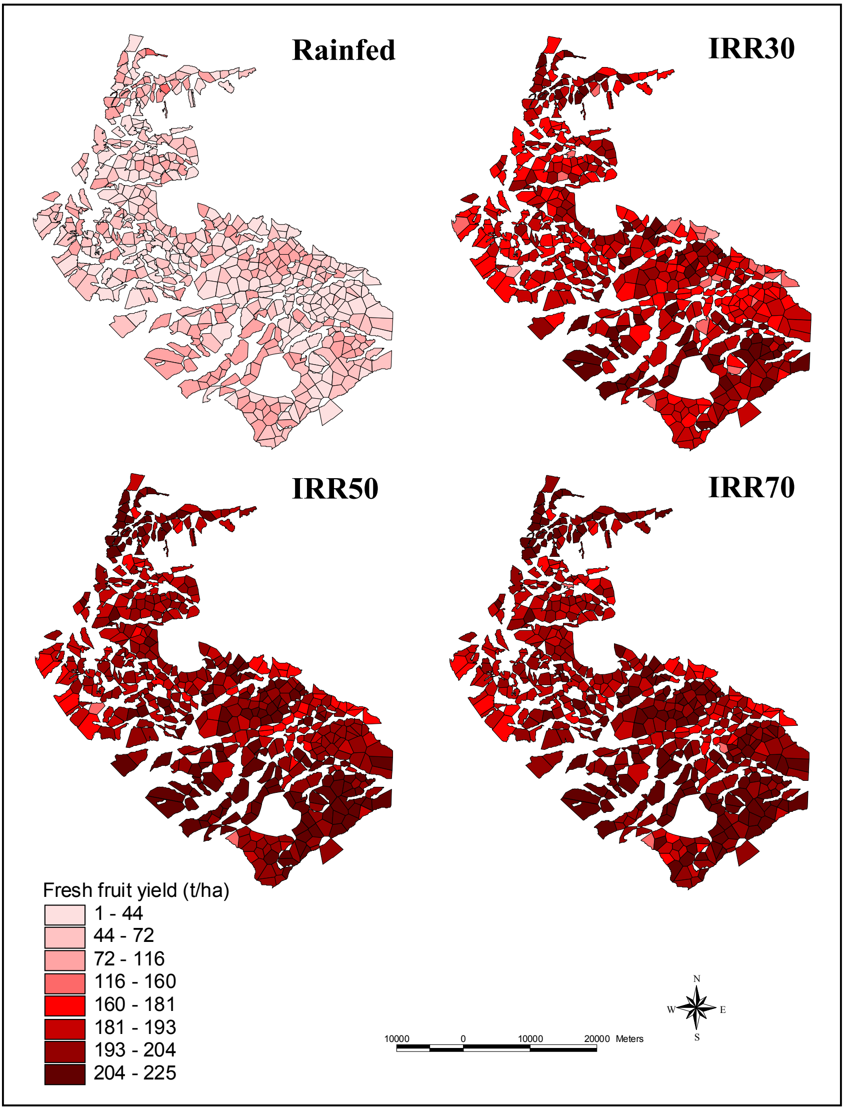

Figure 5: Net return (€ ha^{-1}) of processing tomato simulated by CERES-Wheat model in the four irrigation scenarios and mapped for the 481 soil polygons.

Fresh fruit yields in the irrigated scenarios were generally higher than those usually recorded in the test area (from 50 to 150 t ha^{-1}), but this overestimation is explained because the model does not consider the effect of pest damages and weed competition. The fruit yield of rainfed scenario was very low, 54 t ha^{-1} on average and with a very large variability (Table 2), depending by rainfall, erratic in test area during the crop grown period (May-August). The irrigation stabilized the yield and few differences among the soils were noticed: a light superiority was observed in the areas close to LES station and in the southern part (Fig. 4). In the first case the effect is due to mitigation of the climate (especially during the summer) due to lake and sea proximity and to the richness in organic matter of the alluvial soils; in the second case the hydrological soil characteristics, mainly a greater CSW (Fig. 1), allowed a better tomato yield level and stability.

The effect of irrigation resulted in general the same in the area (Fig. 4) and was markedly evident, with fresh fruit yield three times greater than rainfed scenario. Further, the yearly variability of fruit yield was reduced with irrigation application with a lower standard deviation in irrigated crop (Table 2).

The different irrigation scenarios highlighted a variation in the irrigation volume and number of irrigation supplies, but not in fresh fruit yield, quite uniform among the scenarios and in the 481 soils (Fig. 4).

Economic analysis highlighted the significant advantage of irrigation in tomato crop, with a significant increase of NR (on average +7800 € ha^{-1} than rainfed) and a reduction of its variability (Table 2); the large profitability derived by a clear yield increase due to irrigation, also at the lowest CAW value, according to other simulations carried out in the same area [13]. The rainfed scenario is not too suggestible, because it showed negative NR in the totality of the years and in more than 80% of the soils. The incidence of irrigation in tomato production cost is low (7÷10% of total cost) and for this reason the NR followed the yield spatial variability, depending largely by productivity.

4 Conclusions

Spatial and temporal analyses have been carried out to visualize the most productive and profitable pedo-climatic areas for wheat and tomato crops, when submitted to different irrigation scenarios. DSSAT models, coupled with AEGIS/WIN, allowed one to run long-term simulation and check the locations where the two crops give higher yields and net returns.

The climatic conditions (elevation and sea influence) and soil hydrological characteristics (mainly soil crop available water) influenced crop productivity, especially in wheat. The irrigation scenarios revealed a minimum effect of irrigation on durum wheat (+19%) with no difference among irrigation scenarios. The simulated rainfed scenario productivity matched well with local averages yield and the areas more yielding resulted the central and northern ones.

The conclusion for wheat in the test area is that irrigated wheat is not convenient for the high cost of water and labour and for the low price of product. In fact, the incidence of irrigation cost is higher than tomato and ranges from 35% (IRR10) to 52% (IRR50). Rainfed wheat is a low profit crop management, but important in the farms adopting a rotation with irrigated crops, to allow a positive (agronomic) sequence of different crops on the same land.

The simulation of processing tomato showed a very low productivity in rainfed scenario and high yield in irrigation treatments also at CAW 30%.

The two crops highlighted different responses to irrigation practice: for the wheat the increasing of grain yield respect to rainfed was not enough to compensate the irrigation cost. On the contrary, irrigation in tomato increased three times fruit yield than rainfed scenario, showing a positive net return (NR > 5000 € ha^{-1}). The soil-climate characteristics influenced mainly wheat response and less tomato yield, because this latter is more dependent by irrigation water than rainfall.

Further development of this research will be the application of geostatistical analysis to obtain larger homogenous areas and the checking of vulnerable areas for environmental aspects (leaching, pollution, drought, desertification) using improved simulation models. In the future, this kind of decision support systems could be used by stakeholders to plan agricultural land use and water distribution, simulating water requirement and crop yield at regional level.

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References

[1] Penning de Vries, F.W.T., Jansen, D.M., ten Berge, H.F.M. & Bakema, A., *Simulation of ecophysiological processes of growth in several annual crops*. Pudoc, Simulation Monographs 29: Wageningen, The Netherlands, pp. 271, 1989.

[2] Dent, J.B. & Thornton, P.K., The role of Biological Simulation Models in Farming Systems Research. *Agricultural Administration and Extension*, **29**, pp. 111-122, 1988.

[3] Hartkamp, A.D., White, J.W. & Hoogenboom, G., Interfacing geographic information systems with agronomic modelling. A review. *Agronomy Journal*, **91**, pp. 761–772, 1999.

[4] Hoogenboom, G. & Thornton, P.K., A GIS for agrotechnology transfer in Guatemala. *Proc. Application of Geographic Information System, Simulation Models, and Knowledge-based Systems for land use management,* Blacksburg, VA. 12–14 Nov. 1990. Va. Polytechnic Inst. & State Univ., Blacksburg, pp. 61–70, 1990.

[5] Thornton, P.K., Saka, A.R., Singh, U., Kumwenda, J.D.T., Brink, J.E. & Brisson, N., Application of a maize crop simulation model in the central region of Malawi. *Experimental Agriculture*, **31**, pp. 213–226, 1995.

[6] Calixte, J.P., Beinroth, F.H., Jones, J.W. & Lal, H., Linking DSSAT to a GIS. *Agrotechnology Transfer*, **15**, pp. 1–7, 1992.

[7] Georgiev, G.A., Hoogenboom, G. & Ragupathy, K., Regional yield estimation using a linked geographic information system, crop application of crop models and GIS. *Agronomy abstracts Biol. Eng*., 1st IBE Publ., Athens, GA, p. 63, 1998.

[8] Heinemann, A.B., Hoogenboom, G. & de Faria, R.T., Determination of spatial water requirements at county and regional levels using crop models and GIS. An example for the State of Parana, Brazil. *Agricultural Water Management*, **52**, pp. 177-196, 2002.

[9] Batchelor, W.D., Basso, B. & Paz, J.O., Examples of strategies to analyze spatial and temporal variability using crop models. *European Journal of Agronomy*, **18**, pp. 141-158, 2002.

[10] Nijbroek, R., Hoogenboom, G. & Jones, J.W., Optimizing irrigation management for a spatially variable soybean field. *Agricultural Systems*, 76, pp. 359-377, 2003.

[11] Rinaldi, M., Flagella, Z. & Losavio, N., Evaluation and application of the OILCROP-SUN model for sunflower in southern Italy. *Agricultural Systems*, **78**, pp. 17-30, 2003.

[12] Rinaldi, M., Water availability at sowing and nitrogen management of durum wheat: a seasonal analysis with CERES-Wheat model. *Field Crops Research,* **89**, pp. 27-37, 2004.

[13] Rinaldi, M., Ventrella, D. & Gagliano, C., Comparison of nitrogen and irrigation strategies in tomato using CROPGRO model. A case study from Southern Italy. *Agricultural Water Management*, **87**, pp. 91-105, 2007.

[14] Thornton, P.K., & Hoogenboom, G., A computer program to analyze single-season crop model outputs. *Agronomy Journal*, **86(5)**, pp. 860-868, 1994.

[15] Jones, J.W., Hoogenboom, G., Porter, C.H., Boote, K.J., Batchelor, W.D., Hunt, L.A., Wilkens, P.W., Singh, U., Gijsman, A.J. & Ritchie, J.T., The DSSAT cropping system model. *European Journal of Agronomy*, **18**, pp. 235-265, 2003.

[16] Rinaldi, M., Durum wheat simulation in Southern Italy using CERES-Wheat model. I. Calibration and validation. *Proc. of 2nd International Symposium "Modelling Cropping Systems"*: Florence (Italy), 16-18 July, pp. 81-82, 2001.

[17] Hartkamp, A.D., de Beurs, K., Stein, A. & White, J.W., Interpolation techniques for climate variables, NRG-GIS Series 99-01. CIMMyt, Mexico, DF., 1999.

[18] Engel, T., Hoogenboom, G., James, W.J. & Paul, W.W., AEGIS/WIN: A computer program for the application of crop simulation models across geographic areas. *Agronomy Journal*, **89**, pp. 919–928, 1997.

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Field evaluation of drip irrigation systems in Saudi Arabia

H. M. Al-Ghobari
Agricultural Engineering Department, College of Agriculture and Food Sciences, King Saud University, Saudi Arabia

Abstract

Drip irrigation has become a well-established method for irrigating high-value crops in regions, where water resources are scarce and/or expensive. In Saudi Arabia, the use of drip has increased fivefold since the 1990s, and now due to the rapid increase in the use for drip irrigation system in the fields and greenhouses in recent years in most agricultural regions in the Kingdom has caused the availability of many drip irrigation components and products in the local market made by different manufacturers. These devices differ in their qualities and standards. Hence, the field evaluation of drip irrigation systems in the fields and its components (i.e. emitters) is essentially required for standing the efficiency and performance of the system and these devices during irrigation.

This study was mainly focusing on evaluating 10 drip irrigation systems under field operation located in different farms in Riydh region of Saudi Arabia. Also, the study included a survey study for 50 drip irrigation systems distributed on a number of farms in the most agricultural regions in Saudi Arabia using drip irrigation systems. Results of field evaluation show that the irrigation performances are mostly lower than the accepted values for most evaluated systems and that the ten systems were varied in their uniformities of the applied water.
Keywords: drip system, water uniformity coefficient, emitter clogging, field evaluation, water distribution, water applied, Saudi Arabia.

1 Introduction

With population growth in the world, the demand of water is increasing and in many parts of the world there are major concerns regarding the sustainability of water resources for irrigated agriculture. Hence, the necessity for conservation of

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water resources increases, particularly in countries of limited water supply, where the agricultural irrigation has traditionally been the major water use sector in these areas, usually in the range of 80 to 90%. There is widespread acceptance that part of the solution lies in improving agricultural productivity and water use efficiency, and obtaining more “crop per drop” (FAO [1]). For this reason, drip (i.e. drip) irrigation is being heavily promoted, often by regulators and governments as well as the drip industry. Compared to traditional surface or overhead methods, drip irrigation offers the potential for greater water use efficiency (Al-jamal et al [2]) and has often been reported to produce crops of higher yield and quality (Mantovani [3]). Despite its higher costs. These characteristics make drip an attractive option in regions where irrigation water resources are scarce and/or expensive. Therefore, the number of drip irrigation systems has increased rapidly in the last two decades in the Kingdom of Saudi Arabia as automatic and modern irrigation systems.

To understand the current state of the drip irrigation sector in Saudi Arabia, it is useful to briefly consider how it has evolved over the last two decades. Drip irrigation was introduced commercially into the glasshouse sector in the 1980s. Its use spread into outdoor soft fruit, notably on orchard fruit and later into date palm trees, where high returns could justify the high capital investment. In the mid 1990s there was a growing interest in the use of drip for field-scale vegetable production due to product improvements and the introduction of low-cost drip tape. Its appropriateness for specific crops such as tomatoes was tested in a number of commercial small-scale field trials. However, these did not lead to widespread use on field vegetables, because of its relatively high cost then compared to center pivot sprinkler irrigation. During the 1990s there was significant growth in the amenity and landscape sectors, alongside continued expansion in the horticultural sector, underpinned mainly by consumer demand for salads and other rich vegetable crops. More recently, there has again been growth in the field-scale vegetable sector.

Drip or Drip irrigation system is widely used nowadays, in many parts of the country, due to the many advantages that the system offers, compared to other irrigation methods, its potential for conserving water in a country suffers shortage in water resources, energy and labor in addition to the reported results of higher yield and better qualities of irrigated crops (Bralts [4]. Proper system design, management and maintenance are essentials for higher Irrigation efficiency (Bucks et al [5]). Although nonuniformity of water distribution by a drip system (low efficiency), may be attributed to many factors, the hydraulic characteristics of emitters are considered the most important of these factors (Bucks and Davis [6]). The variation in water distribution by emitters may occur due to pressure changes, manufacturing variations, emitter sensitivity to clogging, temperature effect and others.

Therefore, the improvement of irrigation water management is becoming critical to increase the efficiency of irrigation water use and to reduce irrigation water demands. Drip irrigation evaluation in the field under operating conditions is very important to ensure that the desired emitter discharge uniformity required for the system design is met, and to see whether the system could be operated efficiently. Also the results of an evaluation could be used by the maintenance personnel to determine the proper operation of the system and to suggest any maintenance action if required (Bucks et al [5]).

Numerous of investigations and works has been made on the surface distribution of water from emitters (Christiansen [7]; Decroix and Malaval [8]; Merriam and Keller [9]; Karmeli [10] and Zhu et al [11]) and stated the procedures of sprinkler and emitters distribution testing above soil surface (Merriam [9] and ASAE [12]). A necessary step before calculating an applied water distribution parameter is the accurate measurement of applied water from sprinklers using catch cans or collectors (Karmeli et al [13] and Sporre-Money et al [14]). Procedures to determine the distribution of water from different sprinkler systems are given in ASAE Standard (ASAE [12]). The flow characteristics of most emitters are described by the following equation (Merriam and Keller [9] and Karmeli [10]):

$$q = b h^{\beta} \tag{1}$$

where, q = Emitter discharge rate (liter/hr)
b= Emitter discharge coefficient
h = Pressure head at the emitter (kPa)
β = Emitter discharge exponent.

The numerical presentation for the amount of variations in emitter flow rate due to manufacturing processes could be evaluated by the coefficient of manufacturing variation (Solomon [15]), using the following equation:

$$C_v = \frac{S_d}{q_a} \tag{2}$$

$$S_d = \sqrt{\frac{q_1^2 + q_2^2 + \ \dots, \ q_n^2 - nq_a^2}{n-1}} \tag{3}$$

where, C_v = Coefficient of manufacturing variation
S_d = Standard deviation of the discharge rates (liter/hr)
q1,q2, ..,qn = discharge of emitters tested (liter/hr)
q_a = average discharge rate of all the emitters tested (l/hr)
N = Number of emitters tested.

In order to determine whether the system is operating at acceptable efficiency, evaluate the uniformity of emission, which is dependant on the pressure variation at emitters and the coefficient of manufacturing variation, C_v, and could be calculated by the following equation (Wu et al [16]):

$$Eu = \frac{q_n}{q_a} \times 100 \tag{4}$$

where:

Eu = The Emission uniformity (%)
N_p = Number of emitters tested

q_n = average rate of discharge of the lowest one-fourth of the field data emitter discharge readings (l/hr)

q_a = average discharge rate of all the emitters tested (l/hr)

A simplified way to quickly determine a measure of uniformity is to calculate the emitter flow variation (q_{var}) using the following equation:

$$q_{var} = \left(1 - \frac{q_n}{q_m}\right) \times 100 \tag{5}$$

A key element in the design of drip irrigation systems is the close balance between the crop water requirement and the emitter discharge. To properly maintain this balance, it is important that the discharge along a lateral have high degree of uniformity. Quantification of the uniformity is given by the design emission uniformity (EU_D)

$$EU_D = \left(1 - \frac{1.27 \times C_v}{\sqrt{N_p}}\right) \times \frac{q_n}{q_a} \times 100 \tag{6}$$

where:

N_p= number of point source emitters per emission point.

Since the drip irrigation systems have very low flow rates and extremely small passages for water. These passages are easily clogged by mineral particles and organic debris carried in the irrigation water and by chemical precipitates and biological growths that develop within the system. The result of clogging is either the complete or partial stoppage of flow through clogged components (Wu et al [16]; Bucks et al [5] and Bucks and Davis [6]). Therefore, the improvement of irrigation water management is becoming critical to increase the efficiency of irrigation water use and to reduce irrigation water demands. The field evaluation of irrigation systems and in particular drip irrigation systems is essentially required for standing the efficiency and performance of the system during operation. The evaluation data can be useful in indicating any defects regarding system operation, water distribution and water losses (Karmeli [10] and Solomon [15]). Also, the evaluation of the system performance in the field will indicate both the location and magnitude of water losses that are occurring, and then determining how to improve the irrigation system and/or its operation. This problem has a great influence on water availability and conservation and hence on the water resources planning on local and national levels. The objective of this field investigation is to evaluate ten drip irrigation systems under operating conditions in different farms in Riyadh region and to determine their indexes performances and the coefficient of manufacturing variation, and also, to report the most problems are facing the drip system users in Saudi Arabia farms.

2 Materials and methods

Field experiments were conducted on ten drip irrigation systems randomly selected from farms and greenhouses in Riyadh region of Kingdom of Saudi

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Arabia, to evaluate their water distribution uniformity and efficiency under operating conditions.

The distribution of water application depths and discharges from emitters along the lateral were measured using ASAE Standards (ASAE [12]). Specific procedures have been established so results from different fields are comparable (Merriam and Keller [9]). These procedures are based on making measurements of emitter discharge along four lateral lines on a sub main: one at the inlet, one at the far end, and two in the middle at the one-third and two-thirds positions. Four positions are tested on each lateral: one at the inlet, one at the far end, and two in the middle at the one-third and two-thirds positions. This gives a total of 16 measurement positions. Measurements of emitter discharge are made at each measurement position for two adjacent emitters. This is done by measuring the flow volume collected in a graduated cylinder over a one-minute period. The average discharge, minimum discharge and coefficient of variation are calculated using data from the 16 positions. Also, the coefficients of uniformity of water distribution from emitters were determined for each drip system.

Also, a survey study for more than 50 farms using drip irrigation systems located in different regions of Saudi Arabia was carried out. This study was mainly focusing on evaluating the problems and the management of these drip irrigation systems under field operation.

3 Results and discussion

The emitter discharges along each lateral for the four lateral lines on a submain were measured and calculated for each individual drip system in ten farms, and the results are shown in figure 1. The discharge distributions along the lateral line were noticed to be variable as shown in the figure for each drip system. The variation of emitter discharge in a drip system is the result of a variety of factors. The primary factor is hydraulic design, other important factors are emitter type and emitter plugging. It was noticed during measurements there was pressure variation and emitter plugging in some emitters at different farms, also the emitters used in these farms were different in type, age and manufacturers. Also, the average discharge for the ten drip systems were compared and plotted in figure 2. It can be noticed that there was high variation in the average discharges between the ten drip systems evaluated in these farms.

Also, to investigate the emitter flow variation between these drip system, the average discharge and minimum discharge and coefficient of variation are calculated using data from the 16 positions and equations (2) and (3). The obtained curves are shown in figures 3 and 4. The results of flow variation can be used as a simple way to judge the water distribution uniformity from emitters. The general criteria for the emitter flow variation are (a) 10% or less—desirable; (b) 10 to 20%—acceptable; and (c) greater than 20%—not acceptable (Bralts [4]). From figure 3 it can be said that the emitter flow variations (q_{var}) are not acceptable for most drip system evaluated. Also, from figure 4) the system coefficient of manufacturing variation (C_v) for each system for ten drip systems was determined and shown in figure 4). Standards have been developed by the

ASAE to classify emitters based on the value of coefficient of variation. According to this classification, the emitters vary in the values of C_v, and range from 0.03 with excellent quality to 0.35 with an unacceptable quality, which will result in high discharge variations from emitters.

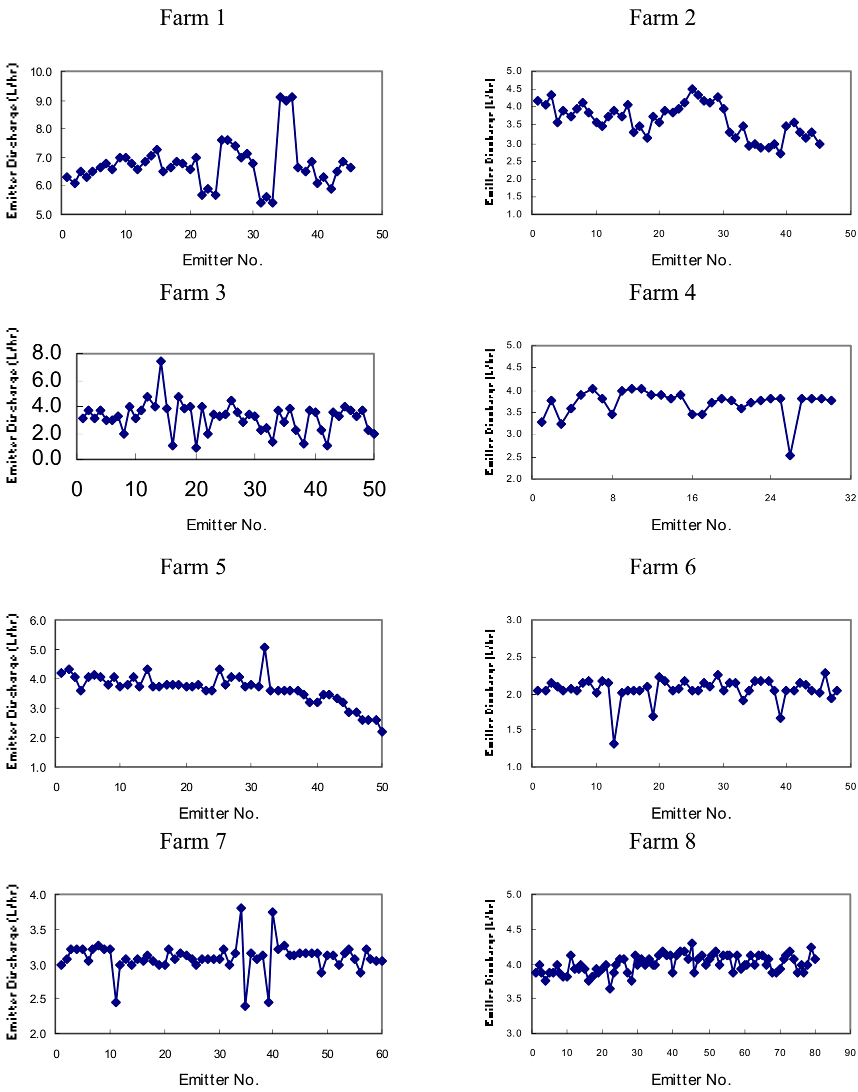

Figure 1: Emitter discharge distribution patterns along the lateral line from different farms.

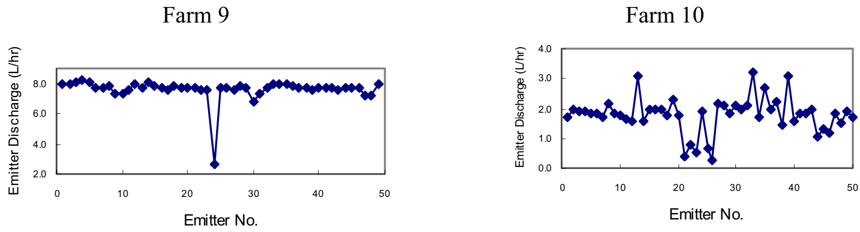

Figure 1: (cont.)

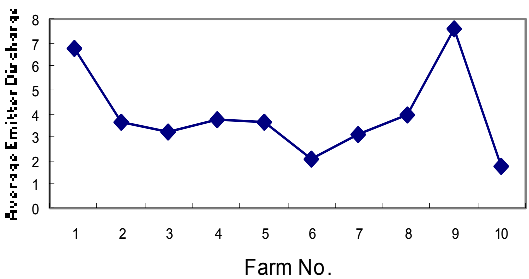

Figure 2: The average discharge measured from ten farms.

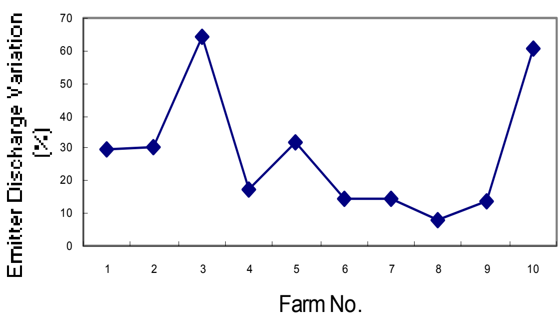

Figure 3: The emitter discharge variation (%) from tem farms.

Also, the uniformity of water distribution from each system emitters can be evaluated by computing the emission uniformity (*Eu*) and the design emission uniformity (EU_D). The *Eu* and EU_D were determined for each system emitters using equations (4) and (5), and the results were presented in figures 5 and 6. It can be noticed from the figures that the values of *Eu* and EU_D for the evaluated drip systems were ranged from 54.42% to 96.05% for the *Eu* and from 30.07% to 92.34% for EU_D respectively. To judge the drip system performance and water distribution from *Eu* and EU_D values, the ASAE developed standards to classify the *Eu* and EU_D values. General criteria for *Eu* values for systems which have

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been in operation for one or more seasons are: greater than 90%, excellent; between 80% and 90%, good; 70 to 80%, fair; and less than 70%, poor. Therefore, the performance of each system at each farm differs from one system to another and their water distribution also varies.

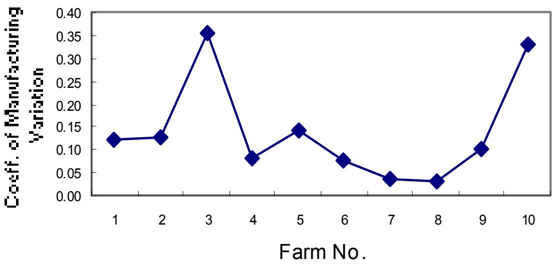

Figure 4: Coefficient of manufacturing variation fro ten farms.

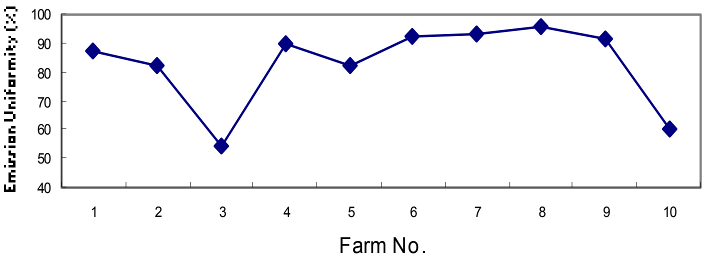

Figure 5: Emission uniformity (EU) for ten drip systems.

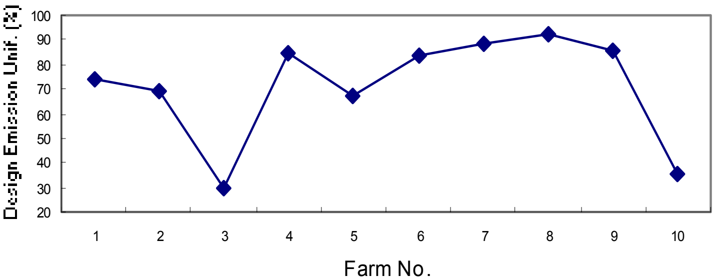

Figure 6: Design emission uniformity for ten drip systems.

Also, the survey study results showed as presented in figure 7 and figure 8 that the number of maintenance and field evaluation at these farms. Generally, it can be noticed that maintenance for the components of drip system is poor, and there was no evaluation for most systems at these farms as can be seen from the figure. Also the survey study showed that the major problems are facing farmers with drip systems are emitter and filter clogging, water leakage at emitter bases and the need for continues maintenance.

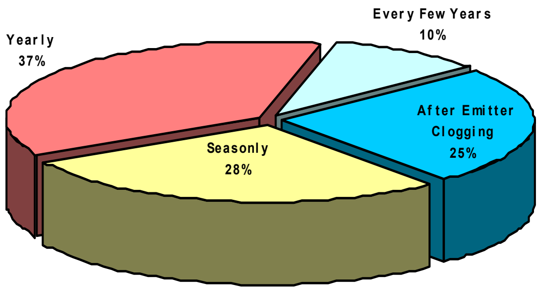

Figure 5: The occurrence of maintenance for drip systems at the farms.

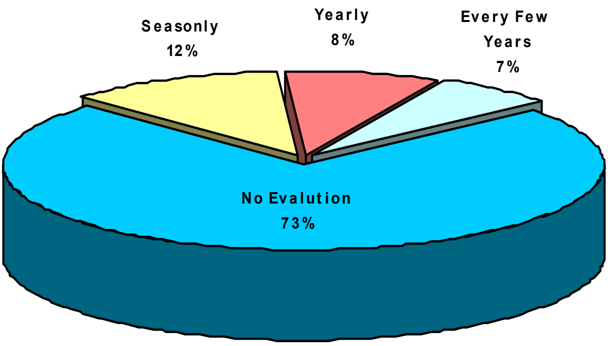

Figure 6: The occurrence of field evaluation of drip systems at the farms.

4 Conclusions

The study was conducted to investigate the performance and water distribution from ten drip irrigation systems used by local farmers irrigating crops. It was found that the ten systems were varied in their uniformities of the applied water. Also, results of field evaluation show that the irrigation performances are mostly lower than the accepted values for most evaluated systems. The causes of non-uniformity and low efficiency could be related to some factors such as, pressure variation in the system, in correct system design and emitter discharge variation. The generated results are expected to be helpful to the farmers and irrigators to make the correct maintenance to increase the irrigation system efficiency.

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References

[1] Food and Agriculture Organization (FAO). Water precious and finite resource. Website report. www.fao.org/ag/magazine/0210sp1.htm, 2003

[2] Al-jamal MS, Ball S. and Sammis TW. Comparison of sprinkler, drip and furrow irrigation efficiencies for onion production. *Agricultural Water Management **46:*** 253-266, 2001

[3] Mantovani, E. C.; Villobos, F. J.; Orgaz, F. and Fereres, E. Modeling the effects of sprinkler irrigation uniformity on crop yield. Agricultural water management, 27:243-257, 1995.

[4] Bralts, V.F. Field Performance and Evaluation. In: *Drip Irrigation for Crop Production.* Chapter 3, by Nakayama, F.S. and Bucks, D.A. Amsterdam, the Netherlands: Elsevier Science Pub., 1986

[5] Bucks, D.A, Nakayama, ES. and Warrick. A.W. 'Principles. Practices and Potentialities of Drip (Drip) Irrigation." In: Hillel, D. (Ed). *Advances in irrigation* 1, New York: Academic Press, 1982.

[6] Bucks, D.A. and Davis, S., "Historical Development", In: *Drip Irrigation for Crop Production".* Chapter 1 by Nakayama, ES. and Bucks, D.A. Amsterdam, the Netherlands: Elsevier Scientific Publishers, 1986.

[7] Christiansen, J. E. Irrigation by Sprinkling. California Agricultural Experiment Station, Bulletin 670, 1942.

[8] Decroix. M. and Malaval, A. "Laboratory Evaluation of Drip Irrigation Equipment for Field System Design." *Proc. 3rd Inter. Drip/Drip Irrigation Congress.* Fresno. California: 325-330, 1985

[9] Merriam, J.L. and Keller, J. Farm irrigation system evaluation: A guide for management. Utah State University. Logan, Utah. U.S.A., pp 271. 1978

[10] Karmeli. D. 'C1assification and Flow Regime Analysis of Drippers. *J. Agric. Eng. Res.* 22, 1977.

[11] Zhu, H., Sorensen, R. B., Butts, C. L., Lamb, M. C. and Blankenship. 2002. A Pressure regulating system for variable irrigation flow controls. Applied Engineering in Agriculture. 18 (5): 533-640, 2002

[12] American Society of Agricultural Engineers. *ASAE Standard No. 405.* ASAE, 2950 Niles Rd. St. Joseph. MI, USA, 1997.

[13] Karmeli, D; Pen, O and Todds, M. *Irrigation Systems: Design and Operation.* Cape Town: Oxford University Press, 1985.

[14] Sporre-Money, J. L., Lanyon, L. E. and Sharpley, A. N. Low-Intensity sprinkler for evaluating phosphorus transport from different landscape positions. Applied Engineering in Agriculture. 20 (5): 599-605, 2004

[15] Solomon. K.H. Manufacturing Variation of Emitters in Drip Irrigation Systems.' *Transactions, ASAE,* 22. No. 5, 1979.

[16] Wu, l.P., Howell, T.A. and Hiler, E.A. "Hydraulic Design of Drip Irrigation Systems". *Hawaii Agric. Exp. Sta. Tech. Bull.* 105. Honolulu, Hawaii: pp. 80, 1979.

Effect of subsurface amendments and drip irrigation on tomato growth

A. M. Al-Omran[1], A. S. Sheta[1], A. M. Falatah[1] & A. R. Al-Harbi[2]
[1]Department of Soil Science, College of Food and Agricultural Sciences, King Saud University, Saudi Arabia
[2]Department of Plant Production, College of Food and Agricultural Sciences, King Saud University, Saudia Arabia

Abstract

The management that increases yield and reduce excessive amount of water is a priority for agriculture development in arid and semi-arid regions. This research investigated the effectiveness of previously subsurface placement (25 cm) of the clay deposits and drip irrigation on tomato yield, water use efficiency (WUE), soil moisture and salt distribution in the root zone. A calcareous sandy soil had a subsurface amendments and surface and subsurface drip irrigation applied for one year and planted with squash crop prior to seedling tomato. The field experimental site, and randomization, and consequently location of each treatment, replication was the same for the tomato experiment. The clay deposits were collected from western (Khulays) and central (Dhruma and Rawdat) regions of Saudi Arabia. Surface and subsurface drip irrigation were used at rates ranging from 234 mm (T1) to 564.5 mm (T4). The results clearly reveal that nutrients levels in all the experimental plots were quite variable depending on the amendments type, and rate of application, and the irrigation systems. Results show that tomato fruit yield and WUE were significantly affected by amendments rates and type and also by irrigation amounts and system. The soil moisture contents of subsurface drip irrigated layer increased dramatically, while salts accumulated at the surface away from the emitters of subsurface drip irrigation.
Keywords: drip irrigation, clay deposit, tomato yield, sandy calcareous soils, water use efficiency.

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1 Introduction

The sustainable use of scarce water resources in Saudi Arabia is a priority for agricultural development. Therefore, practices that increase water use efficiency and reduce excessive amount of water applied to the field are important in water management. Also, adoption of modern irrigation techniques is needed to be emphasized to increase water use efficiency. Drip irrigation is the most effective way to apply directly water and nutrients to plants and not only save water but also increases yields of vegetable crops (Tiwari et al. [1, 2]). Tiwari et al. [3] and Bryla et al. [4] reported that drip irrigation improved production and water use efficiency of faba bean in California using different levels of irrigation based on percentage of evapotranspiration. Ayars et al. [5] reported from their studies on subsurface drip irrigation and furrow irrigation in the presence of shallow saline ground water that yield of drip irrigated tomato were greater under drip irrigation than under furrow irrigation. Lamm and Trooien [6] reported that a successful application of subsurface drip irrigation for 10 year in Kansas, USA reduced the irrigation water use for corn by 35-55% compared with traditional forms of irrigation. Phene et al. [7] reported that subsurface drip irrigation improved water use efficiency (WUE) of tomato plants. Phene et al. [8] studied the distribution of roots under sweet corn as a function of drip placement and fertilization treatment. They reported differences between surface and subsurface drip irrigation on sweet corn rooting system in the top 45 cm. High root length density was observed below 30 cm in the subsurface drip irrigation than in the surface drip.

2 Materials and methods

A field experiment has been established in 2002 to 2003. Squash plants (*Cucurbita pepo*) were grown in field plots on the College of Agricultural Research Station at Dirab (24°25 N, 46°34 E), 40 km southwest of Riyadh, Saudi Arabia.. In brief, the soil was non-saline, non-sodic, calcareous and sandy texture. The irrigation systems layout is surface and subsurface drip irrigation while water quality was highly saline and moderately sodic. The natural clay deposits used in the experiment were collected from western region (Khulays) and central region (Dhurma and Rawdat areas) of Saudi Arabia. The experiment included also surface (S) and sub-surface (SS) drip irrigation methods.

Tomato (*Lycopersicon esculentum L.* cv. Bascal) seedlings were transplanted on 15 November 2003. Four irrigation levels 234 mm (T1), 330 mm (T2), 388.5mm (T3), and 564.5 mm (T4) were applied over the entire season. The experiment has been laid out following the complete randomized block design with three replicates for each treatment. Each treatment consists of 7 drippers (2.8 m tubing) and the distance between two rows was about 1 m. Three seedlings were transplanted at each dripper. Irrigation was commenced after transplantation and continued every other day until the end of experiment. Fertigation was used to deliver N-P-K soluble fertilizers to the plant root zone.

Fruits were picked five times till the end of the season (April 17th 2004) weighted, and the total yield was determined.

Twenty soil samples were collected before irrigation from the root zone area on a grid bases (15 cm apart) around the dripper at the three growth stages. Water contents were determined by gravimetric method and alt distributions were assessed by measuring EC in 1:1, soil to water extract.

3 Results and discussions

Data in Table 1 showed that differences due to water regime, surface and subsurface drip irrigation and the interactions between water regime and irrigation methods were highly significant (at 1% level) for both tomato fruit yield and WUE. Differences in WUE and tomato fruit yields due to amendment rates and the interactions between amendment rates and water regime or irrigation methods were also significant (at 1% or 5% levels) whereas the interaction between amendment types and rates was not significant. Data also showed that differences due to amendment types and the interaction between water regime and amendments or the irrigation methods and amendments were not significant. These results reflect the positive effect of water regimes, surface and subsurface drip irrigation and amendment rates on tomato fruit yield and WUE.

The results are further elaborated in order to evaluate the effect of each treatment on tomato fruit yield and WUE. Effect of amendments types, irrigation regimes, irrigation methods and the amendment rates on tomato fruit yield and WUE are presented in Table 1 and graphically illustrated in Figures 1-4. It indicated that at high irrigation levels (non-stressed T4 and T3 treatments), fruit yield were high and decreased significantly at low irrigation levels (stressed, T2 and T1 treatments). The average yield increased by about 20.3% in the T4 treatment when compared with T3 treatment, whereas average yield decreased in the T2 and T1 treatments by about 72.0 and 123.0%, respectively. However, WUE was highest at T3 and T1 treatments. The increase in the amount of irrigation water significantly affected the yield and WUE. The yield was the highest (73.67 ton ha^{-1}) at T4 (564.5 mm) and reduced to 32.89 ton ha^{-1} in T1 (234 mm), a stress treatment.

The water production function (water applied vs. yield) (figures 1-2) showed that the subsurface drip irrigation has a better correlation and r^2 ranged from (0.98-.99), while the value of r^2 for surface irrigation ranged between (0.17-0.78). This result is with full agreement with the result reported by Lamm and Trooien. The decrease in yield at low water application could be attributed to the unavailability of water and the possible accumulation of salts in root zone area as a result of using a high saline water (TDS = 3300 ppm), where no proper leaching took place. An increase in the irrigation amount did not show a definite trend in WUE. Results in Table 2 indicate that amendments type significantly affected the yield and WUE compared to the control. Dhurma clay deposit resulted in producing the highest average fruit yield and as well as WUE followed by Khulays and Rawdat. The yield increase was 12.92%, 2.48% and

Table 1: Effect of clay deposits (type and rates), irrigation regimes and irrigation methods on Tomato yield (ton ha^{-1}) and WUE (kg m^{-3}).

Treatments	Yield (ton ha^{-1})	WUE (kg m^{-3})
	Effect of clay deposits type	
Dhruma	56.34 A	14.64 A
Khulays	51.13 B	13.72 AB
Rawdat	50.49 B	13.44 B
$LSD_{0.05}$	3.52	1.12
	Effect of irrigation water regimes	
T1	32.89 D	14.04 A
T2	42.81 C	12.96 B
T3	61.24 B	15.76 A
T4	73.67 A	13.04 B
$LSD_{0.05}$	4.07	1.28
	Effect of irrigation methods	
Surface drip	50.76 B	13.84
Subsurface drip	54.55 A	14.04
$LSD_{0.05}$	2.88	n.s.
	Effect of amendment rates	
Control	49.89 B	13.28 B
1%	54.34 A	14.52 A
2%	53.71 A	14.08 AB
$LSD_{0.05}$	3.53	1.12

* The same letter in each column represents no significant difference at 5% level.

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1.2% for Dhruma, Khulays and Rawdat, respectively when compared with the control. The differences could be due to the clay deposit characteristics and the variation in $CaCO_3$ content, ECe, CEC and the dominant clay minerals. Khulays deposit showed some desired characteristics such as low $CaCO_3$, high CEC and the dominance of smectite clays, whereas it has relatively high original salinity which could be leached out of the root zone area before cultivation. Differences in tomato fruit yield due to irrigation methods were significant and the yield increase due to subsurface drip irrigation was about 7.47% over the surface drip irrigation. WUE did not show any significant difference between the two irrigation methods. It seems that subsurface drip irrigation creates more suitable conditions in the root zone area for the plant growth, which is in agreement with the result reported by Lamm and Trooien [5].

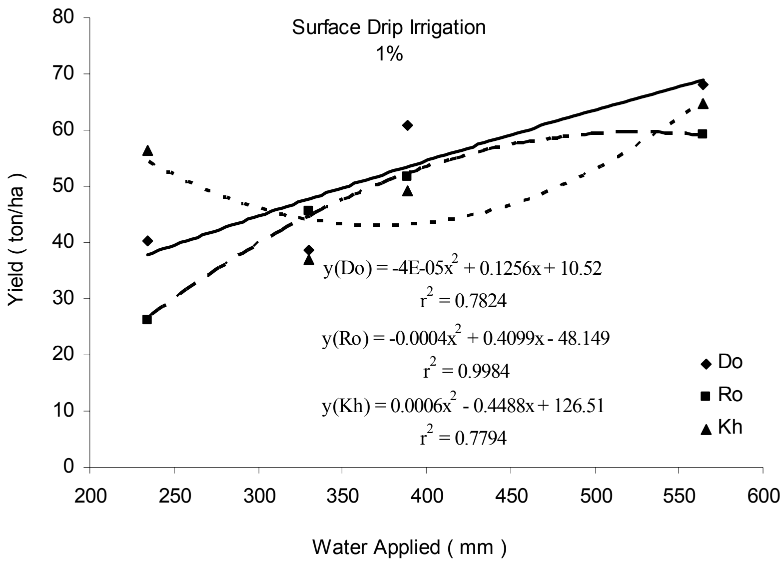

Figure 1: Relationship between tomato yield and water applied at surface drip irrigation under 1% rate of different natural deposits.

The amendment rates significantly affected tomato fruit yields under the experimental conditions. The average fruit yield was increased by 8.9% and 7.6% at 1 and 2% amendment rates when compared with control. Such increase in yield could be due to the improvement of sandy soil characteristics particularly the available water content and nutrient status. Also, differences in WUE due to amendment rates were significant. The application of clay deposits could have positive effects on soil texture, structure, swelling and increasing CEC and soil water retention, hence resulting in improved soil water contents in the tomato root zone.

Data of water and salt distributions in the root zone area for all treatments were graphically illustrated using Matlab software and data of selected treatments were presented in Figures 3 and 4. It indicated that water distributions show specific distribution patterns in the amended soil in both surface and subsurface drip irrigation. Such distribution pattern depends on the type and rate of amendment in the subsurface treatment. Water content was generally low (about 3-4%) on the surface and increased gradually with depth without clear distribution trend (5-7%). There was no clear difference between surface and subsurface drip irrigation in non amended soil where soil profile was not modified. Again, this trend could be due to water evaporation from the surface and hence decrease water content in the surface layer. The treated soils showed relatively high water content below 30 cm depth indicating deep percolation and partial losses of water below root zone. In amended soil water content was quite high at either surface or the subsurface drip irrigation treatment (Fig. 3) particularly in the amended subsurface layer (soil water content = 10-12% in the soil treated with Khulays clays). It was clear that water seems to be stored in the treated layer with no or little percolation below 30 cm depth. The surface layer of the subsurface drip treatment was relatively dry and it seems to be uniform in dryness compared with the surface irrigation where dryness seems to be on the sides. Therefore, applications of clay deposits to sandy soils modifies the distribution of water content in the root zone area where water could be retained by clays applied to the subsurface layer. The desired characteristics of clay deposits could be reflected on the improvement of soil texture, structure, swelling, increasing CEC and soil water retention, hence resulted improved soil water contents in the tomato root zone. These data and conclusions agreed well with the results obtained for the previous crops as mentioned by (Al-Omran et al. [9]).

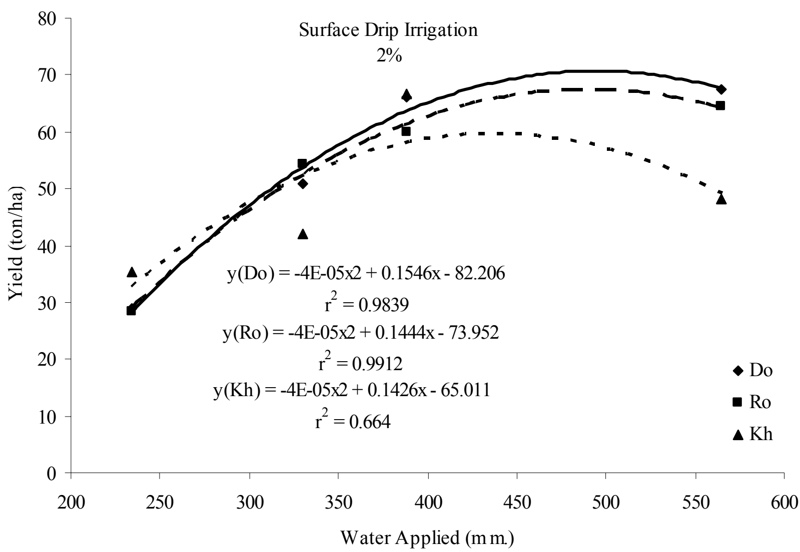

Figure 2: Relationship between tomato yield and water applied at surface drip irrigation under 2% rate of different natural deposits.

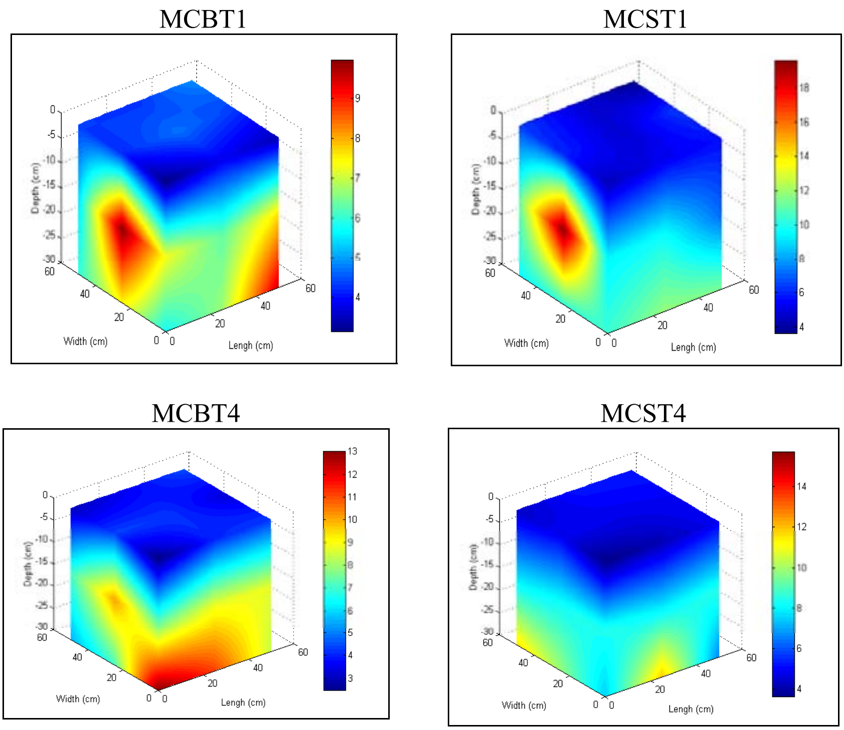

Figure 3: Water content distributions in the root zone area in surface and subsurface drip irrigation as affected by clay deposits. (MC = moisture content, B = subsurface drip irr., S = surface drip irr., T1 = level 1 of water applied, T4 = level 4 of water applied).

Soluble salt distributions (EC, dS/m) in the root zone area (Fig. 4) showed an adverse trend when compared with water distributions; it was high on the surface and decreased gradually with depth to the lowest values (at 15-30 cm depth). Amended soil with clay deposits (Rawdat deposit) indicated clear different trend particularly in the amended layer (about 20 cm depth). Salt concentration was relatively low in the amended layer while it accumulated on the surface in the subsurface drip irrigated soil and around the emitter in the surface drip irrigated soil. Salt accumulation appears to be reversibly related to water distribution in either surface or the subsurface drip. Again it appears that the subsurface amended layer have the lowest salt concentrations without clear differences when compared with the control. Therefore, increasing water content in the clay amended layer under subsurface drip seems to alleviate the harmful effect of salts and create more suitable conditions for root growth.

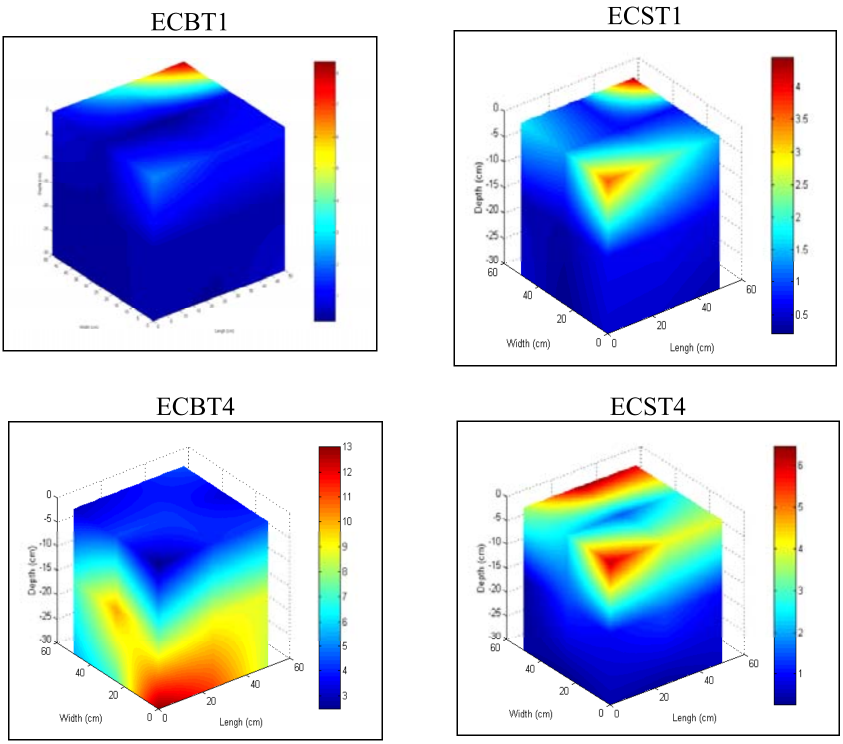

Figure 4: Salt distributions (dS/m) in the root zone area in surface and subsurface drip irrigation as affected by clay deposits. (EC = Electrical Conductivity, B = subsurface drip irr., S = surface drip irr., T1 = level 1 of water applied, T4 = level 4 of water applied).

4 Conclusions

Types of clay deposits significantly affected tomato yields and WUE compared with the control. The yield increase was 12.92%, 2.48% and 1.2% for Dhruma, Khulays and Rawdat, respectively when compared with the control. Clay amendment application as a subsurface layer to sandy calcareous soils increased water content, decrease soil salinity and improve the distribution of tomato roots in the treated layer.

References

[1] Tiwari, K. N., Mal, P. K., Singh, R. M. and Chattopadhyay, A. Response of Okra (Abelmoschus esculentus"L." Moench) to drip irrigation under mulch and non-mulch condition. Agric. Water Manage. 38: 91-102, 1998a.

[2] Tiwari, K. N., Mal, P. K., Singh, R. M. and Chattopadhyay, A. Feasibility of drip irrigation under different soil covers in tomato. J. Agric. Eng., 35 (2): 41-49, 1998b.
[3] Tiwari, K. N., Singh, A. and Mal, P. K. Effect of drip irrigation on yield of cabbage (Brassica oleracea L. var. capitata) under mulch and non-mulch conditions. Agric. Water Manage., 58: 19-28, 2003.
[4] Bryla, D. R., Banuelos, G. S. and Mitchell, J. P. Water requirements of subsurface drip-irrigated faba bean in California. Irrig. Sci: 22 (1): 31-37, 2003.
[5] Ayars, J. Lschoneman, R. A., Dale, F., Meso, B. and Shouse, P. Managing subsurface drip irrigation in the presence of shallow ground water. Agric. Water Manage. 47: 243-264, 2001.
[6] Lamm, F.R. and Trooien, T.P. Subsurface drip irrigation for corn productivity: a review of 10 years of research in Kansas. Irrig. Sci: 22: (3–4), 195–200, 2003.
[7] Phene, C. J., Davis, K. R., Hutmacher, R. B. and McCormick, R. L. Advantages of subsurface drip irrigation for processing tomatoes. ActaHortic. 200: 101-113, 1987.
[8] Phene, C. J., Davis, K. R., Hutmacher, R. B., Bar-Yosef, B., Meek, D. W. and Misaki, J. Effect of high frequency surface and sub-surface drip irrigation on root distribution of sweet corn. Irrig. Sci., 12: 135-140, 1991.
[9] Al-Omran, A.M., Falatah, A.M., Sheta, A.S. and Al-Harbi, A.R. Effect of drip irrigation on squash (Cucurbita pepo) yield and water-use efficiency in sandy calcareous soils amended with clay deposits. Agric. Water Manage: 73: 43-55, 2005.

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An automation system based on labview to control the test of mechanical flow meters

J. Martínez[1], R. Alvarez[1], P. Cortez[1], V. Mejia[1] & V. H. Zarate[2]
[1]*Mexican Institute of Water Technology (IMTA), Mexico*
[2]*Technological Institute of Superior Studies of Monterrey (ITESM), Mexico*

Abstract

We present a computational system, based on a PC and LabVIEW to test and characterize the response of mechanical flow meters of conical cup type with vertical axis. A mechanical flow meter is a device used mainly to measure and calculate velocity of water flow on rivers and open channels.

These devices, suffer mechanical imperfections, over time, which is why it is important to calibrate them, normally twice a year, depending on its time of use. At the Mexican Institute of Water Technology (IMTA in Spanish) was designed and developed a circular water tank for the purpose of testing these meters.

This paper shows the automation systems designed to control the tests to calibrate these mechanical meters. The system is based on LabVIEW. LabVIEW is a general purpose programming tool with extensive libraries for data acquisition instrument control, data analysis, and data presentation. With this tool and a special hardware interface, it was possible to automate the process to test these mechanical meters.

The system is called SCM (system for characterization of mechanical meters). SCM controls the test of two mechanical meters simultaneously, and has some user control features that permit the Operator an easy to use human machine interface.
Keywords: mechanical flow meters, testing laboratory, LabVIEW based automation system.

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1 Introduction

In 1996, the Mexican Institute of Water Technology built a water channel laboratory to test and calibrate the mechanical meters that are used all around Mexico [1]. These mechanical meters are very robust and priceless, that is the reason why they are very common in many countries. Figure 1 shows a typical mechanical flow meter.

The flow mechanical meters of conical cup type with vertical axis are devices that convert the velocity of flow water into revolutions by minute. This effect is produced because the force produced by the flow water into the conical cup converts velocity into counts of rotations. At each rotation a mechanical contact, inside of a contact chamber, close momentarily. This relationship between velocity and revolution per minute (RPM) is a characteristic obtained by experiments [2]. The purpose of the water tank laboratory is to obtain the characteristics of each mechanical meter. To test, two mechanical meters are placed at the end of the rotary metallic arm. This arm moves around the channel producing the same effect of water stream flow.

The circular water channel laboratory has 12 m of diameter, 80 cm of cross section and 1.40 m depth. The channel is associated to a mechanical arm moved by a motor of 15 HP. At the end of the arms, there exists two vertical axes to carry up one mechanical meter on each [1]. A picture of this channel is shown in figure 2. This circular channel is the first of its type, as show the different water channels, normally linear and 100 or 200 m long [3]. This circular channel represents a real challenge in this kind of laboratory.

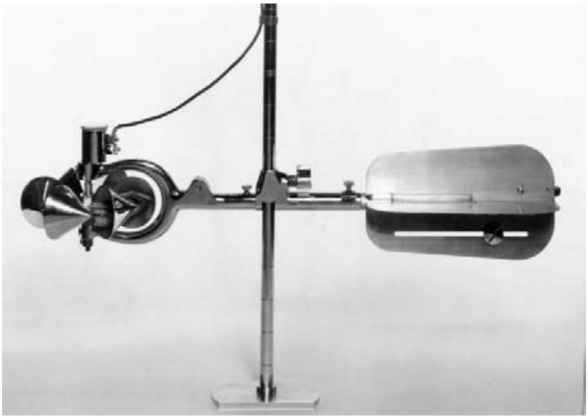

Figure 1: A typical mechanical meter of conical cup type.

Figure 2: A picture of the circular channel.

2 Methodology

The objective of the system was to create an automation system easy to use, and flexible, in a way that all the different versions of mechanical meters could be characterized. At the same time, the resulting system must be capable of performing as good a calibration as would be made by a linear channel. We used a top-down methodology to arrive at design and development of the SCM system:

i) First of all, we analyze the different requirements,
ii) We decompose our problem in two subsystems: hardware and software
iii) We select the architecture for each subsystem
iv) We design and development each subsystem
v) We integrate and test the two subsystems
vi) We had a stage of testing by the end user: the Operator

The next sections show the design of the two main subsystems: software & hardware.

3 The system for characterization of mechanical meters (SCM)

This systems mission is to control all the processes to characterize the mechanical meters. In figure 3 we show the architecture of the SCM.

3.1 The computational subsystem (software)

The functions of the subsystem are:

i) To control the characterize process of two simultaneous mechanical meters with the help of a man-machine interface.
ii) To manage the data base with the history of the mechanical meters tested
iii) To control the different velocities of testing based on the control of a motor drive, from 0.14 to 3 m/s with 12 different velocities.
iv) To calculate the characteristic equation velocity vs. RPM for each mechanical meter under test.
v) To generate the graphs, the reports and the tables for every mechanical meter tested.
vi) To eliminate the electrical spurious bounces.

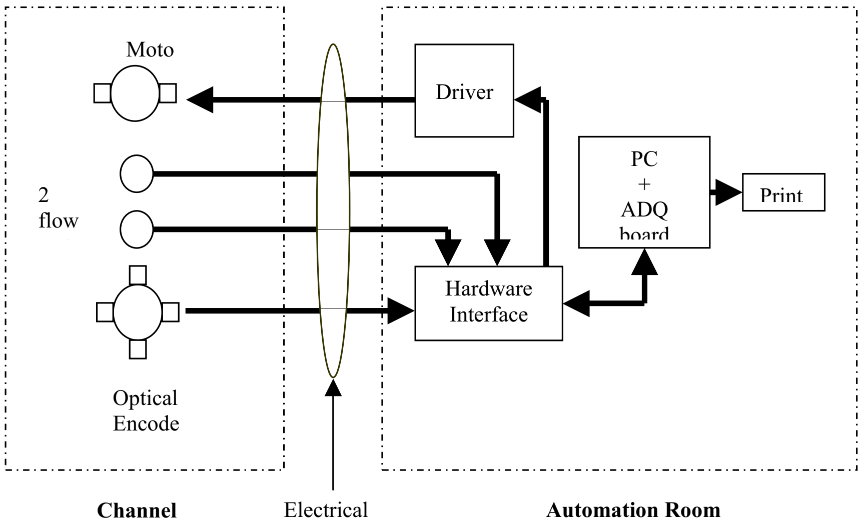

Figure 3: Architecture of the SCM.

3.1.1 Control the characterizing process

LabVIEW is a very easy to use language for creating man machine interfaces in automation systems. The system consists of different dialog menus to help the Human Operator control the test of two different mechanical meters. The principal program follows the next sequence:

Initialize all different parameters
REPEAT
Select option
IF option is characterize the meters:
Ask for the different parameters: time and velocity of test

```
            Manage the test at different velocities
            Make the final graphs, update the Date Base
      ENDIF
        ELSE
         IF option is consulting Date Base:
            Select Date Base.
            Show it
      ENDIF
          ELSE
         IF option Configuration of the system:
            Show parameters
                Select parameters.
            Change them
      ENDIF
  UNTIL Operator exit the system
```

The full system based LabVIEW has approximately 64 different routines.

3.1.2 Manage the data base

The Date Base is an array of registers, one for each mechanical meter. The SCM can make updates to each register, based on the parameters read during the test. Also, it is possible for the Operator to review and update some of the fields on the register, for example:

i) Details of the maintenance made,
ii) Date of entrance to the laboratory
iii) Owner
iv) Special features

3.1.3 Control the different velocities

This characteristic permits the selection of 12 different velocities in a wide range, from 0.14 to 3 m/s. Also, performs global or individual tests. The global test means that the system automatically begin at the lower velocity and step by step reach the highest velocity. In each step, the system makes the arrangement to control the velocity of test, and also, it reads the train of pulses of the two mechanical meters.

It is possible also to perform individual tests. In this mode, the Operator selects one special velocity and the system executes all the process automatically, including the steps to move the arm to reach the velocity. It is important to say that the final velocity is reached following the dynamic conditions of the motor and its load.

3.1.4 Calculate the characteristic equation velocity vs. RPM

When the process of testing is finished, the arm's movement stops, then the system calculates, based on the data read, the equation of the line.

Two parameters are calculated, the slop “m” and the intersection “b”. We use the function that LabVIEW has for this purpose, and the system graphs this result.

3.1.5 Generate the graphs, the reports and the tables

Finally, the SCM system generates a table, based on the equation of the line calculated. This table is practically the tabular representation of the graph. It is important because this table is the tool people will use when they use this mechanical meter in the field. In figure 4 we show one typical graph.

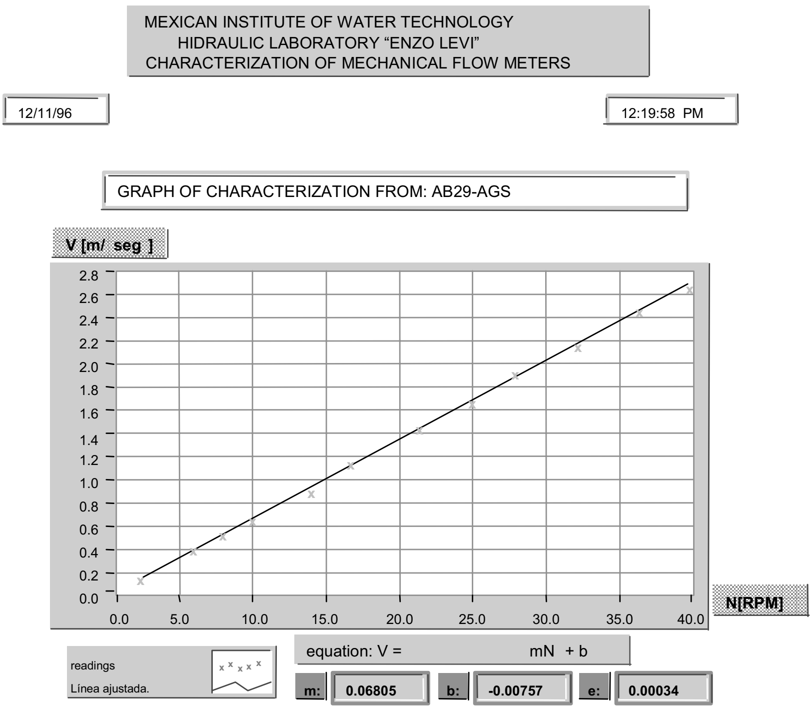

Figure 4: Typical graph based on the incoming data.

At the end of this process, the system prints out this information.

3.1.6 Elimination of the electrical bounces

The process to convert the revolutions per minute produced by the stream flow into the electrical train of pulses consists of injecting an electrical current through the mechanical movement of the meter to transform the revolution by minute into a train of electrical pulses. In this process, electrical bounces appear because of the mechanical nature of the contacts. A typical pulse is shown in figure 5. The problem arrives because the range of velocities that the mechanical meters measure is wide, from 0.14 to 3 m/s. With this range of velocity, the frequency of the electrical pulse vary from 0.2 Hz to 5 Hz.

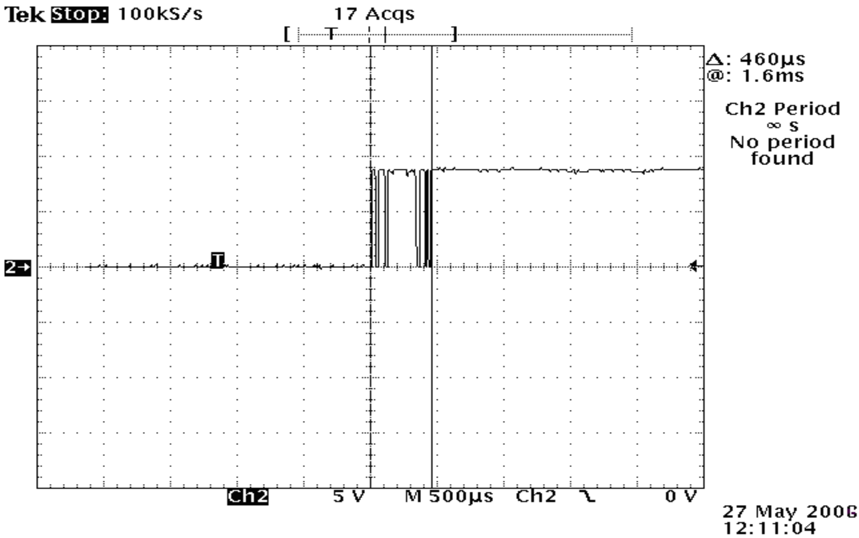

Figure 5: Noisy electrical signal produced by the mechanical flow meter when the contact close.

As seen in figure 5, the train of pulses is associated with a train of electrical pulses from the mechanical nature of the meter. These pulses have a random behavior, however based on experiments; we found that it has some characteristics that we can use to eliminate this electrical noise:

i) The maximal frequency of rotation is 300 RPM, at a period of 200 ms.
ii) The minimal frequency is 12 RPM, at a period of 5 s.
iii) The duty cycle between the time the contacts are closed and opened is less than 30%.
iv) The maximum time the electrical pulses appear at the maximal frequency is less than 10 ms.
v) The maximal time the electrical pulses appear at the minimal frequency is less than 300 ms.

The analysis of this data shows that, for low frequency, the electrical spurious pulses are equivalent to a train of pulses generated by the mechanical meter at high frequency. If we decided to eliminate that noise with a low-pass filter, this filter could eliminate also valid pulses. This problem is amplified because every mechanical meter has its own response. To solve this problem, we need an adaptable filter, based on an estimated frequency of rotation, which is able to eliminate the spurious noise. We can then calculate the right rotary frequency of the mechanical meter. To implement this dynamic filter, we use two different approaches depending on the system, as we will see in the next section.

To build the dynamic filter, we use the knowledge related to the behavior of this quasi-random noise. Furthermore, we know the velocity of testing because we control the rotary electrical motor. Our algorithm takes into account these two factors and adapts the parameter of the filter to eliminate these electrical spurious pulses. The algorithm is, for a specific velocity of test, shown in figure 6, on the next page.

Control the electrical motor to put the testing velocity
Put the filter parameters based on the velocity
REPEAT
Monitoring the pulse arrival
Monitoring the actual velocity with the optical encoder and modify if necessary
IF pulse arrives:
Check if the period of arrives was larger than the filter parameter.
IF yes, pulse is ok, count it
ELSE ignore pulse
ENDIF
UNTIL time of test ends **or** Operator stops the test

Figure 6: Algorithm to denounce electrical noisy signals.

3.2 The electronic subsystem (hardware)

The hardware consists of a PC Pentium II 400 MHz computer associated to a dispositive date acquisition (DAQ) card. We use the PCI-6071-E of National Instruments, and it is charged to communicate with our homemade interface board. Finally, we have a drive to control the motor. We use the ACS 501, adjustable frequency (ABB mark) driver

We designed an electronic board to interface the different devices to the dispositive date acquisition (DAQ) card. The elements and function of the interface are:

i) Injection of electrical current at each mechanical meter in test.
ii) Reading of two asynchronous train of electrical pulses, each one may have different behavior
iii) Generation of different electrical control signals to the motor drive
iv) Reading the different electrical status signals from the motor drive
v) Reading the train of pulses generated by the optical encoder to measure the actual velocity of movement

4 Results and conclusions

This way of making the characterizations by means of the annular channel is not very common since by regulating the characterizations were made in straight channels. The construction of the system and its automatization are an innovation of the technology to characterize flow meters in short times and with greater precision and trustworthiness. In order to verify the effectiveness of the developed system I am made a comparative experimental study of the characterization of a lot of flow meters, as much in the annular channel as in the certificate channel of the laboratory of hydraulics of Canada Centers for Inland Waters (CCIW) the obtained characteristic equations in both channels for each flow meters are very similar, which is translated in acceptable uncertainties of the annular channel like characterization system with which we can say that the results that are obtained are correct. The characteristics of the system are:

i) Easy of use
ii) 0ne hour per test,
iii) Easy of manage the Data Base y Make statistical analysis

Right now we are preparing the laboratory of test to achieve an International Certification.

References

[1] Petronílo Cortés, Ricardo Alvarez. "Circular water channel for testing mechanical flow meters" In Spanish. XIV Congreso Nacional de Hidráulica. Asociación Mexicana de Hidráulica. Acapulco Gro..pp. 1-5, October 1996.
[2] U.S. Department of the Interior. "Water Measurement Manual "third edition 1997 (these web pages are based on the third edition) http://www.usbr.gov/wrrl/fmt/wmm/ [consulted November 1999].
[3] The Canada Centre for Inland Waters. World Wide Web servers at Burlington, Ontario. http://www.cciw.ca/ [consulted November 1999].

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Section 10
Urban water management

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Entropy-based reliable design of water distribution networks

J. J. O. Sousa[1], M. C. Cunha[2] & J. A. Sá Marques[2]
[1]*Polytechnic Institute of Coimbra, Portugal*
[2]*University of Coimbra, Portugal*

Abstract

Water distribution networks are crucial for the wellbeing of communities, but their construction involves huge investment. The need to optimize investments has resulted in the development of methodologies to determine the minimum cost design of these infrastructures. However, as cost minimization tends to eliminate any redundancy in the networks, this kind of approach leads to less reliable solutions. From the reliability point of view, an ideal water distribution network should ensure reasonable service levels even when a failure occurs. But such a scenario, if possible, would certainly imply an intolerable level of investment. The following question therefore arises: by how much is it reasonable to increase investment to reduce the risk of failure? Trying to help decision makers answer this question, this work offers a tool to solve the reliable design of water distribution networks problem. It is based on an optimization model comprising two antagonistic objectives: minimizing the cost and maximizing reliability (here indirectly evaluated by the network entropy). The multiobjective problem is solved by the constraint method. The result is a set of cost minimization problems, each constrained by a different minimum level of entropy, which are solved by the Simulated Annealing method. This approach leads to a set of solutions known as Pareto solutions. Each of these solutions represents a different level of compromise between cost and reliability, and they can be very helpful for decision makers.
Keywords: water distribution networks, optimization, simulated annealing, reliability, entropy.

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1 Introduction

> *"Nothing in this world is 100% reliable, and, as water engineers are painfully aware, urban water supply systems are no exception. ... In planning, engineers try to minimize the probability of failure, but 100% reliability is impossible. ... the heart of the design problem is to decide where along the reliability-cost frontier the system should be."*

These statements, from Hobbs and Beim [2], may have been written almost two decades ago, but they especially illustrate a problem that remains very much one of today: the analysis of water supply systems' reliability. The reliability of these systems may be affected by several factors, namely: changes in water availability at source; changes in water quality at source; errors in consumption predictions used in the design and in the operation; failures in system components (treatment stations, reservoirs, pipes, pumps, valves); operational errors in defining necessary storage capacity to face peak consumption.

In general, reliability is defined as the probability that a system will perform well for a certain time when subject to specified conditions. In the sphere of water distribution network operations, reliability is related to the capacity to meet water consumption needs with pressure conditions that satisfy minimum requirements. The concept of fault is famously associated with reliability. In the present context, fault means the occurrence of a situation that impairs the network's ability to provide enough discharge, or sufficient pressure, to meet water demands, and from the evaluation point of view, water distribution networks reliability is affected by different kinds of fault: errors in water demand prediction, mechanical faults and faults following accidents.

The most usual way to assign reliability is by introducing some redundancy into the network. In the traditional design of water distribution networks, reliability is introduced by the application of some empirical rules, viz.: overestimation of pipe diameters, introduction of additional pipes to form loops, equipping pumping stations with emergency pumps, etc. In the context of optimal design, the reliability problem has been tackled in several ways in the last twenty years, and has given rise to a wide range of methodologies whose goal is to obtain optimal solutions (from the economics point of view) that are simultaneously reliable. The reliability of a network can be evaluated by analytical methods (based on probabilistic measures derived from graph theory) or by simulation methods (using simulators to estimate the network behaviour under fault scenarios and evaluating reliability from the results of the simulations performed). The diversity and complexity of situations they can deal with means that simulation methods are more likely to produce realistic analyses. Notwithstanding the efforts made in the last two decades, today there is still no universally accepted criterion to evaluate water distribution network reliability, and this situation constitutes a serious impediment to the progress of research in this sector. Most of the criteria proposed in the literature rely on the following: measures related to network topology; measures involving simultaneously the network topology and pipe capacities; deterministic and/or probabilistic measures related to the causes of network faults; measures based on the quantification of consumptions that cannot be totally supplied; resilience measures.

Concerning all these criteria, there are three major aspects that must be pointed out: some criteria are easy to implement but not rigorous enough to evaluate reliability; others are too demanding in terms of data needed for implementation; lastly, it is important to mention that, in the case of the most rigorous criteria, the evaluation of network reliability is by itself an *NP-hard* or *NP-complete* problem, in other words, a complex problem to tackle.

Nowadays one of the most promising measures to evaluate water distribution network reliability seems to be the maximization of the network entropy, and so this is the criterion adopted in this work.

2 Reliable design of water distribution networks: entropy

The use of the entropy concept to evaluate water distribution network reliability is based on two main principles: it is desirable for each node to be supplied by more than one pipe; pipes incident on each node should have similar capacities. The theoretical basis of this criterion is the principle of Shanon's entropy maximization, proposed several decades ago and used widely in the most varied areas of knowledge. Tanyimboh and Templeman [4] presented a way to evaluate entropy in water distribution networks. This approach relies on a multiple probability space model and the conditional entropy formula of Khinchin. Later, Walters [5] simplified the original expression and obtained a simpler one:

$$\frac{E}{K} = -\sum_{ij \in M} \frac{Q_{ij}}{QT_0} \cdot \ln\left(\frac{Q_{ij}}{QT_0}\right) + \sum_{n=1}^{N} \frac{QS_n}{QT_0} \cdot \ln\left(\frac{QS_n}{QT_0}\right) \tag{1}$$

where: E : entropy;

K :arbitrary positive constant (in the present context K has no significant meaning such that it is usual to call network entropy by the relation E/K);

M :set of all positive flow links *ij*, including external inflows and outflows;

$$Q_{ij} \begin{cases} i = 0 \text{ e } j \neq 0 \text{ - external flow entering node } j; \\ i \neq 0 \text{ e } j = 0 \text{ - external flow leaving node } j; \\ i \neq 0 \text{ e } j \neq 0 \text{ - flow in the pipe connecting nodes } i \text{ and } j; \end{cases}$$

QT_0 : total supply or demand of the network;

N : set of junction nodes in the network;

QS_n : outflows at node *n*, including the node consumption.

This criterion to evaluate water distribution network reliability has two characteristics that make it very attractive, which are: it is not too demanding, and can be easily incorporated into optimization models. There are three different ways to incorporate this criterion into optimization models: first, by formulating an economical design model constrained by a minimum level of

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reliability, *E/K*; second, by formulating a reliability maximization model constrained by a budget, and finally by applying the Linear Programming method proposed by Alperovits and Shamir [1] to design the network subject to the flows that maximize its entropy. In most of the models to optimize the design of water distribution networks reliability is introduced as an additional set of constraints whose goal is to guide the design in order to obtain reliable solutions. The reliability constraints usually include some redundancy in the connections between reservoirs and consumption nodes to assure minimum service levels even in fault scenarios. This being so, and taking into account the enormous flexibility showed by modern heuristics in incorporating any kind of constraints, the methodology proposed by Cunha and Sousa [3], based on the Simulated Annealing method (see flowchart in Figure 1), was adapted to solve the reliable design of water distribution networks problem.

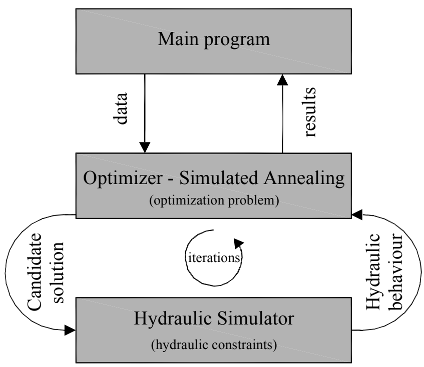

Figure 1: Flowchart explaining the methodology developed to solve the optimal design of water distribution networks.

The adaptation was restricted to the implementation of a subroutine that uses Eq. (1) to evaluate the entropy of each candidate solution. The reliable design of water distribution networks proceeds as follows: establish a minimum admissible level for the entropy, $(E/K)_{min}$; throughout the search, after the hydraulic simulation of each candidate solution, the new subroutine is summoned to evaluate the entropy; if the entropy value surpasses the minimum required the solution is feasible; otherwise, the solution can be immediately rejected or accepted with a penalty as a consequence of the minimum entropy constraint violation. Meanwhile, the rest of the optimization process remains unchanged. The optimization model used in this work is the following:

$$\text{Minimize } \sum_{j=1}^{NP} C_{pipe,j}\left(D_j\right)\cdot L_j \tag{2}$$

subject to:

$$\sum_{j=1}^{NP} I_{ij}\cdot Q_j = Qc_i \qquad i = 1, 2, \ldots, N \tag{3}$$

$$\Delta H_j = \frac{10.674\, L_j}{C_{HWj}^{1.852} \cdot D_j^{4.87}} \cdot Q_j^{1.852} \qquad j = 1, 2, \ldots, NP \tag{4}$$

$$H_i \geq H_{i,min} \qquad i = 1, 2, \ldots, N \tag{5}$$

$$D_j = \sum_{i=1}^{ND_j} YD_{j,i} \cdot DC_{j,i} \quad \text{with} \quad \sum_{i=1}^{ND_j} YD_{j,i} = 1 \qquad j = 1, 2, \ldots, NC \tag{6}$$

$$E/K \geq \left(E/K\right)_{min} \tag{7}$$

where:
NP: number of pipes in the network; N: number of junction nodes in the network; L_j: length of pipe j (m); D_j: diameter of pipe j (m); $DC_{j,1}$, $DC_{j,2, \ldots,}$ $DC_{j,ND}$: set of commercial diameters for pipe j (m); ND_j: number of commercial diameters in the set of pipes j; $YD_{j,i}$: binary variables that identify the optimal commercial diameter for pipes j; $C_{pipe,j}$ (Dj): unit cost of pipe j as a function of the diameter (€/m); I: network incidence matrix (NxNP); Q_j: discharge in pipes j (m^3/s); ΔH_j: headloss in pipes j (m); C_{HWj}: Hazen-Williams coefficient of pipes j; Qc_i consumption at junction node i (m^3/s); H_i: piezometric head at node i (m); $H_{i,mim}$: minimum piezometric head for junction node i (m); E/K: network entropy; $(E/K)_{min}$: minimum level of network entropy.

This optimization model is nonlinear and the decision variables are discrete (binary variables that identify the optimal commercial diameters for the pipes).

3 Application

This section sets out to illustrate the procedure proposed to undertake the reliable design of water distribution networks. The optimization problem that must be solved has two objectives: cost minimization and reliability maximization. The methodology used here to measure the reliability is based on the entropy concept, E/K, evaluated by Eq. (1). Therefore, the reliability maximization is represented by the entropy maximization. This two-objective optimization problem is solved by the constraint method, by which one of the objectives is replaced by a constraint imposing a minimum level for that objective. In the present case the original two-objective problem gave way to a set of cost minimization problems, each one constrained by a different minimum level of entropy: $E/K \geq (E/K)_{min}$. The application of this procedure is illustrated with the design of a new water distribution network (Figure 2) taken from Xu and Goulter [6].

This network is supplied by a single reservoir and has 33 pipes and 16 nodes. To achieve the design it was necessary to specify a set of commercial diameters whose unit costs were known. However, as in the methodology of Xu and Goulter [6] the diameters were continuous variables, the original version of this example only provided a cost function to evaluate the unit cost for each diameter value. In the absence of a set of commercial diameters, it was decided to use a ductile iron pipes set, with unit costs evaluated by the cost function from the original example (Table 1).

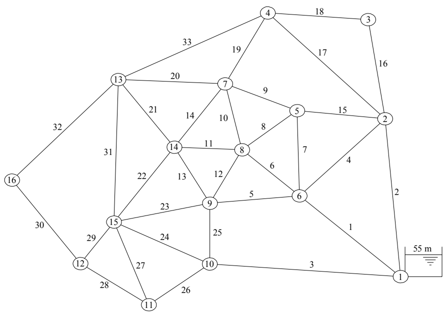

Figure 2: Scheme of the network.

Table 1: Available diameters and corresponding unit costs.

Diameter (mm)	Unit cost ($/m)	Diameter (mm)	Unit cost ($/m)	Diameter (mm)	Unit cost ($/m)
100	12.052	300	63.317	600	180.332
125	16.881	350	79.911	700	227.595
150	22.231	400	97.763	800	278.439
200	34.326	450	116.793	900	332.637
250	48.079	500	136.933	1000	390.000

After assembling all the necessary data, the reliable design process could be started. There was only one small dilemma: as the magnitude of the entropy values was not known a priori, what value should be specified for the minimum level to be imposed on the design? It is suggested that this difficulty may be overcome by using a preliminary design imposing $(E/K)_{min}$=0, that is, relaxing the minimum entropy constraint. Depending on how the network entropy is evaluated, the result of this procedure is equivalent to a minimum cost design, and so a branched network solution should be obtained (any branched solution would present the same entropy value, which is precisely the minimum value it can take). The result of this design was, in effect, a branched network, costing $2.781x10^6 and with entropy equal to 2.708 (this solution is presented in Figure 3 and in Table 2). This minimum entropy value was then used as a reference to establish the minimum level of entropy to impose for each design, $(E/K)_{min}$, and the design was repeated, but now with the entropy constraint active.

The goal of this work is to identify the solutions that optimize both objectives: economy (cost minimization) and reliability (entropy maximization). The solutions were obtained after executing several minimum cost designs, each one subject to a different minimum entropy level, starting from 2.708. The result of this procedure is a set of solutions (Pareto solutions) given in Figure 4, where a virtual line can be seen connecting them. This line separates the feasible region from the non-feasible region, that is: any feasible solution will be represented by a point over the virtual line (Pareto solution) or above that line; any point under the virtual line must represent a non-feasible solution (violates at least one of the problem's constraints). Table 2 presents some of the Pareto solutions obtained for this example, whose graphical representations can be seen in Figure 3.

To facilitate the reading of Figure 3, the lines' thicknesses are proportional to the diameters of the pipes they represent.

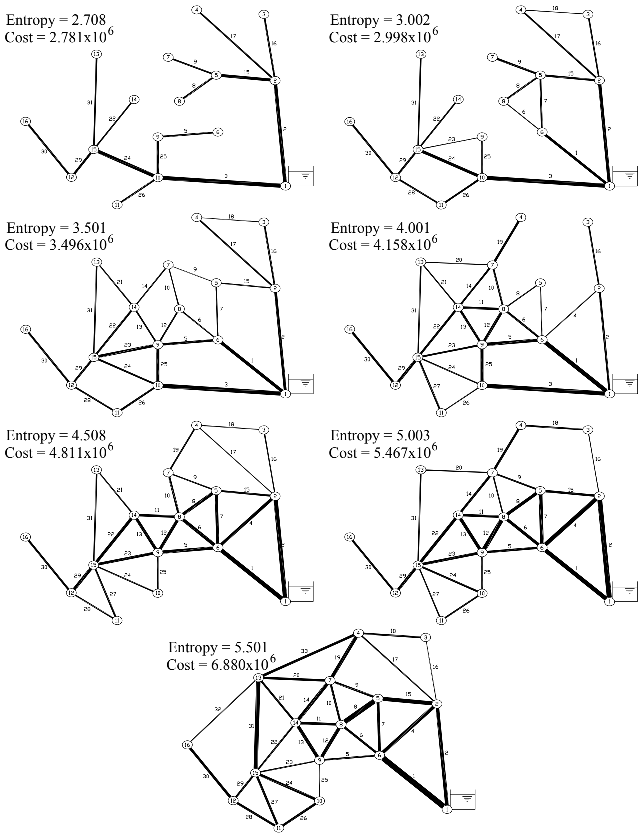

Figure 3: Design solutions for different levels of network entropy.

The analysis of these solutions leads to the following conclusions:

- the increase in the entropy was followed by an increase in the number of loops in the network. Keeping in mind that loops are a synonym of redundancy, and redundancy contributes to reliability, this result can be viewed as a demonstration of the relation between entropy and reliability;
- the increase in the entropy resulted in the increase in the diameters for the pipes located in the vicinity of the reservoir and in centre of the network. This phenomenon can also be viewed as an increase in reliability. As the diameters of the main pipes get bigger, the flexibility of the network will improve, and so the negative impacts resulting from an eventual failure of one of those pipes will be minimized;
- for the higher entropy levels the network is supplied only by two pipes leaving the reservoir. This result can be a direct consequence of the

increase in diameters mentioned above, that is, the diameters of these pipes are so big that, most probably, if one of them fails the supply will be assured by the other.

Table 2: Design solutions for different levels of network entropy.

Pipe	Diameter (mm)						
1	0	350	450	600	600	600	800
2	500	450	400	300	600	700	500
3	600	600	600	600	0	0	0
4	0	0	0	150	450	600	500
5	250	0	350	450	450	350	250
6	0	150	200	300	500	450	350
7	0	250	200	150	450	500	400
8	250	250	0	200	450	500	700
9	250	300	100	0	200	250	300
10	0	0	200	250	300	200	300
11	0	0	0	350	350	350	400
12	0	0	250	400	450	600	500
13	0	0	250	350	450	500	500
14	0	0	200	300	0	350	500
15	450	300	200	0	300	350	600
16	250	250	250	250	200	150	100
17	250	250	250	0	150	0	200

Pipe	Diameter (mm)						
18	0	125	100	0	125	250	250
19	0	0	0	350	250	350	500
20	0	0	0	150	0	200	350
21	0	0	200	200	200	0	300
22	250	250	300	350	450	450	250
23	0	125	400	300	400	400	200
24	500	450	300	200	200	300	400
25	350	200	400	450	200	200	150
26	250	300	300	150	0	250	350
27	0	0	0	250	250	200	350
28	0	250	250	0	200	0	350
29	350	300	300	400	450	400	300
30	300	300	300	350	300	300	400
31	250	250	200	200	200	200	600
32	0	0	0	0	0	0	150
33	0	0	0	0	0	0	400
Entropy	2.708	3.002	3.501	4.001	4.508	5.003	5.501
Cost (10^6\$)	2.781	2.998	3.496	4.158	4.811	5.467	6.880

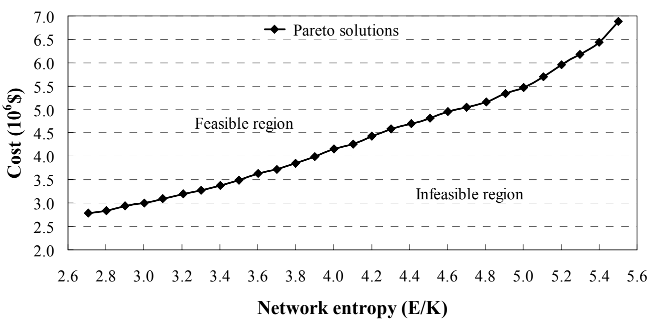

Figure 4: Pareto solutions (cost/entropy).

In the end, these solutions were compared with those arising from the economical design of the network, imposing different minimum pipe diameters, a procedure usually used as a means of conferring some level of reliability on the economical design. The minimum cost design was then found, imposing different minimum values for the pipes diameters, D_{min}, namely: $D_{min} = 0$ (pure economical design), 100mm, 125mm, 150mm, 200mm and 250mm. After achieving the designs, the entropy values were evaluated and these solutions were compared with those obtained before (Figure 5), leading to the following conclusions:

- the increase in the minimum diameter value was followed by an increase in the entropy;

- as the value of D_{min} increases the results deviate considerably from those obtained by the entropy procedure (the imposition of higher values for D_{min} results in overdesign of some pipes, mainly in the periphery of the network, and to the consequent cost increase);
- except for the solution obtained with D_{min} = 0, which is equivalent to *(E/K)min* = 0, the solutions obtained for different values of D_{min} present costs considerably higher than those obtained with similar entropy levels.

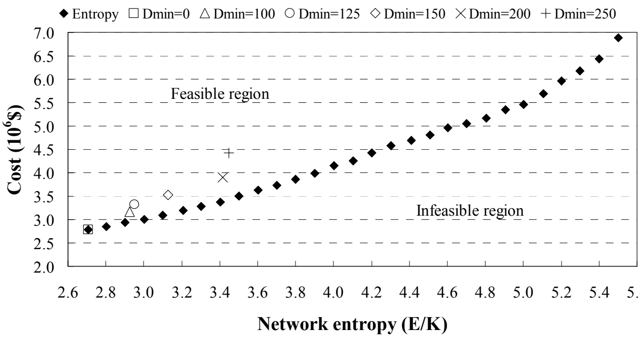

Figure 5: Minimum entropy constraints vs. minimum diameter constraints.

In the light of the above we can conclude that the imposition of minimum diameters can indeed be viewed as a contribution to increased network reliability, but it is not certain to be the most appropriate procedure to attain that goal.

4 Conclusions

This work tackles the reliable design of water distribution networks. The reliable design problem is transformed in an optimization problem with two objectives: minimization of cost and maximization of the reliability (here indirectly evaluated by the entropy, one of the most promising measures to evaluate water distribution network reliability). This problem is solved by the constraint method, by which one of the objectives is replaced by a constraint imposing a minimum level for that objective. In the present case the original two-objective problem gave way to a set of cost minimization problems, each constrained by a different minimum level of entropy. These problems were solved by the Simulated Annealing method.

The applicability of this methodology was illustrated with an example taken from the literature. The process begins by solving the cost minimization problem without any minimum entropy constraint, which resulted in a branched network, as expected. This solution presents the minimum value for the entropy, which is subsequently used as a reference. The problem is then solved again imposing different higher minimum levels for the entropy. This procedure yields a set of solutions, called Pareto solutions, arising from different levels of compromise

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between cost and reliability, which can be very helpful for decision makers. Analysis of the results shows that increased the entropy is followed by an increase in the network's redundancy, that is, its reliability. The Pareto solutions were compared with those arising from the economical design of the network imposing different minimum pipe diameters, a procedure normally used to confer some level of reliability on the economical design. This comparison showed that the imposition of minimum diameters does indeed contribute to an increase in reliability, but the solutions obtained with this procedure deviate considerably from the Pareto solutions, and therefore the use of this procedure is not advised.

References

[1] Alperovits, E., and Shamir, U. (1977). "Design of optimal water distribution systems." *Water Resources Research*, Vol. 13, No. 6, pp. 885-900.
[2] Hobbs, B.F., and Beim, G.K. (1988). "Analytical simulation of water system capacity reliability. 1. Modified frequency-duration analysis." *Water Resources Research*, Vol. 24, No. 9, pp. 1431-1444.
[3] Cunha, M.C., and Sousa, J.J.O. (1999). "Water distribution network design optimization: Simulated Annealing approach." *Journal of water Resources Planning and Management*, ASCE, Vol. 125, No. 4, pp. 215-221.
[4] Tanyimboh, T.T., and Templeman, A.B. (1993). "Calculating maximum entropy flows in networks." *Journal of Operational Research Society*, Vol. 44, No. 4, pp. 383-396.
[5] Walters, G.A. (1995). Discussion of "Maximum entropy flows in single source networks" de T.T. Tanyimboh, and A.B. Templeman. *Engineering Optimization*, Vol. 25, pp. 155-163.
[6] Xu, C., and Goulter, I.C. (1999). "Reliability-based optimal design of water distribution networks." *Journal of Water Resources Planning and Management*, ASCE, Vol. 125, No. 6, pp. 352-362.

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Multi objective approach for leakage reduction in water distribution systems

R. Magini, I. Pallavicini & D. Verde
Department of Hydraulics, Transportation and Roads,
University of Rome "La Sapienza", Italy

Abstract

This paper faces the problem of reducing leakages in water distribution systems (WDS) through dynamic heads control with pressure reduction valves (PRV). To achieve this aim a multi-objective optimization approach is followed for the location of control valves and for their setting during different working conditions. In particular, the objectives to be achieved are: 1) to minimize the leakages over all of the network, 2) to minimize the investment costs for the control devices. The objectives must be satisfied considering the variability of the nodal water demand. For a more realistic simulation of the WDS performance the implementation of a fully pressure-dependent leakage specification is required. The multi-objective optimization problem is tackled using a MOGA (Multi-Objective Genetic Algorithm) with a Pareto based approach. The hydraulic modelling of the network is performed with the software EPANET2. The nodal demands are considered to be uncertain input parameters and the most robust solution is also found. The procedure is here applied to the WDS of a small town not far from Rome afflicted by heavy losses. A reduction of about 50% of initial water losses is achieved.
Keywords: water distribution systems, leakage reduction, pressure control, multi objective optimization, genetic algorithms.

1 Introduction

Nowadays the problem of water losses control in WDS is more and more pressing owing to the growth of population, the increasing of water consumptions and at the same time because of the deterioration and the reduction of available drinkable water resources as a consequence of pollution and climate changes. Leakages in urban water networks can be a very high percentage of the

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supplied water. In some cases in Italy they can be even more than 50%. This lost volume represents an economic damage not only for the cost of water pumping and treatment but even because it makes necessary investments into systems capacity expansion or in searching for other water sources. Losses are usually divided into 'apparent losses', including unauthorised consumptions by illegal connections and metering inaccuracies, and 'real or physical losses' produced by all types of leaks, burst or overflows, which are due to the state of degradation of the networks and to their bad management. Usually all the methods of leak detection are expensive, complex and time consuming. Physical losses, moreover, sometimes manifest themselves with local huge leakages, sometimes they are the sum of small leakages distributed all over the network. Then, it is necessary to define a critical threshold of the leakage over which the benefits from the detection and repairing are greater than the costs. The level of this threshold depends on the economic value of the resource, including all the costs for drawing, conveying, treating, pumping, etc. The threshold increases as the number of leakage points increases, that is, when it is related to 'distributed leaks'. In this case leak detection and repairing operations are often non realistic and too expensive also if the volume of the water losses is relevant. Then it may be better to plan medium-long term rehabilitation policies of the network combined with suitable management policies for controlling pressure heads in the system. Pressure regulation can be achieved by means of pressure reduction valves (PRV) remotely controlled, whose purpose is to decrease excessive pressures, ensuring the service conditions (Reis *et al* [1], Lumbers and Vairavamoorthy [2], Araujo *et al* [3]).

In this paper a method to determine the optimal number and location of the valves and their optimal setting for different demand scenarios is presented and the case-study of a little town not far from Rome has been considered. At this purpose the network simulation has been carried out considering the leakages as pressure-driven through the 'emitter function' of EPANET2 (Araujo *et al* [4], Burrows and Zhang [5], Tabesh *et al* [6]).

Firstly a calibration procedure for assessing nodal demands and distributing properly leakages throughout the network has been performed. The successive optimization analysis for the location of control valves and for their setting during different working conditions has been tackled using a MOGA (Multi-Objective Genetic Algorithm) with a Pareto based approach. Finally a robust optimization analysis, aimed at finding the most robust solution in respect to non deterministic parameters, has been also performed. In particular uncertainty of water demands has been considered.

2 Calibration and assessment of leakage and demand

In order to treat water losses as pressure dependent quantities, the calibration of the WDS simulation model has been performed concentrating leaks at the nodes and considering their flow Q^{leak} as a function only of the local pressure p according to the expression:

$$Q^{leak} = Kp^n \quad (1)$$

with n=1.18, as experimental experiences have shown (Goodwin [7]). The presence of the leakage term modifies the continuity equation of the generic node i as follows:

$$\sum_{p=1}^{N_i} Q_p - D_i - K_i p_i^n = 0 \qquad (2)$$

where Q indicates the flow in the p^{th} trunk connected to the node i and D the water demand at the node. For the parameter K, called emitter coefficient, the following expression has been considered:

$$K_i = c\sum_{p=1}^{N_i} \frac{1}{2} L_p \qquad (3)$$

where L is the length of the p^{th} trunk connected to the node i and c an unknown coefficient valid for the entire network.

To calibrate the parameter c, a procedure presented by Araujo [4] has been followed. This method requests to know flow and pressure measurements in the inlet and outlet nodes of the network. The difference between the inlet and the outlet flow represents the discharge Q^{net} flowing into the network. For the evaluation of the coefficient c an estimation of the percentage $\%F$ of the global leakage discharge at the minimum night flow (MNF) time has been also carried out. Then the equality between the supposed total leakage flow at the MNF time and the sum of leakage discharges modelled with eqn (1) has been imposed:

$$\left[Q_{tot}^{leak} \equiv \%FQ^{net} = \sum_{i}^{N} K_i(c) p_i^{1.18} \right]_{MNF} . \qquad (4)$$

The estimation of the coefficient c can be carried out with an iterative procedure. In this work an optimization problem solved with GA coupled with the hydraulic model of the network has been adopted.

The coefficient c has been estimated for the MNF hour but it has been supposed constant at any consumption conditions. For this reason it is necessary to calculate a coefficient f_t for each time interval t in which the day is split (i.e. in 24 hours) in order to satisfy the following equation:

$$Q^{net}(t) = f_t \sum_{i=1}^{N} D_i(t) + \sum_{i=1}^{N} K_i p_i^{1.18}(t) . \qquad (5)$$

This strategy enables a better assessment of the nodal demands, taking into account the measurements available in the inlet and outlet nodes and considering the presence of leakages too. Also in this case the estimation of the coefficients f_t can be carried out with a GA coupled with the hydraulic model of the network.

Once the c and f_t coefficients have been estimated the WDS is completely characterized.

3 Optimization procedure

An optimization procedure has been set up for finding the optimal number of PRVs, their best location inside the network and their setting for different demand scenarios with the aim of minimizing the leakages and the costs (number of valves). The problem constraints were defined by the lowest acceptable nodal pressures at each node in order to satisfy the service conditions.

The optimization process has been divided into two phases. In the first one, denoted 'designing phase', the optimal position and number of valves has been found. In this phase the MNF condition, for which the maximum value of leakages is expected, has been considered.

The opening degrees of each valve define the number of decision variables. Their number is equal to the number of pipes of the network and each of them can assume 11 integer values in the range 0-10. The value 0 corresponds to the absence of the valve, while the other values define different opening settings. A set of L binary counters x identifies the number of installed valves. Then the two objective functions are:

$$\min\_ \; Leakage = \sum_{i}^{N} K_i P_i^{1.18}, \tag{6a}$$

$$\min\_ \; Valve\_number = \sum_{k=1}^{L} x_k \,. \tag{6b}$$

The problem constraints are represented by the hydraulic equations of the network and by the minimum pressure levels to be assured at each node. To solve this problem, an evolutionary MOGA algorithm, using specific operators such as a multi directional crossover and elitism, has been employed and combined with the hydraulic solver Epanet2.

In this phase also the valve opening degrees have been implicitly optimized. The goal of this phase was to find the best configuration that gives the largest reduction of leakages throughout the whole day. However the optimization has been performed considering only the MNF hour, when the greatest value of water losses should be expected.

A multi-objective optimization problem yields a set of non-dominated solutions, which form the Pareto front in the objective functions space. It is then necessary to select the best one with suitable criteria.

The second phase, denoted 'management phase', has been carried out in order to determine the best setting of the installed PRVs for the different demand scenarios of a typical day. Now the only objective to be considered is the minimization of the leakages, eqn (6a). Moreover the number of decision variables is obviously reduced to the number of PRVs previously determined. Different optimizations have been performed for the 24 hours.

4 Robust design

Robust design is a procedure of optimization, which takes into account that some parameters of the problem can be affected by uncertainty. In particular here we have considered water demands at each node as stochastic variables.

In an optimization problem a robust solution must be not only optimal, but also not too sensible to variations of the uncertain parameters. The method used was based on a Montecarlo generation of different scenarios for any possible solution evaluated by the GA. A mean value of the fitness function and of the other outputs are then considered. The solution is robust if the mean value of the fitness function is optimized and its standard deviation minimized.

This methodology has been applied to the second phase optimization problem. Water demands at each node have been considered to be normal distributed, with a mean value deriving from a study on the characteristics of the end users (Pallavicini *et al* [8]). This value is the same used both in the first and in the second phase. The standard deviation has been considered variable in the different hours of the day, following experimental studies on instantaneous water demand (Guercio *et al* [9]).

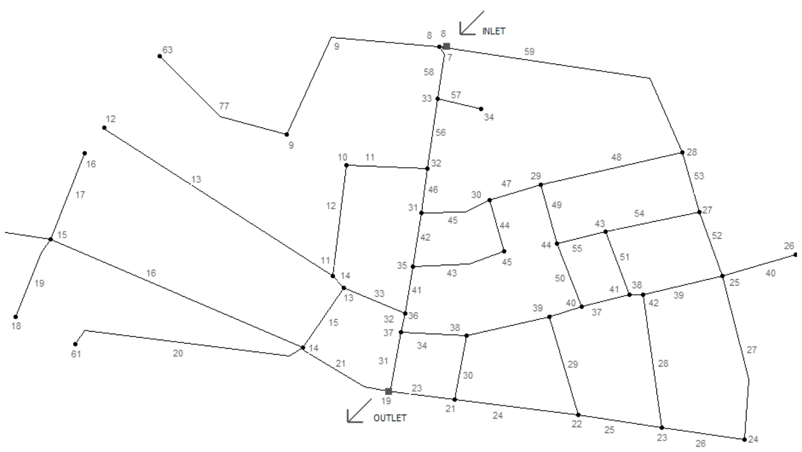

Figure 1: The case study WDS.

5 Case study

The application of the discussed procedure has been carried out on the real water distribution network of Piedimonte S. Germano, a small town of about 4700 inhabitants in the south of Latium. This network is affected by serious water losses. Actually only the portion of the network serving 714 equivalent users and supplying the oldest part of the town has been considered, fig. 1. The network, mainly composed by cast iron pipes and partly by HDPE pipes, is situated at a mean elevation of 112 m asl and is served by a unique reservoir located at 180 m asl. The inlet and outlet nodes are monitored with electromagnetic flow meters and pressure transducers connected to data loggers.

5.1 Calibration results

The results of this preliminary study are summarized in fig. 2. In particular in fig. 2(a) the values of the difference between the inlet and the outlet measured flows, Q^{net}, and of the measured pressure at inlet node are represented for 24 hours. In the same figure is also reported the daily global demand pattern *D*, obtained by means of an accurate description of the different kind of users at each network node. From the Q^{net} curve it is evident that the MNF occurs at 3:00 am. At this time the percentage *%F* of the leakages has been fixed equal to 95% of Q^{net}. In this figure the leakages curve has been obtained from the difference between the Q^{net} and the demand without considering any dependence on the pressure.

The calibration procedure has yielded the coefficient *c* and the *N* emitter coefficients *K* calculated according to eqn (3). Then the global demand has been corrected using the coefficients ft. The final results of the calibration are summarized in fig. 2(b) where the modified behaviour of the demand and leakages can be observed. In this figure leakages follow a more realistic behaviour as they are related to pressure according to eqn (1). The same calibration procedure has enabled to estimate the distribution of leaks at each node and at different hours. The main result of this calibration process has been the estimation of the daily water volume lost that is equal to 196 m^3. Considering equal to 357 m^3 the water volume entering the network, leakages reach the very large percentage of about 55%.

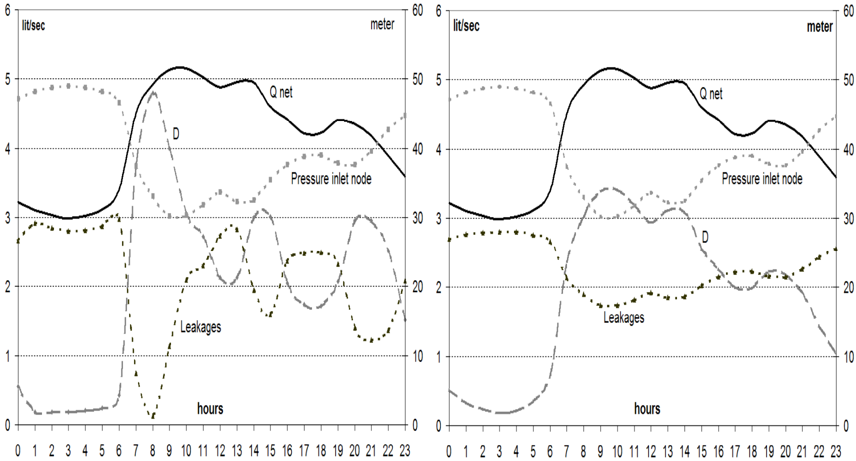

Figure 2: (a) (left) available data; (b) (right) calibration's results.

5.2 First phase results

The multi-objective problem, aimed at determining the number of the PRVs and their best position, has been solved by means of the MOGA II algorithm implemented by the software mode Frontier [10]. Some C++ utilities have been developed to make MOGA II interact with the hydraulic solver Epanet2.

It is interesting to mention that, because the number of decision variables are 51 (number of pipes) and each one can assume 11 different values (opening degrees), the search space dimension is equal to the huge value of 11^{51}. The MOGA has been executed with a population of 1000 individuals and with 50 generations, for a total of 50000 solutions evaluated. In fig. 3 the space of the objective functions with some of the obtained solutions is represented. Focusing on the Pareto frontier it is easy to deduce that the introduction of more than 4 valves is useless because the consequent marginal reduction of the losses would be negligible. The location of the 4 valves of the best solution (circled point), is represented in the fig. 4. Their position is close to the inlet node because the network is characterized by high pressures owing to the reservoir elevation.

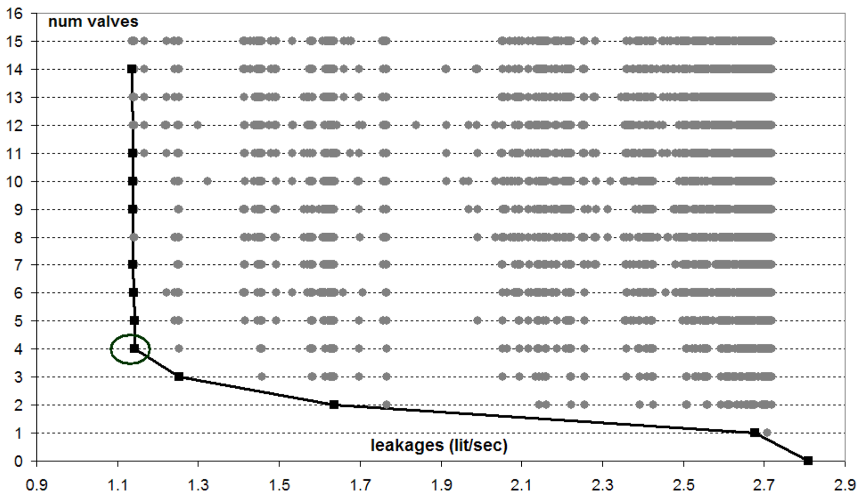

Figure 3: Results on objective functions space, Pareto Front.

5.3 Second phase results

The second phase has been concerned with the optimization of the opening degrees of the four installed valves during the 24 hours with the only objective to minimize water losses. This problem is single-objective but it has been solved using the MOGA II algorithm anyway. In this case the solutions space has the dimension of 11^4. The 24 optimizations processes have been performed with a population of 150 individuals and 10 generations, therefore the algorithm has found the solution by evaluating 1500 possible solutions. The effect of the four installed valves and their dynamical setting is summarized in fig. 5. The comparison with fig. 2(b) shows that leakages are drastically reduced and that they are almost constant during the whole day. The effect of the valves in fact maintains the pressures close to the minimum requested value. The final result is that the average supplied volume of water in a day is reduced to about 256 m^3. This means that a reduction of about 50% of the lost volume has been achieved. Leakages are reduced to 95 m^3/day corresponding to about 37% of the total entering volume.

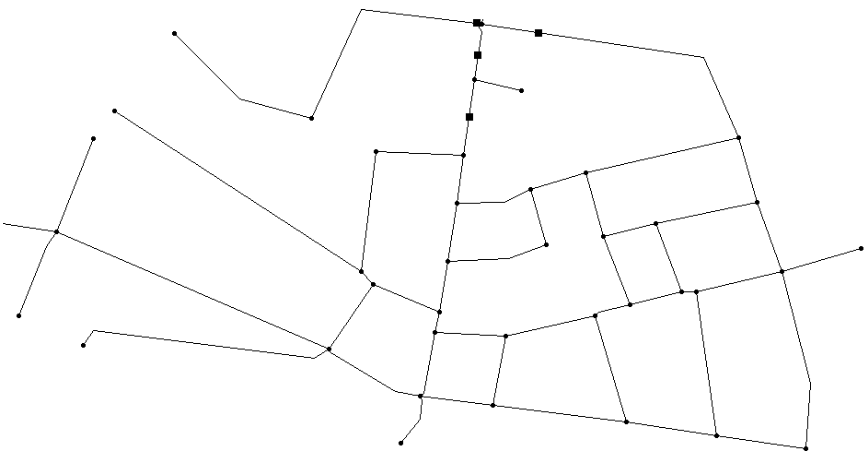

Figure 4: Position of the four valves inside the network.

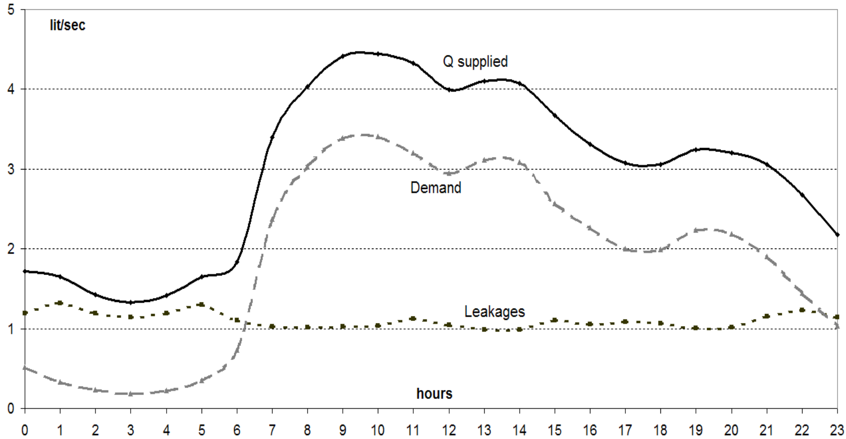

Figure 5: WDS daily average performance with the four valves.

5.4 Robust design analysis

The valve setting optimization has been performed at the two extreme demand scenarios of 3:00 a.m. and 9:00 a.m., considering for each node a normal distribution of the demand with a standard deviation respectively equal to 10% and 5%. These values are about twice the results obtained from experimental data (Guercio *et al* [9]). With the Montecarlo technique 15 different demand scenarios have been generated. The standard deviation of the objective function (min_leakages) has been fixed equal to 2% of the result obtained in the optimization of previous phases.

In both cases the valve setting obtained considering the robust design coincides with the one previously obtained with a deterministic approach.

6 Conclusions

In this paper a multi-objective approach for the optimal location and the optimal setting of pressure control valves in a water distribution network has been presented. The multi-objective optimization problem has been tackled using MOGA with a Pareto based approach. The methodology has been applied to a district of the water distribution network of a small town in the south of Latium (Italy) characterized by water losses exceeding the 50% of the entering water volume. A calibration process has been firstly carried out in order to obtain a complete and realistic characterization of water demand and leakages. In particular leakages have been considered strongly related to pressures. The successive optimization procedure has been split into two phases. In the first one the optimal location and the best number of valves has been determined at MNF condition, when leakages are expected to be largest. The multiple objective has been here represented by the minimization of the total leakage volume all over the network and the minimization of the costs. The optimal solution suggests the installation of four valves. In the second phase the optimal setting of the installed valves has been defined for the whole day. A reduction of about 50% of the lost volume has been achieved and leakages decrease to about 37% of the total entering volume. Finally a robust design analysis was also carried out in order to examine the sensitivity of the optimal solutions with respect to uncertainty of water demand.The good results achieved could be further improved introducing a forecasting model for water demand to be integrated in a real time system for the optimal management of the valves. At this aim it should be advisable to design an optimal monitoring system for flow and pressure which could enable during the ‘calibration phase’ a better characterization of the leakage and demand distribution, and in the ‘management phase’ could support the real time control.

References

[1] Reis F.R., Porto R.M. & Chaundhry F.H., Optimal location of control valves in pipe networks by genetic algorithm. *J. Water Resour. Plng and Mgmt.*, ASCE 123 (6), 1997, pp. 317–326.

[2] Lumbers J. & Vairavamoorthy K., Leakage reduction in water distribution systems: optimal valve control. *J. Hydraulic Engineering*, ASCE 124 (11), 1998, pp. 1146–1153.

[3] Araujo L.S., Coelho S.T. & Ramos H.M., Optimization of the use of valves in a network water distribution system for leakage minimization. *Proc. of Int. Conf. on Advances in Water Supply Management*, London, pp. 97–107, 2003.

[4] Araujo L.S., Coelho S.T. & Ramos H.M., Estimation of distributed pressure-dependent leakage and consumer demand in water supply networks. *Proc. of Int. Conf. on Advances in Water Supply Management*, London, pp. 119–128, 2003.

[5] Burrows R. et al. & Zhang J. et al., Introduction of a fully dynamic representation of leakage into network modelling studies using Epanet. *Proc. of Int. Conf. on Advances in Water Supply Management*, London, pp. 109–118, 2003.

[6] Tabesh M., Asadiani Yekta A.H. & Burrows R., Evaluation of UFW and real losses in WDS by hydraulic analysis of the system considering pressure dependency of leakage. *Proc. of CCWI*, Exeter, pp. 125-130, 2005.

[7] Goodwin S.J., The result of the experimental program on leakage and leakage control. Technical Report TR 154, W.R.C., Swindon, UK, 1980.

[8] Pallavicini I, Magini R & Guercio R., Assessing the spatial distribution of Pressare Heads in Municipal Water Networks, *Proc. of CCWI*, Exeter, pp. 257-262, 2005

[9] Guercio R., Magini R. & Pallavicini I., Monitoraggio ed analisi di consumi idrici residenziali. *Proc. of XXIX Convegno Idraulica e Costruzioni idrauliche*, Trento, Vol.C, pp.271-278, 2004. In Italian.

[10] modeFrontier, http://frontier.enginsoft.it/software/index.html

Analysis of intermittent supply systems in water scarcity conditions and evaluation of the resource distribution equity indices

C. M. Fontanazza, G. Freni & G. La Loggia
Dipartimento di Ingegneria Idraulica ed Applicazioni Ambientali, Università degli Studi di Palermo, Italy

Abstract

Generally, urban water distribution shortage situations are solved by introducing discontinuous service and rationing the available water resources. This approach is widely adopted, not only in developing countries but also in developed ones, for solving short term scarcity conditions which can be caused by unpredicted drought periods. Intermittent distribution has the advantage of requiring small financial efforts but it leads to network operating conditions that are very far from the usual design ones. With the aim to analyse and describe the water supply network behaviour in intermittent conditions, a network hydraulic model has been set up in which both user and manager dependent regulation structures have been schematised (pumps, private reservoirs, etc.). The analysis allowed evaluating network performance introducing several control strategies so suggesting operational plans for reducing the impact of water scarcity events on population and improving resources distribution equity. The presented model has been applied to the water distribution network of Palermo (Italy).
Keywords: distribution network performance, water scarcity conditions, intermittent distribution.

1 Introduction

The urban development and the population growth have several consequences connected with design and management of water distribution systems. The contemporary increase in water consumption and reduced availability of water resources produce unbalances between water demand and offer, so leading to

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water shortage scenarios. This happens even in those situations where water resources procurement was not historically an issue.

Water shortage situations are commonly solved by using discontinuous water distribution and rationing the available water resources. This approach is widely adopted not only in developing countries [1–4] but also in developed ones for solving short-term scarcity conditions, which can be caused by drought periods [5].

Intermittent distribution is quite frequent in Mediterranean countries as well. In this area, the lack of natural resources management and network maintenance plans, explicitly considering the possibility of scarcity scenarios, produces unexpected water shortage situations that can be handled only by mean of emergency interventions. Intermittent water distribution has the advantage of requiring small financial efforts but it leads to network operating conditions that are very far from the usual design ones. The network is subjected to cyclical filling and emptying periods and users need to collect water during distribution periods for covering their needs when supply service is not available.

In intermittent distribution, the users try to compensate water service intermittency by searching new local resources, when available (as an example by perforating private wells), or, more commonly, by building private reservoirs, used for collecting water during serviced periods and distributing it when public water service is not available. During intermittent distribution periods, the public network is greatly influenced by the presence of such reservoirs that are usually filled in a very short period after the reactivation of water service, leading to very high peak flows and, consequently, inequity in water resources allocation among population. Moreover, those local reservoirs are often over-designed in order to take into account higher water consumption and possible leakages. In these cases, the intermittent distribution is useful to limit water losses due to pressurization more than to limit water consumption by users.

Intermittent distribution systems are affected by several problems connected with inconsistence between design and operational conditions. In the design phase, in fact, it is supposed continuous operational conditions and sufficient water resources to match the volumes required by the users and the available ones. When a continuous system is managed as an intermittent one, pressure and flow distributions are inadequate and not homogenous.

Intermittency generates inequitable water distribution due to pressure dependent flow conditions, with obvious disadvantages for consumers located faraway from the supplying nodes or at higher elevation in the network. In distribution systems designed for continuous water supply, the consumers exposed to intermittent supply conditions are likely to collect as much water as possible in their reservoirs whenever the service resumes [6]. In this condition, consumer reservoirs are filled once the supply has been restored and this contemporary use of water service generates larger peak flows than predicted in the network design process, increasing the pressure losses in the network. Consequently, disadvantaged consumers will always collect less water than those nearer to the source. Intermittent distribution can also have a large impact on

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water quality allowing for the introduction of soil in the pipes when they are empty [7].

For these reasons, it is absolutely necessary designing and managing water distribution systems according to their operational conditions in order to improve system performances and to deliver equitably the available water resource. Intermittent distribution networks, therefore, have to be designed in a particular way, absolutely different to those applied to systems delivering water 24-hours per day [8].

In order to efficiently analyze urban distribution networks in scarcity conditions, it can be helpful to evaluate how water scarcity and intermittent service affect water consumption. The study proposes a methodology for identifying those users that are more disadvantaged by the intermittent distribution condition providing a useful tool to be used when managing a network in such a delicate operational condition. The identification of disadvantaged users is carried out by the mean of network performance indicators specifically defined for intermittent distribution and described in the next paragraph. In the study, a real distribution network has been analysed proposing some indices for assessing the equity of water service in intermittent distribution conditions.

2 Intermittent distribution network mathematical model

The primary objective of a water distribution system is to provide water at a sufficient pressure and quantity to all its users. In traditional demand-driven analysis, the network solution is achieved by assigning the assumed demands for all nodes and computing the nodal pressure heads and link flows from the equations of mass balance and pipe friction headloss [9]. For networks operating under intermittent conditions, a demand-driven analysis can yield nodal pressures that are lower than the minimum required service level or which even become negative. In the real network, the design demands would not be met. Although this is a well-known problem that has been tackled by many researchers [10–16], it is still sometimes ignored. Since the 1980s, researchers have proposed various methods to compute actual water consumptions, node pressures and flows in networks operating in conditions different from design ones (such as intermittent systems). Most of the proposed methods involve an assumption on the relationship between pressure and outflow at the demand nodes. These methods are generally termed head-driven analyses.

Bhave [10], who firstly acknowledged demand driven analysis does not behave well when node heads are lower than required service standard ones, proposed the following pressure-consumption relationship:

$$\text{If } H_j^{avl} < H_j^{\min}, \quad q_j^{avl} = 0 \tag{1a}$$

$$\text{If } H_j^{avl} \geq H_j^{\min}, \quad q_j^{avl} = q_j^{req} \tag{1b}$$

where q_j^{avl} is the actual outflow at node j, q_j^{req} is the required outflow at that node (water demand), H_j^{avl} is the available head and $H_j^{\min}$ is the minimum head required to have outflow at the node.

Germanopoulos [11] suggested the use of an empirical pressure-consumption relationship to predict the outflows at various nodal head:

$$\text{If } H_j^{avl} \le H_j^{\min}, \quad q_j^{avl} = 0 \tag{2a}$$

$$\text{If } H_j^{avl} > H_j^{\min}, \quad q_j^{avl} = q_j^{req} \left\{1 - 10^{-c_j\left[\left(H_j^{avl} - H_j^{\min}\right)/\left(H_j^{des} - H_j^{\min}\right)\right]}\right\} \tag{2b}$$

where H_j^{des} is the head required to satisfy the water demand, q_j^{req}, at the node j and c_j is a calibration parameter ranging from 1 to 5.

Then, Wagner *et al.* [12] proposed the use of a parabolic curve to represent the pressure-consumption relationship at a demand node for head between $H_j^{\min}$ and H_j^{des}:

$$\text{If } H_j^{avl} \le H_j^{\min}, \quad q_j^{avl} = 0 \tag{3a}$$

$$\text{If } H_j^{\min} < H_j^{avl} < H_j^{des}, \quad q_j^{avl} = q_j^{req} \left(\frac{H_j^{avl} - H_j^{\min}}{H_j^{des} - H_j^{\min}} \right)^{\frac{1}{n}} \tag{3b}$$

$$\text{If } H_j^{avl} \ge H_j^{des}, \quad q_j^{avl} = q_j^{req} \tag{3c}$$

where n is a calibration parameter ranging from 2 to 1.

Reddy and Elango [13] introduced a method completely different from the others previously referred to. The authors suggested a pressure-consumption function without above boundary as the following equations show:

$$\text{If } H_j \le H_j^{\min}, \quad q_j^{avl} = 0 \tag{4a}$$

$$\text{If } H_j \ge H_j^{\min}, \quad q_j^{avl} = S_j \left(H_j - H_j^{\min}\right)^p \tag{4b}$$

taking the value of the coefficient S_j from the following condition:

$$\text{If } H_j = H_j^{des}, \quad q_j^{avl} = q_j^{req} \tag{5}$$

and being p a coefficient ranging from 0.5 to 1.

This method has been introduced to evaluate the node consumptions in water distribution networks operating in intermittent conditions. In this case, all the users are endowed with private reservoirs and the node outflow is the maximum taken by the network, only related to the available nodal head. The outflow stops when the reservoir is just completely full.

Where water distribution is periodically provided on intermittent basis, the users often provide private reservoirs with pumps to collect as much water as possible even if nodal pressure is lower than minimum required having outflow at the node. In such situations, the method proposed by Reddy and Elango [13] has to be modified to take into account the pressure-consumption relationship in the range of node head lower than the minimum value. In order to do this, eqn. (6) has been defined setting $H_j^{\min}$ equal to zero in eqn. (4b):

$$q_j^{avl} = k \cdot H_j^{\ p} \tag{6}$$

where k and p are calibration coefficients.

This algorithm has been readily implemented into an existing hydraulic network solver, EPANET 2 [17]. Furthermore, a private reservoir under the roof and a pump have been associated with each node (fig. 1) thus providing a complete model for analysing intermittent distribution networks.

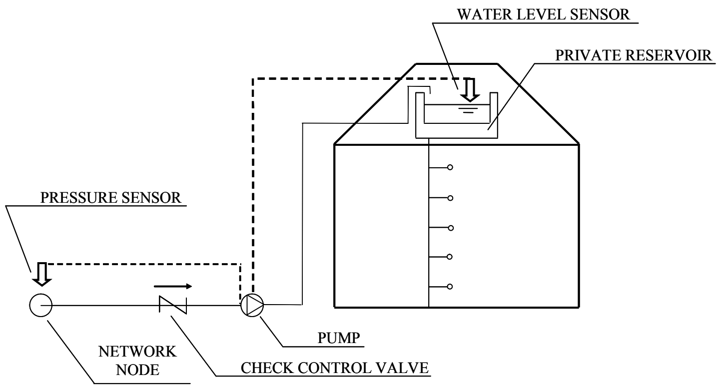


Figure 1: Distribution node numerical scheme.

The reservoir has been designed according to nodal daily water demand and a pump has been chosen being able to fill the reservoir in 4 or 5 hours. The pump is turned on if the reservoir is empty and turned off if the reservoir is full or the pressure on the network is negative.

In order to evaluate the equity in the distribution during intermittent operational conditions, two performance indices have been proposed in the present study:

1. the ratio between the water volume supplied to the users in a service cycle (if the service is intermittent on daily basis, the service cycle is correspondent to two days) and the user demand:

$$EQ1 = \frac{V_{\text{int}}}{D} \tag{7}$$

where V_{int} is the water volume supplied to the users in a service cycle and D is the user water demand in the same period;

2. the ratio between the water flow discharged to the user during a service day in intermittent and continuous distribution conditions:

$$EQ2 = \frac{Q_{\text{int},i}}{Q_{cont,i}} \tag{8}$$

where $Q_{int,i}$ is the water volume supplied to the users in a service cycle and $Q_{cont,i}$ is the user water demand in the same period.

The index *EQ1* represents the ratio between supplied and required water volumes in intermittent distribution and it is able to identify the users that will obtain less water than their needs and the advantaged users that will have available volumes even higher than their needs. But even if globally in a service cycle water volumes distributed at the users do not greatly differs among them, wide differences may be possible during the distribution period because advantaged users can fill their reservoirs much faster than disadvantaged ones. This aspect can create difficulties in water supply of disadvantaged users and it can modify the users' perception of the water service reliability. For this reason, the index *EQ2* can be useful for analysing the behaviour of the network in different operational periods (at the start or at the end of the distribution service after a 24-hours stop).

3 The adopted case study

To provide a more effective description of the proposed methodology, an analysis has been conducted on one of the 17 distribution networks of Palermo city (Sicily). This network has been chosen because it is recently built and it is precisely known all its geometric characteristics, the number and the distribution of user connections, the water volumes delivered and measured, and pressure and flow values in a few important nodes.

The network is fed by two tanks at different levels that can store up about 40.000 m^3 per day, and supply around 35.000 inhabitants. It has been designed to deliver about 400 l/capita/d but the actual mean consumption is about 260 l/capita/d. Pipes are made of polyethylene and their diameters ranging from 110 to 225 mm. The network is about 40 km long and users' elevation ranges between 47 m and 3 m above the sea level. Fig. 2 shows the distribution network adopted in the present study.

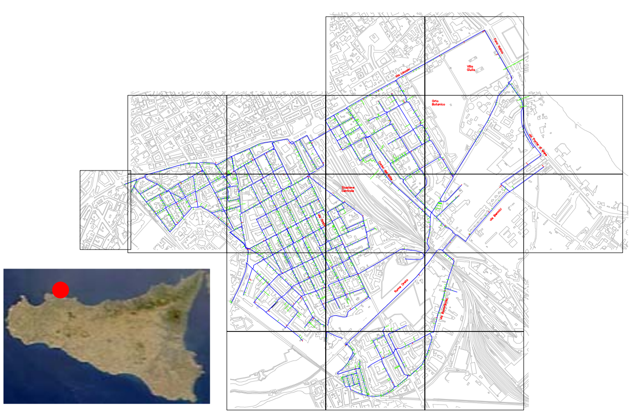


Figure 2: Case study network scheme.

However, because of the great amount of water losses occurring in the pipe connecting the tanks with the network and the recurrent lack of water resources, the water service manager has decided to operate the network on intermittent basis and introduced pressure reduction valves at the network inlets in order to reduce pressures and consequently leakages.

4 Model calibration and network simulation

The network has been simulated by the mean of EPANET2 model in which each supply node has been simulated according to the scheme provided in fig. 1. The model has been calibrated by the mean of a large set given by six months of continuous water flow data at the two network inflow nodes and correspondent pressure and flow data in 6 nodes distributed inside the network. The data has been collected on hourly basis.

The only parameter that has been considered in the calibration process was the pipe roughness and the calibration has been performed by the least square method fitting the simulated pressures and flows in the network internal nodes with the measurements. Fig. 3 shows the calibration results. The relative pipe roughness obtained from the calibration was equal to 0.64 mm.

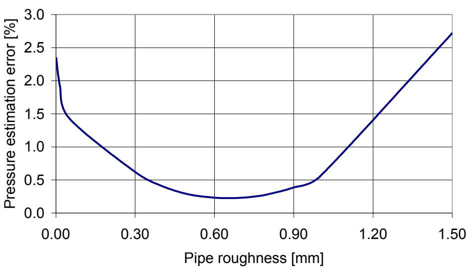


Figure 3: Modelling calibration results.

The model has been firstly run in order to analyse pressures distribution over the network in continuous and in intermittent distribution condition in order to identify the different network behaviour in the two situations. Afterwards a long term analysis has been performed over a service cycle in order to estimate the inequities in water distribution when the network has been subjected to intermittent distribution.

5 Results discussion and possible network management strategies

As discussed above, network diameters are largely overestimated with respect to the real need of population. The design user water demand is, in fact, almost two times of the real population needs. For this reason, in ordinary conditions the network is characterised by low water velocities and correspondently high pressures. The high pressures on the network caused in the past high leakages and, for this reason, pressure reduction valves have been introduced for reducing the problem (fig. 4). As discussed above, the level of leakages did not allowed to maintain continuous distribution (at least in summer period) in the last 5 years and intermittent distribution on daily basis was introduced as a common practice convincing the users to build up local private reservoirs with schemes presented in fig. 1.

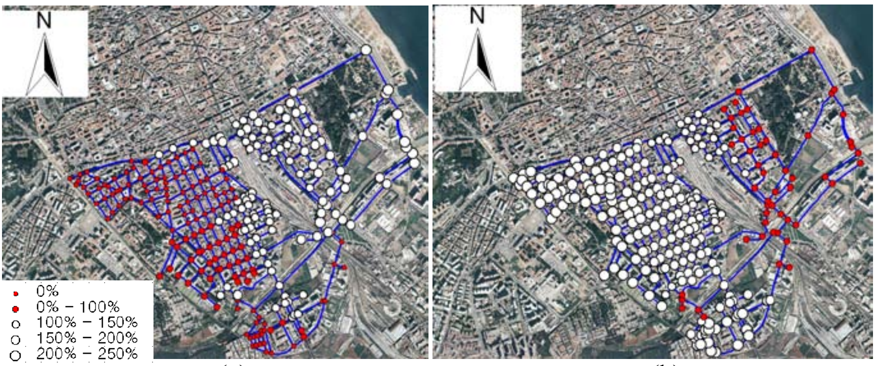
Figure 4: Water head distribution over the network.

The network was analysed by the mean of a long term simulation, with a time step equal to one hour, involving a whole service cycle in intermittent distribution condition. In this simulation, water resources availability is considered equal to the demand (intermittent distribution is adopted only for leakages reduction) so the iniquitous water supply is due to wrong distribution of resources among users. Index *EQ1* has been computed for the whole period and for each network node. The index *EQ2* has been computer for each node and each analysis time step.

The analysis of equity index *EQ1* shows that in intermittent conditions the inequity in water volumes distributed to users are limited within 15% (fig. 5). This number can be considered acceptable because it does not generate a great compression of user water consumption but it has to be stressed that this value has been generated in a condition where water supply would be able to fully satisfy users demand.

Figure 5: Distribution of EQ1 values over the network.

The analysis of index *EQ2*, on the contrary, has demonstrated that, in the first part of the service day (just after the restart of the water service), the lower nodes drain the most part of the available resources for filling their local reservoirs showing values of *EQ2* near to 250%; in the meantime, at higher elevated nodes, water is still not available and *EQ2* values are near to zero (fig. 6(a)). When lower reservoirs are filled (fig. 6b), water resources are available for elevated nodes that show high values of *EQ2* while lower nodes (where reservoirs are filled) show low *EQ2* values. *EQ2* is highly variable and it demonstrates that the network subjected to intermittent distribution will work in conditions that are quite far from the design ones and also from the operating conditions that take part in continuous distribution.



(a) (b)

Figure 6: EQ2 values at the beginning (a) and at the end (b) of the service period.

6 Conclusions

The present study proposed a methodology for analysing a distribution network under intermittent distribution condition. The methodology has defined two performance indicators able to identify inequality in water supply both considering long term effect with user consumption compression and short term during the service cycle with large variations in network hydraulic behaviour.

The application of the methodology to a real case study has shown that intermittent distribution can greatly affect both water availability for the users and the behaviour of the network. In the case study, even if water supply would be sufficient for fulfilling user demand, intermittent distribution will create inequalities among users by reducing water supply to high elevated nodes and increasing supply to lower nodes. The effect on the short term behaviour of the network is much higher with differences higher than 200% in the ordinary continuous distribution condition.

The intermittent distribution has thus a great impact on users and networks. Users change their water supply patterns with higher peaks (connected with the filling of local reservoirs) and large periods with very low discharges and they can also receive higher or lower water volumes depending on their elevation and position in the network. Intermittent distribution changes radically the behaviour of the network with parts of it that are interested by flows that are much higher than the design ones and parts that are interested by almost null discharges.

References

[1] Vairavamoorthy, K., Akinpelu, E., Lin, Z.& Ali, M., Design of sustainable system in developing countries. *Proceedings of the World Water and Environmental Resources Challenges*, Environmental and Water Resources Institute of ASCE, Orlando, Florida, 20-24 May, 2001.

[2] McIntosh, A. C. & Yñiguez, C. E., *Second Water Utilities Data Book: Asia and Pacific Region*, Asian Development Bank: Manila, 1997.

[3] McIntosh, A. C., *Asian Water Supplies*, Reaching the Urban Poor Asian Development Bank: Manila, 1993.

[4] Hardoy, J. E., Mitlin, D. & Satterthwaite, D, (2001). *Environmental Problems in an Urbanizing World: Finding Solutions for Cities in Africa, Asia and Latin America*, Earthscan: London.

[5] Cubillo Gonzales, F. L. & Ibanez Carranza, J. C., *Manual de abastecimento del Canal de Isabel II*, Graficas Fanny: Madrid, 2003.

[6] Totsuka, N., Trifunovic, N. & Vairavamoorthy, K., Intermittent urban water supply under water starving situations. *Proceedings of 30th WEDC International Conference,* Vientiane, Lao PDR, 2004.

[7] Yepes, G., Ringskog, K., & Sarkar, S, The High Cost of Intermittent Water Supplies. *Journal of Indian Water Works Association*, **33(2)**, 2001.

[8] Batish, R., A New Approach to the Design of Intermittent Water Supply Networks. *Proceedings of World Water and Environmental Resources Congress 2003*, eds. P. Bizier & P. DeBarry: Phil, Penn., USA, 2003.

[9] Ang, W. K. & Jowitt, P. W., Solution for Water Distribution Systems under Pressure-Deficient Conditions. *Journal of Water Resources Planning and Management*, **132(3)**, pp. 175–182, 2006.

[10] Bhave, P.R., Node Flow Analysis of Water Distribution Systems. *Transportation Engineering Journal*, **117(4)**, pp. 457–467, 1981.

[11] Germanopoulos, G., A technical note on the inclusion of pressure dependent demand and leakage terms in water supply network models. *Civ. Eng. Syst.*, **2(3)**, 171–179, 1985.

[12] Wagner, J. M., Shamir, U. & Marks, D. H., Water distribution reliability: Analytical methods. *Journal of Water Resources Planning and Management*, **114(3)**, 253–275, 1988.

[13] Reddy, L. S. & Elango, K, Analysis of water distribution networks with head dependent outlets. *Civ. Eng. Syst.*, **6(3)**, 102–110, 1989.

[14] Chandapillai, J., Realistic simulation of water distribution system. *Journal of Transportation Engineering*, **117(2)**, 258–263, 1991.

[15] Jowitt, P. W.& Xu, C., Predicting pipe failure effects in water distribution networks. *Journal of Water Resources Planning and Management*, **119(1)**, 18–31, 1993.

[16] Gupta, R. & Bhave, P. R., Comparison of methods for predicting deficient-network performance. *Journal of Water Resources Planning and Management*, **122(3)**, 214–217, 1996.

[17] Rossman, L.A., *Epanet: User's Manual*. Environmental Protection Agency, EPA: Reston, Virginia, 2000.

Simulating floods in urban watersheds: hydrodynamic modelling of macro, micro-drainage and flows over streets

M. G. Miguez, F. C. B. Mascarenhas, J. H. A. Prodanoff
& L. P. C. Magalhães
Computational Hydraulics Laboratory,
Federal University of Rio de Janeiro, Brazil

Abstract

Floods in urban environments generally require a different mathematical treatment in order to achieve an adequate simulation technique. When dealing with these river catchments, it must be taken into account a set of features that distinguish such phenomena from rural basin flood flow simulations. It is quite usual that rainfall run-off spills out of the macro-drainage net, causing inundation at urban landscapes, like streets, squares, parks and so on. The hydrodynamic modelling in such situations must go towards a systemic approach. Several models available nowadays focus mainly on the macro-drainage net, evaluating only its capacity to convey flows resulting from heavy rainfalls. Usually, these models are not able to simulate the hydraulic behaviour of common urban landscapes. MODCEL is a hydraulic-hydrological distributed model that seems to apply to those situations where traditional approaches fail to simulate urban catchments flood flows. One of the major advantages of that model lies on the fact that streets may represent channels, squares and parks can be considered as water storage areas, micro-drainage communicates to macro-drainage through gullies and manholes, trying to best reproduce what happens in urban watersheds subjected to flooding. In this work it is also presented a case study regarding a sub-basin in Rio de Janeiro state, Brazil. Model calibration made use of flood maps and river gauges data. The model was also validated for another rainfall input. Finally, some flood control measures were implemented in the model in order to compare results with present situation. The model showed to be a valuable tool for flood management.
Keywords: urban environment, flood management, mathematical modelling.

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1 Introduction

Watersheds of urban rivers tend to present large inundated zones. In general, urban floods involve a great variety of flow patterns on different hydraulic structures, acting in an integrate way on a complex surface configuration, where the city arises. Considering this context, it is possible to state that it is very difficult for managers to administrate a drainage system and deal with urban flood problems in a systemic way, when practical engineering tools are not available. In this way, there are indications for the use of a mathematical model. A practical model, fulfilling the requisites to simulate urban floods should be able to represent hydrologic and hydraulic basin processes in a distributed way. The model called MODCEL is capable to do this is. Mascarenhas and Miguez [4] presented an initial version of this model, using the concept of flow cells (Zanobetti *et al.* [2]). This paper presents the evolution of MODCEL and a case study that shows it as an urban flood management tool.

2 MODCEL

MODCEL is an integrated model, which conjugates a hydrological distributed model with a hydraulic-hydrodynamic looped net flow, in a spatial three-dimensional representation. In this context, it is possible to say that MODCEL is a hydrological-hydraulic pseudo 3D-model. In fact, all mathematical relations written for this model are one-dimensional. The pseudo 3D representation is achieved by hydraulically linking vertically two different layers of flow: a superficial one, corresponding to free surface channels and flooded areas; and a subterranean one, related to free surface or drowned flow in galleries

2.1 Hypotheses for the mathematical model

Watershed modelling is achieved by the definition of a set of linked homogeneous compartments, called flow cells, which can be posed together in order to provide a topographic and hydraulic spatial representation. The proposed model performs an articulation of the cells in a looped form, allowing discharges to occur in different directions in a flow net over the modelled watershed. In a cell, free surface profile and water storage are assumed to be directly related to the water level at the cell centre: In this context, mass conservation law is applied to all cells and each of them receives a precipitation contribution and performs rainfall-runoff transformation. Cells are then arranged in a topological scheme, constituted by formal groups, where a cell of a particular group can only be linked either to cells in the same group or to cells of the preceding and/or the following groups. This allows the use of the double sweep method as a numerical solution. Flow between cells can be evaluated through known hydraulic laws, such as Saint-Venant dynamic equation, in its complete form or in a simplified one, the flow over free or submerged weirs equation, the flow equation through orifices, the flow equations through gullies, among others. Discharge between two adjacent cells is considered to be only function of the water levels at their centres, in any time.

2.2 Urban watershed modelling through MODCEL representation

The cells, solely as units or taken in pre-arranged sets, are capable to represent the watershed scenery, composing more complex structures. On the other hand, the definition of a set of varied flow type links, which represent different hydraulic laws, allows the simulation of several flow patterns that can occur in an urban landscape. Therefore, the task related to the topographic and hydraulic modelling depends on a pre-defined set of cell types and the set of possible links between cells.

The pre-defined set of cell types considered in MODCEL is listed below:

- River or channel cells – this kind of cells is used to model the main free open channel drainage flow, in which the cross section is taken as rectangular and may be simple or compound;
- Underground gallery cells – act as complements to the drainage net;
- Urbanized superficial cells – are used to represent free surface flow over urban floodplains, as well as for storage areas linked to each other by streets. These cells present a gradation level degree for the compound rectangular cross section, assuming a certain pre-defined urbanizing pattern. Superficial cells can also represent slope areas, with little storage capacity or they can simulate a broad crested weir to the spilled waters from a river to its neighbour streets.
- Natural superficial cells – these cells are similar to the preceding case, however having prismatic shape without accepting any kind of urbanization.
- Reservoir cells – used to simulate water storage in a temporary pond or reservoir, which presents a curve for the elevation versus surface area. The reservoir type cell plays the role of damping an inflow discharge.

2.3 Hydrological model

Rainfall-runoff separation was originally represented in MODCEL (Mascarenhas and Miguez [1]) by applying a runoff coefficient, considered according each cell land use characteristics. Thus, for a given time interval, the effective rainfall in any cell could be obtained multiplying its runoff coefficient by the rainfall occurred in that time interval.

Aiming the improvement of the hydrological model capability regarding flood generation in the flow cell model, it was developed a simple hydrological model to represent infiltration, vegetal interception and depressions retention, being the two latter considered in a combined way in an abstraction parcel. Abstraction losses occur until this reservoir gets full. On the other hand, the infiltration can occur as long as there is water accumulated over the surface of the modelled cell. At every time step, the calculations related to the hydrologic model routines are performed and then routing is done through the hydrodynamic routines.

2.4 Hydrodynamic model

The hydrodynamic model uses the conservation mass law and hydraulic and hydrodynamics relations as the core engine of MODCEL. The water level variation in a cell i, at a time interval t, is given by the continuity equation as stated in eqn (1).

$$A_{S_i} \frac{dZ_i}{dt} = P_i + \sum_k Q_{i,k} \tag{1}$$

where:
$Q_{i,k}$ is discharge between neighbours cells i and k; Z_i is the water surface level at the centre of the cell i; A_{S_i} is the water surface area for the cell i; P_i is the discharge related to the rainfall over the cell; and t is an independent variable related to time.

2.4.1 River/channel link

This type of link is related to river and channel flows. It may eventually also be applied to flow on the streets. More specifically, it corresponds to the free surface flow represented by the Saint-Venant dynamic equation. Here it is considered that the flow velocity time variation is much larger than the spatial one, in such a way that the velocity derivative with respect to the longitudinal distance can be neglected. Eqn (2) results from the consideration of rectangular cross section and fixed bottom (Cunge *et al.* [3]).

$$\frac{1}{A_{i,k}} \frac{\partial Q_{i,k}}{\partial t} - \frac{QB_{i,k}}{A_{i,k}^{\ 2}} \frac{\partial Z}{\partial t} + g \frac{\partial Z}{\partial x} + gS_f = 0 \tag{2}$$

where:
$B_{i,k}$ is the surface flow width between cells i and k; $A_{i,k}$ is the wetted flow cross-section area between cells i and k; S_f is the energy line slope; $R_{i,k}$ is the hydraulic radius of the flow cross-section between cells i and k; n is Manning's roughness coefficient; and x, t are independent space and time variables.

The parameters n, $A_{i,k}$ and $R_{i,k}$, representative of the flow section between cells i and k, are evaluated through a weighting procedure between the water levels of cells i and k, here assigned as Z_p. Approximating the derivatives by finite differences and doing $S_f = \frac{Q_{i,k}^2 n^2}{A_{i,k}^2 R_{i,k}^{4/3}}$, one has:

$$\begin{aligned} &\frac{1}{A_{i,k}^t} \cdot \left[\frac{Q_{i,k}^t - Q_{i,k}^{t-1}}{\Delta t} \right] - \frac{Q_{i,k}^t . B_{i,k}^t}{\left[A_{i,k}^t\right]^2} \cdot \left[\frac{Z_p^t - Z_p^{t-1}}{\Delta t} \right] + \\ &+ g \frac{\left(-\left|Z_i^t - Z_k^t\right|\right)}{\Delta x} + \frac{gn^2}{\left[A_{i,k}^t\right]^2 \left[R_{i,k}^t\right]^{4/3}} \cdot \left[Q_{i,k}^t\right]^2 = 0 \end{aligned} \tag{3}$$

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It must be stressed that the only unknown term in equation (3) is the discharge $Q_{i,k}^{t}$, which, after evaluated, will be introduced into the mass conservation equation written for Z_i^{t+1}. The value of Z_i^t and the values at *t-1* are all already known. Then multiplying equation (3) by the cross-section area between cells and developing to turn explicit the unknown discharge at time t, eqn (4) is obtained

$$\left(\frac{gn^2}{A_{i,k}^t\left[R_{i,k}^t\right]^{4/3}}\right).Q_{i,k}^{t}{}^2+\left(\frac{1}{\Delta t}-\frac{B_{i,k}^t}{A_{i,k}^t}.\frac{\Delta Z_p^{t,t-1}}{\Delta t}\right).Q_{i,k}^t+\left(gA_{i,k}^t\frac{\left(-\left|Z_i^t-Z_k^t\right|\right)}{\Delta x}-\frac{Q_{i,k}^{t-1}}{\Delta t}\right)=0 \quad (4)$$

This is a second-degree equation, which can give two real roots, and needs a complementary analysis in order to characterize the result. Considering the relations shown in eqn (5), eqn (6) is written.

$$a=\frac{gn^2}{A_{i,k}^t\left[R_{i,k}^t\right]^{4/3}}\ ,\ b=\frac{1}{\Delta t}-\frac{B_{i,k}^t}{A_{i,k}^t}.\frac{\Delta Z_p^{t,t-1}}{\Delta t}\ ,\ c=gA_{i,k}^t\frac{\left(-\left|Z_i^t-Z_k^t\right|\right)}{\Delta x}-\frac{Q_{i,k}^{t-1}}{\Delta t} \quad (5)$$

$$a.Q_{i,k}^{t}{}^2+b.Q_{i,k}^t+c=0 \quad (6)$$

Eqn (6) is a typical second-degree equation, which solution is given by $Q_{i,k}^t=\frac{-b\pm\sqrt{b^2-4.a.c}}{2.a}$. It is needed an additional analysis of eqn (6) terms to verify the occurrence of real roots. Term “a” is always positive, because it depends only on geometrical cross-section elements, on Manning coefficient and on gravity acceleration. Term “b” is also always positive as $b=\frac{1}{\Delta t}\left(\frac{A_{i,k}^t-B_{i,k}^t\Delta Z_p^{t,t-1}}{A_{i,k}^t}\right)>0$, because $A_{i,k}^t=B_{i,k}^t\left(Z_p^t-Z_{0p}^t\right)>B_{i,k}^t\Delta Z_p^{t,t-1}$. Term “c” may be positive or negative, depending on the situation. A value negative for “c”, being “a” positive, allows real roots results; “c” positive, however, can or cannot give real roots. Therefore one must have $c<0$, assuring that $\sqrt{b^2-4.a.c}>b$. Then the discharge is given by $Q_{i,k}^t=\frac{-b+\sqrt{b^2-4.a.c}}{2.a}$.

Generally, for all type of links and for mass balance purposes, by convention, the discharge entering into a cell is considered positive and the one that leaves the cell is taken with a negative sign.

2.4.2 Surface link

This type of link corresponds to the free surface flow without inertia terms, being frequent its use between superficial urbanised or natural cells. This link is used to represent flow over streets, joining inundated areas. The dynamic Saint

Venant equation is then reduced to a simplified form, which developed in discrete terms and re-arranging leads to eqn (7).

$$Q_{i,k} = \left(\frac{A_{i,k} R_{i,k}^{2/3}}{n \Delta x^{1/2}} \right) \left(\left| Z_k - Z_i \right| \right)^{1/2} \tag{7}$$

2.4.3 Gallery link

This kind of link represents free surface flow in closed conduits, as well as under pressure flow conditions for drainage galleries, after they became drowned. Free surface flow is modelled in this case exactly as it is in surface links, using simplified Saint Venant dynamic equation. On the other hand, when galleries become full, pressure flow conditions are given by energy conservation law considerations. MODCEL considers the gallery linked by gullies to a street above it. In this situation, flow over the street occurs with free surface and the water level associated to this flow can be considered equal to the pressure flow line level for the gallery-drowned flow. Thus, using Bernoulli equation:

$$Z_i + \frac{v_i^2}{2g} = Z_k + \frac{v_k^2}{2g} + S_f \Delta x \tag{8}$$

It will be assumed that, by hypothesis, for any given time step, the discharge, between cells i and k, remains constant, in such a way that $Q_i = Q_k = Q_{i,k}$. On the other hand, the value for S_f can be obtained from the Manning equation, as suggested by Hromadka II *et al.* [4]. Thus, these considerations lead to eqn (9).

$$Q_{i,k} = -\left[\frac{2g(Z_k - Z_i)}{\frac{1}{A_i^2} - \frac{1}{A_k^2} - \frac{2gn^2 \Delta x}{A_{i,k}^2 R_{i,k}^{4/3}}} \right]^{1/2} \tag{9}$$

Equation (9) was developed considering cell i in the upstream position. An important model feature to be stressed is that if the cell i is located upstream, drowned, and the downstream cell k, also drowned, has a water elevation at the surface cell, Z_k, greater than the elevation Z_i, the flow may be forced gallery upwards. It is also important to detail how the model manages the flow transition from free surface to under pressure flow. At a gallery stretch, while it does not become drowned, open channel flow equations apply. When the water level evaluated by those equations indicates a value greater then that referred to the top of the gallery, the calculated exceeding water is returned to the surface cell, through the associated gully link. From this moment on, until the gallery is drowned, the relationships developed for under pressure flow become to be valid. It must be also noticed that from the drowning of a cell gallery the gullies related to it stop to contribute with inflow. Actually, once the mass balance is done, if a drowned stretch receives more water from upstream than that released to downstream, this discharge difference is sent to the surface, through the modelled gullies. At this time all happens as the gully works as a spout.

2.4.4 Inlet gallery link

This link acts in a variable way, depending on computed flow conditions at every time step. If there is a free surface flow at the entrance of the gallery, this link acts as a channel link, with a local head loss, associated to the contraction of flow, when passing from a channel to a gallery. If the entrance is drowned, then Bernoulli equation is used, but still considering the possible occurrence of local head losses. In this case, the gallery entrance may be drowned by downstream effects or by local lack of conveyance capacity for the open channel flow that arrives to the gallery. Both situations are treated by this link in MODCEL.

2.4.5 Outlet gallery link

This link is analogous to the previous one, but this time the model deals with a possible expansion of flow leaving the gallery and returning to an open channel.

2.4.6 Gallery discharge into an open channel link

This link allows a gallery to discharge into an open channel in a condition different from a junction. In MODCEL, junctions are treated as special cells with "Y" shape, where mass balance equilibrates inflows and outflows. This gallery discharge into an open channel link is associated to galleries that arrive at river bank at a level higher than that of the river bottom, acting as free broad crested weirs, drowned weirs, or orifices, depending on water level in the channel.

2.4.7 Broad crested weir link

This link represents the flow over broad-crested weirs. It is used, mainly, to represent the flow between a river and its margins. The classic formula of flow over broad-crested weirs is used here; however, the coefficients of discharge must be adjusted.

2.4.8 Orifice link

This link represents the classic formula for flow through orifices.

2.4.9 Gully link

This link promotes the interface between surface and gallery cells. When not drowned, this link acts as a weir conveying flow from streets to galleries. This weir has the length of the perimeter of the gully multiplied by the number of gullies along the street modelled by the considered cell. When drowned, this link considers flow occurring through a certain number of orifices associated to the gullies in the street.

2.4.10 Reservoir link

This link combines an orifice, as the outlet discharge of a reservoir, with a weir, that can enter or not in charge, depending on reservoir operation. It is useful to simulate the damping effect of a reservoir, in the design condition, and to verify reservoir functioning in more severe conditions (those in which the weir begin to be used).

2.4.11 Stage-discharge curve link

This link corresponds to special structures calibrated at physically reduced scales in laboratory and basically relates a discharge with a water level, in a particular equation.

2.4.12 Pumping link

This link allows discharges pumped from a cell to another departing from a starting pre-defined operation level.

2.4.13 Flap gate link

This link simulates flows occurring in the direction allowed by the flap gate opening, and can be found, normally, in regions protected by polders.

Figure 1: Aerial view of the Alberto de Oliveira Polder region.

3 Case study

Case study presented here refers to Alberto de Oliveira Polder, in Rio de Janeiro State, Brazil, between Duque de Caxias and Sao Joao de Meriti Cities. This polder was implemented at the right margin of Sarapuí River, in order to prevent this river to inundate local urban areas. Alberto de Oliveira Polder was adequately designed and solved the flood problem by the time of its implementation. Nowadays flood problem, however, is related to an irregular occupation of the storage area. A non-planned process of urbanisation limited the storage area to 20% of its original size. This situation made local waters to become as hazardous as the prior main river flooding that lead to the polder solution. The case study presented in this paper makes an assessment of present situation, using MODCEL, and proposes an alternative to minimise the problem. Figure 1 shows an aerial view of the polder region, detailing the reduction of the storage area.

The modelled region comprises a complex hydraulic system. In the upstream reach of the basin, there is a reservoir, conformed by Gericino dam, which regulates discharges of this reach. Downstream, Sarapuí River is submitted to tidal influence. There are numerous rivers and brooks. Great part of the basin is urbanised, with micro and macro-drainage interacting. The area of greater significance in this case study is protected by a polder system, but there are various flooded areas. All this diversity should be captured by model representation. Figure 2 shows a detailed sketch of the urban cell modelled area. After model calibration and validation it was produced a flooding map, for a 20 years recurrence rainfall, at polder area, combined with a 10 years recurrence rainfall, at Sarapuí river basin, as shown in figure 3. Taking into account this flooding situation, a set of proposed solutions for the region included the protection of the remaining storage area, transforming it in an environment protective area or turning it part of a new park to be built. This option should be accomplished by an increase in the number of flap gates. Complementarily, two alternative design concepts were suggested. One of them proposed a pumping station. The other, showed the need to recuperate part of the polder lost storage capacity, removing a community of a neighbourhood installed in the storage area and using a set o seven soccer fields as multifunctional landscapes. Both solutions lead to similar results in minimising floods. The inundation reduction obtained for pumping option design, considering the water level at the temporary storage area as reference, was about 52% at the cell # 881 for instance, what was sufficient to maintain urban areas protected from inundation.

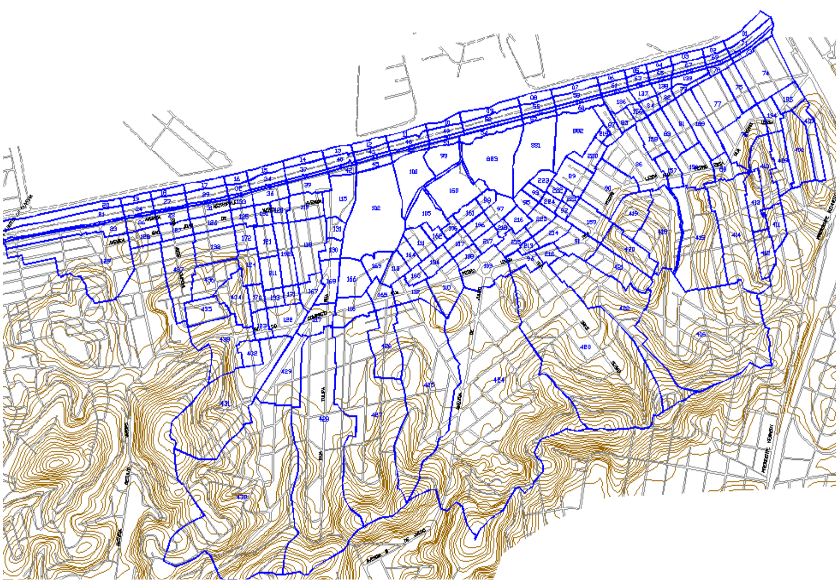

Figure 2: Detailed sketch of the division of the urban area modelled by cells.

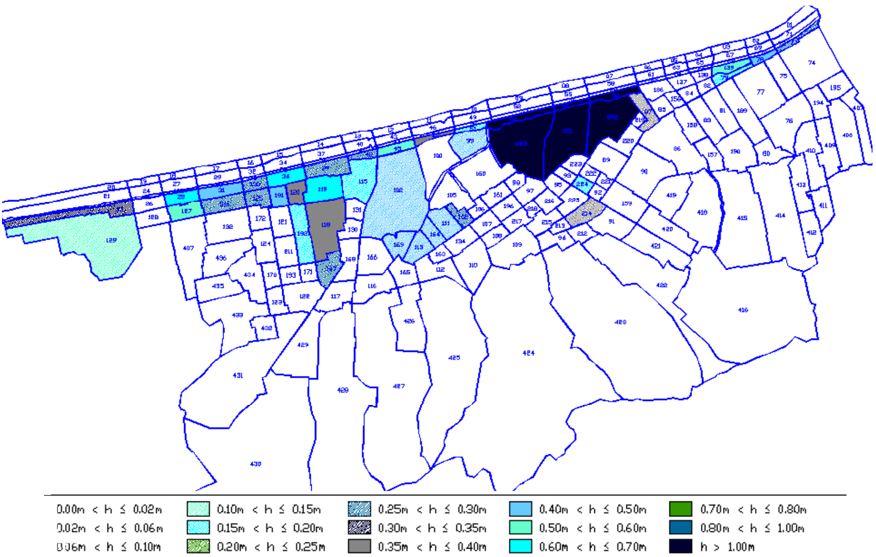

Figure 3: Inundation map at polder area.

4 Conclusion

MODCEL showed to be able to reproduce a great variety of hydraulic patterns in an urban landscape. Its ability to perform rainfall-runoff transformation, in a distributed way, allowed integrating hydrologic and hydraulic processes. The modelling of Sarapuí River basin, where it is located Alberto de Oliveira Polder, in Rio de Janeiro State, Brazil, contained the representation of a mixed rural/urban occupation, the operation of an upstream dam, a tidal influence downstream, and the operation of hydraulic structures, like flap gates and pumping stations, everything integrated in the same model. As MODCEL has a set of integrated modules, it is possible to continue to refine its capacity to represent hydraulic features by increasing types of links and cells used. MODCEL seems to be a useful tool for managing drainage systems, especially where severe flood problems occur, with large inundated areas.

References

[1] Mascarenhas, F.C.B.; Miguez, M.G.; 2002. *Urban Flood Control through a Mathematical Cell*. In: Water International, Vol. 27, Nº 2, pg. 208-218, June 2002; Illinois, E.U.A.
[2] Cunge, J.A.; Holly Jr., F.M.; Verwey, A. . *Practical Aspects of Computational River Hydraulics*. London, Pitman Advanced Publishing Program, 1980.
[3] Hromadka Ii, T.V.; Clements, J.M.; Saluja, H. . *Computer Methods in Urban Watershed Hydraulics*. Mission Viejo, California, Lighthouse Publications, 1984.
[4] Zanobetti, D.; Lorgeré, H.; Preissman, A.; Cunge, J.A. *Mekong Delta Mathematical Program Construction*. Journal of the Waterways and Harbours Division, ASCE, v.96, n.WW2, p. 181-199, 1970.

Section 11
Decision support systems

Using a neural network to build a hydrologic model of the Big Thompson River

W. S. Huffman[1] & A. K. Mazouz[2]
[1]*Nova South-eastern University, Davies, Florida, USA*
[2]*Florida Atlantic University, USA*

Abstract

Methods of modeling the hydrologic process range from human observers to sophisticated surveys and statistical analysis of climatic data. In the last few years, researchers have applied computer programs called Neural Networks or Artificial Neural Networks to a variety of uses ranging from medical to financial. The purpose of the study was to demonstrate that Neural Networks can be successfully applied to hydrologic modeling.

The river system chosen for the research was the Big Thompson River, located in North-central Colorado, United States of America. The Big Thompson River is a snow melt controlled river that runs through a steep, narrow canyon. In 1976, the canyon was the site of a devastating flood that killed 145 people and resulted in millions of dollars of damage.

Using publicly available climatic and stream flow data and a Ward Systems Neural Network, the study resulted in prediction accuracy of greater than 97% in +/-100 cubic feet per minute range. The average error of the predictions was less than 16 cubic feet per minute.

To further validate the model's predictive capability, a multiple regression analysis was done by Dr. A. Kader Mazouz on the same data. The Neural Network's predictions exceeded those of the multiple regression analysis by significant margins in all measurement criteria.
Keywords: flood forecasting, neural networks, hydrologic modelling, rainfall/ runoff, hydrology, modelling, artificial neural networks.

1 Introduction

One of the major problems in flood disaster response is that floodplain data are out of date almost as soon as the surveyors have put away their transits.

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What is needed in flood forecasting is a model that can be continuously updated without the costly and laborious resurveying remodeling that is the norm in floodplain delineation.

Current models that rely on linear regression require extensive data cleaning and re-computation, which is time and data intensive. A new model must be created every time there is a change in the river basin. The process is time, labor, and, data intensive; and, as a result, it is extremely costly. What is needed is a method or model that will do all of the calculations quickly, accurately, using data that requires minimal cleaning, and at a minimal cost. The new model should also be self-updating to take into account all of the changes occurring in the river basin.

With a NN program, a watershed and its associated floodplains can be updated constantly using historical data and real-time data collection from existing and future rain gauges, flow meters, and depth gauges. The constant updating will result in floodplain maps that are more current and accurate at all times.

Another problem with published floodplains is that they depict only the 100-year flood. This flood has a 1% probability of happening in any given year. While this is useful for general purposes, it may not be satisfactory for a business or a community that is planning to build a medical facility for non-ambulatory patients. For a facility of this nature, a flood probability of 0.1% may not be acceptable. The opposite situation is true for the planning of a green belt, golf course, or athletic fields. In this situation, a flood probability of 10% may be perfectly acceptable.

This paper is an effort to demonstrate the potential use, by a layperson, of a commercially available NN to create a model that will predict stream flow and probability of flooding in a specific area. To validate this model a comparison was made between a NN model and a multiple-linear regression model

2 Literature

The term NN is used in this dissertation to represent both the NN and ANN programs.

Muller and Reinhardt [1] wrote one of the earliest books on NNs. The document provided basic explanations and focus on NN modeling. Hertz et al [2] presented an analysis of the theoretical aspects of NNs.

In recent years, a great deal of work has been done in applying NNs to water resources research.

Hjelmfelt and Wang [3] used NNs to unit hydrograph estimation. The authors concluded that there was a basis, in hydrologic fundamentals, for the use of NNs to predict the rainfall-runoff relationship.

Huffman [4] presented a paper that suggested that NNs could be applied to creating floodplains that could be constantly updated without relying on the costly and time consuming existing modeling techniques.

Wei et al [5] proposed using NNs to solve the poorly structured problems of flood predictions.

Rajurkar et al [6] tested a NN on seven river basins. They found that this approach produced reasonably satisfactory results from a variety of river basins from different geographical locations...

Kerh and Lee [7] describe their attempt at forecasting flood discharge at an unmeasured station using upstream information as an input. They discovered that the NN was superior to the Muskingum method.

Filho and dos Santos [8] applied NNs to modeling stream flow in a densely urbanized watershed.

Sahoo and Ray [9] described their application of a feed-forward back propagation and radial basis NN to forecast stream flow on a Hawaii stream prone to flash flooding.

Late in this study, a paper by Hsu et al [10] was discovered demonstrating that results were dramatically improved by adding the previous day's stream flow or stage level input with the other data. This technique was applied in this study. This application resulted in a dramatic improvement of the predictive capability of the model.

3 Methodology

Current methods of stream-flow modeling are based on in-depth studies of the river basin including (a) geologic studies, (b) topographic studies, (c) ground cover, (d) forestation, and (e) hydrologic analysis. All of these are time and capital intensive.

Nine independent and one dependent variables were considered, and two test bed data sets are used, the Drake and Loveland data sets.

The Drake measuring station is described as, "USGS 06738000 Big Thompson R at mouth of canyon, NR Drake, CO." USGS [11]. Its location is: Latitude 40°25'18", Longitude 105°13'34" NAD27, Larimer County, Colorado, Hydrologic Unit 10190006. The Drake measuring station has a drainage area of 305 square miles and the Datum of gauge is 5,305.47 feet above sea level.

The Loveland measuring station is described as USGS06741510 Big Thompson River at Loveland, CO. USGS [12]. Its location is Latitude 40°22'43", Longitude 105°03'38" NAD27, Larimer County, Colorado, Hydrologic Unit 10190006. Its drainage area is 535 square miles and is located 4,906.00 feet above sea level. The records for both sites are maintained by the USGS Colorado Water Science Center USGS [11, 12]. The following data was used in this model:

Tmax is the maximum measured temperature at the gauging site.
Tmin is the lowest measured temperature at the gauging site.
Tobs is the current temperature at the gauging site.
Tmean is the average temperature during the 24-hour measuring period
Cdd are the Cooling Degree Days, an index of relative coldness.
Hdd are the Heating Degree Days, an index of relative warmth.
Prcp is the measured rainfall during the 24-hour measuring period.
Snow1 is the measured snowfall during the 24-hour measuring period.

Snwd is the measured depth of the snow at the measuring site.

The output variable is the predicted flow level.

This is the actual data collected by the meteorological stations. The samples for each site are more than 3000 data sets which are more than enough to (a) run, (b) test, and (c) to validate a Neural Network. For the same data, a linear regression model using SPSS was run. The same variables dependent and independent were considered (Mazouz [13]).

The Ward Systems product, selected for the research, is the NeuralShell Predictor, Rel. 2.0, © 2000. The following description was taken directly from the Ward Systems website, www.wardsystems.com (Ward Systems Group [14]).

The methods of statistical validation to be used in this paper are as follows: R-Squared, Average Error, and Percent in Range.

4 Analysis and presentation of findings

The following is a topographic map of the Big Thompson canyon. It is a narrow, relatively steep canyon.

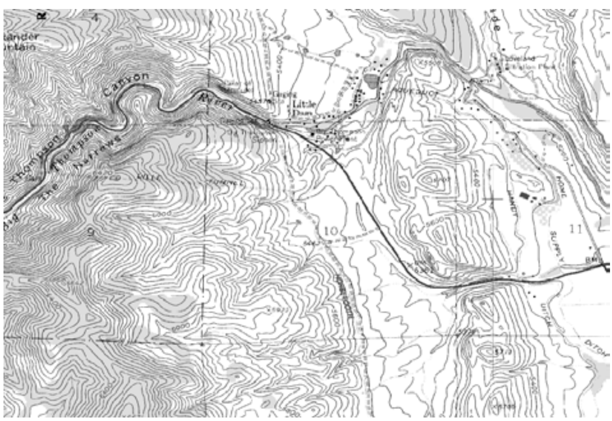

Figure 1: Topography of the Big Thompson Canyon (USGS [15]).

The historical measurements of (a) precipitation, (b) snowmelt, (c) temperature, and (d) stream discharge are available for the Big Thompson Watershed as they are usually available for most watersheds throughout the world. This is in contrast to data on (a) soil characteristics, (b) initial soil moisture, (c) land use, (d) infiltration, and (e) groundwater characteristics that are usually scarce and limited.

For this study, six climatic observation stations were used for the input variables. For the purposes of building a model to demonstrate the feasibility of using the commercially available NN, all six stations' data were used for the independent variables. The description and locations of the stations are as follows:

Coopid.	Station Name	Ctry.	State	County	Climate Div.	Lat./Long.	Elevation
051060	Buckhorn Mtn 1E	U.S.	CO	Larimer	04	40:37/ -105:18	2255.5
052759	Estes Park	U.S.	CO	Larimer	04	40:23/-105:29	2279.9
052761	Estes Park 1 SSE	U.S.	CO	Larimer	04	40:22/-105:31	2372.9
054135	Hourglass Res.	U.S.	CO	Larimer	04	40:35/-105:38	2901.7
055236	Loveland 2N	U.S.	CO	Larimer	04	40:24 /-105:07	1536.2
058839	Waterdale	U.S.	CO	Larimer	04	40:26/ -105:13	1594.1

NCDC [14]

The period of time for the historical data selected was from July 4, 1990, through May 7, 1998, a total of seven years, ten months and three days.

One extreme event occurred during this time period that was well out of the range of data available and was not adequately predicted by this NN. It is well known that a NN cannot predict an event that it has never seen before in the training data. There was no repeat of the magnitude of this event during the time period under study.

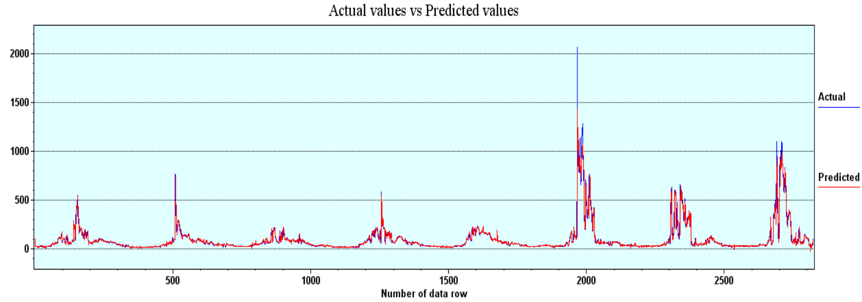

Figure 2: Drake final model, actual versus predicted.

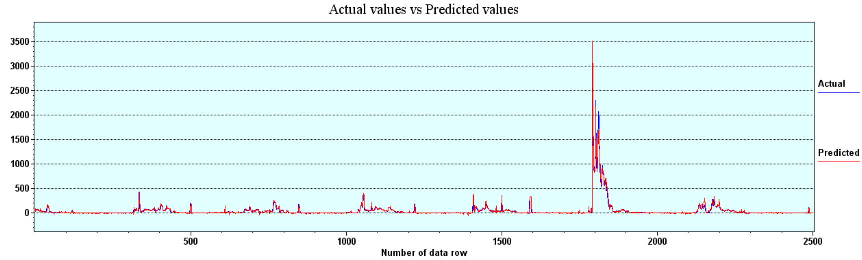

Figure 3: Loveland, final model, actual versus predicted.

The following chart depicts the statistics from the model:

Statistical analysis of the Neural Network Model, Big Thompson River, Co.						
	R-squared	Av. Error	Corrilation	MSE	RMSE	% in Range
Station						
Drake	0.9091	15.24	0.9534	1993.011	44.64	97.3
Loveland	0.9671	11.56	0.9834	1016.943	31.89	98.1

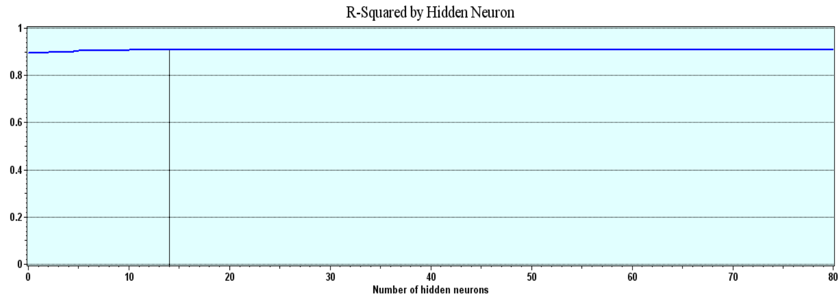

Figure 4: Drake, final model, R2.

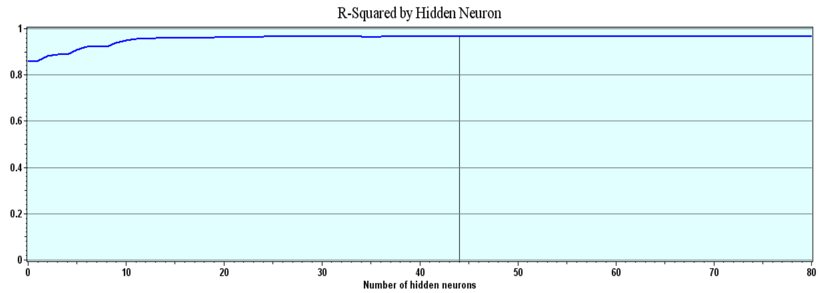

Figure 5: Loveland, final model, R2.

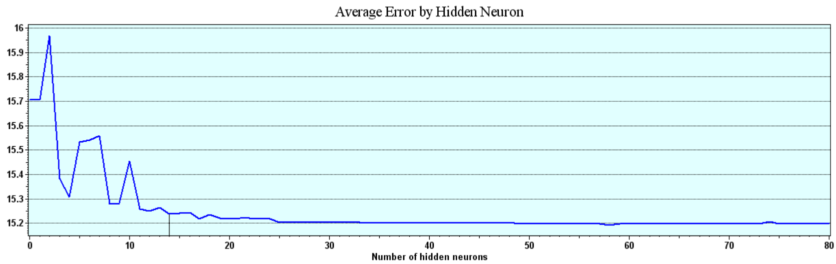

Figure 6: Drake, final model, average error.

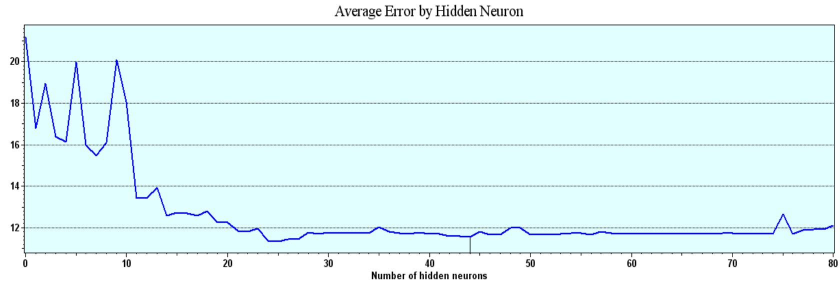

Figure 7: Loveland, final model, average error.

The R^2 results for the Drake and Loveland were 0.9091 and 0.9671.

The average errors for the Drake and Loveland are 15.7 cfm and 11.56 cfm.

The correlation values for both the Drake and the Loveland measuring station for this model are very good at 0.9534 and 0.9834.

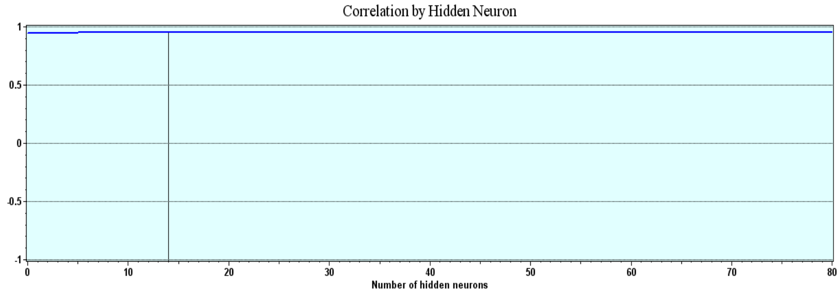

Figure 8: Drake, final model, correlation.

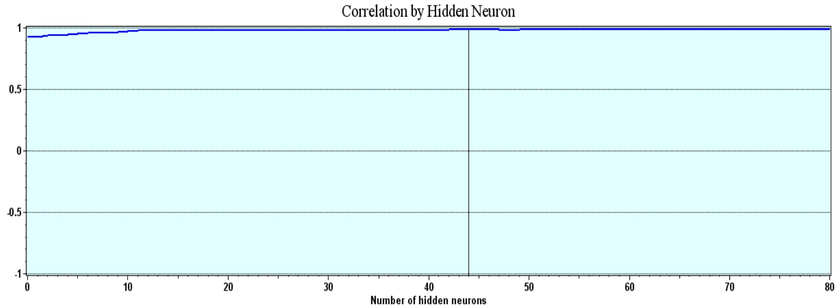

Figure 9: Loveland, final model, correlation.

The Drake and Loveland measuring stations' percent in range ended the run at values of 98.1 and 97.3.

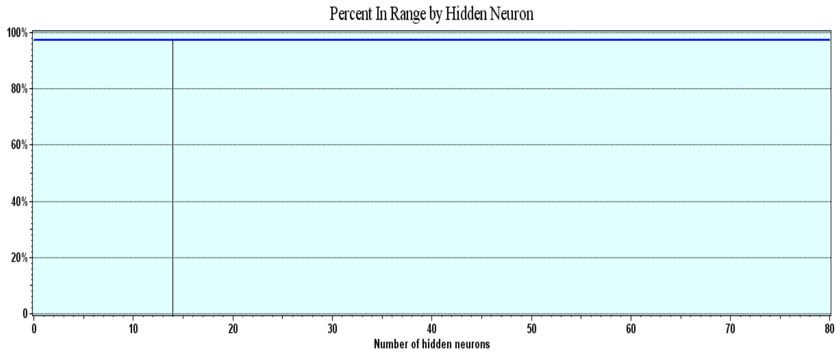

Figure 10: Drake, final model, percent in range.

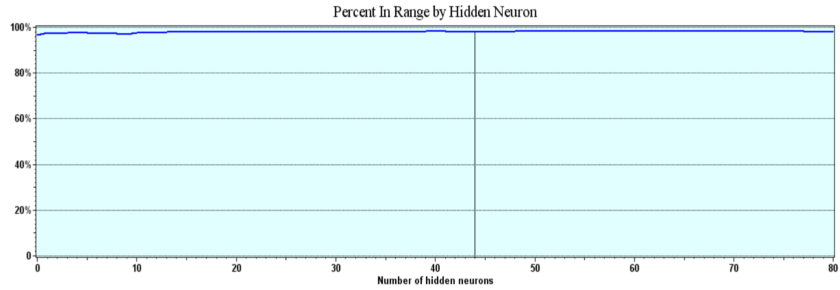

Figure 11: Loveland, final model, percent in range.

The following multi linear regression models were created and provided by Dr. Kadar Mazouz of Florida Atlantic University Mazouz [15].

A stepwise multi-linear regression model was generated for both data sets, Drake and Loveland. Being a multiphase process, it stopped after the seventh model. It gave an R-square of 0.0849, which is less than the NN Model generated for the Drake Data sets.

Actually, the independent variables that contributed the most to the variability of the output are Preflow, TOBS3, Prcp2, Prcp4, Prcp5, and Snow5. Their corresponding coefficients in the model are:

Drake, Model Summary (i)

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.916(a)	0.838	0.838	29.0311
2	0.918(b)	0.842	0.842	28.7095
3	0.918(c)	0.843	0.843	28.6109
4	0.919(d)	0.845	0.845	28.4290
5	0.920(e)	0.846	0.845	28.3747
6	0.920(f)	0.847	0.846	28.3096
7	0.921(g)	0.849	0.848	28.1527

(a) Predictors: (Constant), Preflow
(b) Predictors: (Constant), Preflow, Tobs3
(c) Predictors: (Constant), Preflow, Tobs3, Prcp2
(d) Predictors: (Constant), Preflow, Tobs3, Prcp2, Prcp4
(e) Predictors: (Constant), Preflow, Tobs3, Prcp2, Prcp4, Tmax3
(f) Predictors: (Constant), Preflow, Tobs3, Prcp2, Prcp4, Tmax3, Prcp5
(g) Predictors: (Constant), Preflow, Tobs3, Prcp2, Prcp4, Tmax3, Prcp5, Snow5
(h) Dependent Variable: OUTPUT

The stepwise multi-linear regression model for Drake was generated in eight iterations. It gave an R-square of 0.80, which is less than the R-square generated for the Loveland data using NNs.

The independent variables that contributed the most to the variability of the output are Preflow, OFestes, Prcp3, PrcpA, Tmin3, Snow A, Tobs A. Their corresponding coefficients in the model are:

Loveland, Summary (i)

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	0.881(a)	0.777	0.776	22.06113
2	0.891(b)	0.793	0.793	21.23824
3	0.892(c)	0.796	0.796	21.09537
4	0.893(d)	0.798	0.797	21.01388
5	0.894(e)	0.799	0.798	20.95246
6	0.895(f)	0.801	0.800	20.85033
7	0.896(g)	0.803	0.801	20.80284
8	0.896(h)	0.803	0.802	20.76851

(a) Predictors: (Constant), Preflow
(b) Predictors: (Constant), Preflow, OFEstes
(c) Predictors: (Constant), Preflow, OFEstes, Prcp3
(d) Predictors: (Constant), Preflow, OFEstes, Prcp3, Prcp_A
(e) Predictors: (Constant), Preflow, OFEstes, Prcp3, Prcp_A, Tmin3
(f) Predictors: (Constant), Preflow, OFEstes, Prcp3, Prcp_A, Tmin3, Tmax1
(g) Predictors: (Constant), Preflow, OFEstes, Prcp3, Prcp_A, Tmin3, Tmax1, Snow_A
(h) Predictors: (Constant), Preflow, OFEstes, Prcp3, Prcp_A, Tmin3, Tmax1, Snow_A, Tobs_A
(i) Dependent Variable: LvdFlow

5 Summary and conclusions

In developing this model, a daily rainfall-runoff model for two flow-measuring stations, Drake and Loveland, on the Big Thompson River in Colorado, was developed using the NeuralShell Predictor. The following topics were addressed: (a) the use of a commercially available NN in the development of the daily rainfall, snowmelt, temperature-runoff process model; (b) the evaluation of the reliability of future predictions for this NN model; and (c) the comparison of results of the to a Linear Multiple Regression model developed by Mazouz [15].

For the Big Thompson River, the NN model provides better than 97 percent prediction accuracy within a plus or minus 100 cfm range. When comparing the results of the NN to those of the linear multiple regression analysis, it is apparent that the NN provides a clearly superior predictive capability.

Although the network model developed in this study can only be applied to the Big Thompson River, the guidelines in the selection of the data, training criteria, and the evaluation of the network reliability are based on statistical rules. Therefore, they are independent of the application. These guidelines can be used in any application of NNs to other rivers.

References

[1] Muller B. and Reinhardt, J., *Neural Networks, an Introduction.* Springer-Verlag: Berlin and New York, 1990.

[2] Hertz, J., Krogh, A. and Palmer, R., *Introduction to the Theory of Neural Networks*. Addison-Wesley Publishing Company, 1991.

[3] Hjelmfelt, T. A. Jr., and Wang M. *Proceedings of the Symposium on Engineering Hydrology, ASCE,* San Francisco, 1993. (unpublished)
[4] Huffman, W. S., *Geographic Information Systems, Expert Systems and Neural Networks: Disaster planning, mitigation, and recovery.* River Basin Management, pp. 311-321, 2001.
[5] Wei, Y., Xu, Y., Fan et al., Artificial neural network based predictive method for flood disaster. *Computers & Industrial Engineering,* 42(204*), pp 383-390.*
[6] Rajurkar, M. P., Kothyari, U. C., and Chaube, U. C.., Modelling of the daily rainfall-runoff relationship with artificial neural network. *Journal of Hydrology, 285(1-4), pp. 96-113.*
[7] Kerh, T., and Lee, C. S., Neural networks forecasting of flood discharge at an unmeasured station using river upstream information. *Advances in Engineering Software,* Article in Press, Corrected Proof.
[8] Filho, A. J. P., and dos Santos, C. C., Modelling a densely urbanized watershed with an artificial neural network, weather radar and telemetric data. *Journal of Hydrology, 317(1-2), pp. 31-48.*
[9] Sahoo, G. A., and Ray, C., Flow forecasting for a Hawaii stream using rating curves and neural networks. *Journal of Hydrology, 317(1-2), pp. 62-80.*
[10] Hsu, K. L., Gupta, V., and Sorooshian, S., Artificial neural network modelling of the rainfall-runoff process. *Water Resources Research,* 31, pp. 2517-2530.
[11] USGS (2006 a). USGS 06738000 Big Thompson R at mouth of Canyon, NR Drake, CO. Retrieved November 14, 2006.
[12] USGS (2006 b) USGS 06741510 Big Thompson River at Loveland, CO. Retrieved November 14, 2006.
[13] Mazouz, A. K., Multiple Regression Analysis of the Big Thompson River, 2006. (unpublished)
[14] Ward Systems Group. I., NeuroShell Predictor, Frederick, MD: Ward Systems Group, 2000.
[15] USGS & Inc, MAAC, Topographic map of the Big Thompson Canyon.

Utilization of knowledge management and information technologies theory in water resources management

M. Toman
Department of Hydrotechnics, Faculty of Civil Engineering, Czech Technical University in Prague, Czech Republic

Abstract

Project AQUINpro, described in this paper, derives from a common theory of knowledge management and information technologies. The aim it to expand the theory to the water resources management field in the Czech Republic, a theory that is not currently being applied. Methods of knowledge management allow for much more space for thinking about the substance of water management problems beyond that of the application of partial artificial intelligence methods. The way of solving of water management problems will be possible by finding an effective, elegant and especially simpler way with the use of knowledge and expert systems. Knowledge and expert systems are very effective tools for working and exploiting important knowledge about the operations and control of complicated systems. Such systems are water management systems, which have specific dynamic properties and their real-time control proceeds often in conditions of uncertainty. Expert systems are above all much preferable for operation than conventional algorithms because they are able to imitate logical thinking of the human factor. For solving problems in water management it is necessary to use, what has been until now, a nonexistent methodology. Systems of knowledge management can be suitably adapted to be helpful for extreme situations and for training purposes. It is not always possible for operatives to be involved in solving problems of extreme flood situations in real time, which did happen in the recent past in the Czech Republic. Recent experience of managing flood situations in the year 2002 has highlighted the necessity to methodically develop the generally applicable, fundamental, operational and decision making rules behind extreme situations. It is necessary to do this as soon as possible, in order not to loose recently gained valuable knowledge and experience. Knowledge technologies have now been employed in a number of different areas and the results of these projects can be used as a theoretical background for implementing of knowledge management tools to the water management companies.
Keywords: water management, knowledge and information technologies, optimization of control.

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1 Summary of contemporary knowledge state about the given problem

Utilization of artificial intelligent methods or use of intelligent systems, especially expert systems in complex elements and processes control is, in spite of some known risks or failures, very progressive, and such systems are broadly utilized in a number of areas. In other parts of the world, these problems are solved by organisations such as the IIASA institute in Vienna, and in NATO by specialized workplaces within the headquarters in Brussels that are also interested in this field. Applications of expert systems for the control of complex facilities, like water treatment plants and wastewater plants are designed and utilized in water management throughout the world. Expert systems that evaluate ecological impacts of different constructions are another type of their exploitation. As far as other fields are concerned the largest exploitation of expert systems has been in medicine (diagnostic systems), in the control of complex technological industry processes, in the energy industry, in space research (for example within the bounds of NASA) and in military applications. At the International Conferences on the Application of Artificial Intelligence (AI) in engineering disciplines – AIENG (in July 1995, in Udine, Italy; in July 1996, in Florida, USA; in July 1997, in Capri, Italy; in July 1998, in Galway, Ireland), International Water Association Conference “Management of Productivity at Water Utilities” (June 2002, Prague, Czech Republic) or at conferences River Basin Management and Water Resources Management (for example April 2003, Las Palmas, Gran Canaria) it was demonstrated on some applications, that expert system applications for the area of water management seem to be very promising.

In countries with advanced technologies, any utilization of artificial intelligence methods in water management is considered to be strategically important. Applications of computers and robotics are broadly supported as well as the development of software for operators’ decision support in complex system environments, which require large amount of information, experience and knowledge to be controlled, i.e., require knowledge and expert systems development. In our situation the adoption of these progressive methods is slow, however, a good understanding of the potential benefits for their applications exist and in some cases are being utilized, as in the River-basin enterprises where a routine has already started of automated information systems for the collection and transfer of data from river-basins to the control centres.

The first application of a classic expert system approach in water resources management was solved in the scope of the grant of the Faculty of Civil Engineering of Czech Technical University in Prague (No. 1032 and No. 2027) in 1995 and 1996. The applicable prototype of an expert system was developed in the professional development software NEXPERT Object of the Neuron Data Company. The developed knowledge-based system has been controlling the system of three reservoirs by the water resources management control based on the adaptive principle in dry periods. It had been tested using hydrologic data of the reservoirs Kružberk, Šance a Morávka, supplying the drinking water to the

Ostrava region. The second water resources management application seems to be a prototype of knowledge-based system aimed at optimal manipulations design for water reservoirs of the cascade Slezská Harta, Kružberk and Podhradí so that the maximum electrical energy production can be achieved on the cascade's small electrical power plants, based on Ostrava's region decreased demand for the drinking water supply.

Recently, the project of the Grant Agency of the Czech Republic, No. 103/01/0036 "AQUIN – Utilization of Information Technologies for Optimization Control in Water Management" has been, in the last few years, a direct continuation of the previously mentioned research initiatives. The goal of the project has been to utilize expert systems and knowledge management methods into still very rare water management applications, to achieve important results and contribute thus to their popularization throughout the professional community and consequently also to their broader practical utilization. This goal has been achieved successfully by the development of a prototype knowledge-based system for the reservoir Nýrsko, controlled by the dispatch centre of the River-basin Vltava state enterprise – branch plant Berounka.

Controlling of any process, including controlling of water management, requires knowledge, if it is supposed to be effectively applied in everyday practice. This knowledge is often only available to a small group of specialists (experts), who know when, how and what should be done to ensure a failure-free and sufficient water supply to households, or industry, or to properly manage dramatic flood threads. This knowledge may not always be available for several reasons – experts may not be always available when they are needed, their knowledge and expertise may be lost when they move away or change jobs or they may differ in opinion on how to best solve a specific problem and this may lead to disturbances and delays in solving the problem.

Knowledge can also be hiding in data. Tools for so called machine learning and data mining based on statistic methods are possible to use for getting such data.

For this reason such a solution is appropriate in the domain of water management control that is based on principles of knowledge management using suitable knowledge tools which have proven their usability in other areas.

The area of knowledge management, which is intended to be applied largely in the proposed project, utilizes systematic approaches to identification, understanding, and application of knowledge in order to create new values. It is also a formalization of various approaches based on areas such as artificial intelligence, knowledge based systems, organizational management, etc., aiming to such a treatment of experience, knowledge, or expertise of human specialists, which will lead to the building of new possibilities supporting an increase in intellectual power, introduction of innovations, and enhancing services for future users. Recently, a number of various organizations have already started to introduce at least some parts of the big palette of principles, methods, and tools of knowledge management as well as of the ontological engineering, aimed at sophisticated creation of an explicit specification of important concepts of the problem domain. That is the reason why this application area of research deserves increased attention in water management. The ultimate goal of introducing knowledge management in a company is to collect all the important knowledge (explicit or so-called tacit, that can be found mainly in experts heads) and make it available – shared through the entire company. Usually this task is accomplished using well developed knowledge technologies such as knowledge-based or expert systems.

2 The aim formulation, the schedule of the project and expected outputs, project applicants and their institutions

The goal of the project AQUINpro is an investigation of knowledge transfer, reusing and sharing possibilities in the domain of water management. The research will focus on design of knowledge models specific to control structures of water management and suggestions of appropriate models. Based on the project AQUIN experience and by enriching expert knowledge so far acquired with knowledge specific to other water systems of the River-basin Vltava state enterprise – branch plant Berounka, knowledge models will be created by using methodology of ontology engineering. These models will be used directly for developing several knowledge-based DSS for each of the water systems mentioned earlier, and it will be enable sharing and transferring knowledge contained in these systems, independently on used implementation tool. It will give a base for introducing knowledge management approaches and tools in the River-basin Vltava state enterprise – branch plant Berounka, later on.

Development of knowledge-based system application prototypes will be accomplished by using "Jess" environment for expert systems development. It is a free development environment, which is a successor of the older system CLIPS. The system CLIPS has been used in the previous project AQUIN. In Jess environment it is possible to create applications by declarative ways, i.e. to formulate knowledge in the facts and rules form. The implemented inference mechanism then manipulates with rules, it is possible to work with fuzzy logic and with object oriented programming components. In Jess system it is possible to equip the created application with a useful user interface, unlike the CLIPS system.

The project was carried out over a period of three years. In the last year (2006) the procession of basic material and data supplied by the River-basin Vltava state enterprise came through with an enrichment of acquired expert knowledge specific to further water schemes. Working from this data the selection of knowledge technologies for the next data processing (case-based reasoning, data mining, pattern matching) was accomplished and adequate software was selected.

In this year (2007) the water reservoirs problem domain knowledge models are being developed using ontological engineering principles and three corroborative decision support systems:

1. Algorithmization of the technological procedures from water structures manipulation rules.

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2. System for identification, evaluation and sampling of previous decision scenarios from which is possible to choose the decision in this exact moment.
3. Knowledge simulation of river basin saturation trend that exploits rainfall-runoff river basin model for testing of decision support scenarios.

In the year 2008 – testing, tuning a practical verifying of the knowledge management application, evaluation of achieved results, their presentation and an eventual proposal of further application of the project results will be carried out.

In the frame of the project prototypes of knowledge-based decision support systems for optimization of water structures control in partial basins of the River-basin Vltava state enterprise – branch plant Berounka (principally of those of Mže (for example see fig. 1), eventually Střela or Radbuza) will be developed on the basis of created knowledge models. These prototypes will be tested, debugged and verified in practice.

Anticipated scientific outputs of the project will be beneficial in the domain of water resources management and in the area of applied information and knowledge technologies, and they will contribute to knowledge and experience of these fields. The outputs will encourage further research efforts and will enrich university education. They will become a definite scientific contribution and they will significantly enhance the usage of information technologies in the area of water management.

The project solver - Doc. Ing. Michal Toman, PhD. - has been working eighteen years in the field of water resource systems, applied hydrology and hydraulic at the Department of hydrotechnics, Faculty of Civil Engineering, Czech Technical University (CTU) in Prague. Recently, he was the responsible solver of the grant project (Grant Agency of Czech Republic, No. 103/01/0036) “AQUIN - Utilization of information technologies for optimization control in water management”. He participates in the research in above-mentioned scientific disciplines and he solved the development of expert system for complex water resource system in the range of internal grants of Faculty of Civil Engineering of CTU (No. 1032 and 2027 in 1995 and 1996). He has experience from the research of artificial intelligence methods in the theory of water resource systems (Grant Agency of Czech Republic, No. 103/97/0106) and he is author or co-author of software equipment for relevant research tasks. He published research results as co-author with his colleagues and he presented these results at international specialized conferences (10th and 12th International Conferences on the Application of AI in engineering disciplines, in July 1995, in Udine, Italy and in July 1997, in Capri, Italy; International Water Association Conference “Management of Productivity at Water Utilities”, June 2002, Prague, Czech Republic; “River Basin Management 2003” and “Water Resources Management 2003”, April 2003, Las Palmas, Gran Canaria).

The first project joint solver - Prof. Ing. Peter Mikulecký, PhD. - is a recognized specialist in the field of the information and knowledge technologies applications (especially knowledge-based systems) and in the area of knowledge management. Professor Mikulecký is the head of the Department of Information Technologies, Faculty of Informatics and Management of the University of Hradec Králové, where he is also the Director for research, with a responsibility for the doctoral degree programme. He supervises 8 doctoral students in the programme Information and Knowledge Management. His scientific interests include decision support systems, knowledge and expert systems, knowledge management and human-computer interaction. He is foreign member of accreditation committee of the Slovak Republic government, a member of a number of Czech and international professional organizations; he has participated in many research projects (e.g. 5th Framework Programme, Socrates, Tempus, Leonardo da Vinci, etc.). He led a project of Grant Agency of Czech Republic (No. 201/99/0950, 406/04/2140), a project of INFRA programme no. LB98217 and recently is the principal researcher of a research project of the Ministry of Education MSM 184500001 "Knowledge Management for Information Society".

The second project joint solver - Ing. Jan Janda - represents the practice. He is the director of the River-basin Vltava state enterprise – branch plant Berounka in Plzeň. He is very open to ideas of introducing new ways of control into practice. He is interested in hydrology, in hydro technical constructions and in operations on rivers. He also preserves very productive contacts with research basis and his enthusiasm to be active in problem solving within his company guarantees immediate verification and consequential utilization of research results in practice.

The project solver, joint solvers and their co-workers constitute the team of specialists with considerable knowledge and experience in the given research area. Therefore the project is in safe hands from the professional and scientific point of view. Immediate practical connections are supported.

3 Conception and methodical approaches for the solution of the project and their analysis

Given current problems of water resources management, a research should be promoted, suitable methodical procedures should be sought and new scientific disciplines should be developed that join together for example hydraulics and hydrology with modern knowledge and information technologies in order to simulate the operation of water management structures in various situations and different environmental conditions and influences.

Keeping in mind the objectives of the project, we should identify the difference between important, moderately important and unnecessary types of knowledge. Important types of knowledge will gradually gain importance as the work on the project proceeds and the objectives of the project will be eventually broadened. The moderately important types of knowledge may also become important in the case of modification of the project's goals.

Knowledge sources may differ in the profundity of knowledge they offer. Some knowledge sources have a form of a professional textbook which serves theoretical background knowledge of the domain. Water scheme operating instructions are another source based on an analysis of historical data characterizing climate, river basin and the reservoir behaviour. Heuristic knowledge of dispatchers is usually considered as the most valuable type knowledge. Dispatchers gained deep operation knowledge about the behaviour of the entire river basin and water management structures along the river. The source of this type knowledge is naturally the personal of the river basin management company's decision making centre.

One of the most important criteria of the identification of potentially suitable knowledge source is its availability. Here expectations generally do not correspond to the reality and many potentially interesting knowledge sources may later on turn out as inaccessible or unrecoverable.

An important factor is also the reliability of the knowledge sources used. Many of them consist of a complete series of correctly measured data but it can not be ruled out them some part of the data is missing or got damaged during a transfer from one data medium to another. A big part of available knowledge may be uncertain or inexact. This may be the case of dispatchers' heuristic knowledge but, on the other hand, the heuristic knowledge is the most valuable knowledge that constitutes the base of the knowledge-based DSS in development. For this reason a task of extracting and representing heuristic knowledge is one of the most difficult and most important tasks. Suitable methods for resolving the mentioned tasks can be found in some of the listed references.

4 Expected results of the project AQUINpro and their utilization

Knowledge-based systems and their application in water resources management are successfully accepted in the every day practice of water management owing to focused research in the frame of AQUIN project. The proposed project is a continuation of the original research work in this area and its outputs will be of concrete benefit for every day practice. Current experiences of the River-basin Vltava state enterprise – branch plant Berounka (the second joint applicant) show that solutions reached are useful and important for the control of water management and for the updating of manipulation and operating instructions of water schemes.

Based on experience gained from previous projects the goal of the current project is to develop knowledge-based applications in the domain of water management. Using knowledge technologies the solution of a practical problem can be enhanced and widened. Using the above mentioned knowledge technologies the solution of practical problems of the River basin management company Vltava, branch Berounka, as well as potential flood risks or the solution of non-supply of water in extreme dry periods can be achieved and enhanced. In the frame of the proposed project the achieved experience will be applied also at other river sheds of Berounka (particularly river basin Mže with water schemes Hracholusky – see fig. 1 and Lučina, optionally river basin Střela with water scheme Žlutice or river basin Radbuza with water scheme České údolí).

Figure 1: Hracholusky Dam.

5 The significance of the project for water management practice

In the domain of water resources management a whole range of up-to-date problems exist that require a solution. First of all, models of optimal utilizing of water in reservoirs, rainfall-runoff forecasting models, models of river pollution propagation, models of water quality control in reservoirs etc. can be listed and currently flood and flood situations forecasting models can also be mentioned.

Knowledge-based systems to be developed and can be then used in decision making centres of the river basin companies in order to improve the quality of real-time control and decision making where special attention will be paid to specific local geomorphologic and hydrological circumstances. The significance of knowledge-based systems utilizing lies especially in the possible training of new incoming dispatchers for conditions of control and in practice the decision making processes of current staff of decision making centres. The systems may be also used in a formulation of new approaches to protection against floods at a national and international level.

Contemporary rapid expansion of technologies and production procedures came to be more complicated, as was the processes of their control. It presents a stressful problem for man and may become, that the ability to control such processes can fail. It could lead to considerable economical and ecological collapses. A reasonable way for prevention of such complications is increasing safety factor of human control activity by use of decision supporting systems from the artificial intelligence area. It means that we have to focus on the development of knowledge and expert systems and on their propagation.

Management of water resources is task that is impossible to solve independently on all existing inner and outer structures and surroundings. The resolution of their optimal exploitation is therefore using new system and methodical techniques. At the present time the research into optimal operative management of water sources and water management elements is undertaken by the individual water management dispatching centres of river-basin companies. It is a wide research area with many important practical problems, e.g. operation optimalization of check gate, efficient power usage of small hydraulic power plants, automatization of navigation elements, flows and flood forecasting and solving of their possible consequences, real - time control of complicated water management systems of reservoirs and waterways, monitoring and water quality management in reservoirs, operating of drawing mechanisms of waterways, outlets and protective spillways constructions, water transport management etc.

For all of these problems it is possible to effectively use progressive technology of knowledge and expert systems. Large perspectives of their usage is especially in an analysis of water management construction ecological impacts, they may help to estimate a progression of flood situations and design for an appropriate manner of operative intervention and as well to optimise decision-making processes in water management dispatching centres.

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References

[1] Toman, M., Mikulecký, P., Olševičová, K. & Ponce, D.: Knowledge Technologies in Water Management. CTU Publishing Company, Prague, 2004, 205 pp., ISBN 80-01-03049-0.

[2] Olševičová, K. & Ponce, D.: Knowledge analysis of the system for water reservoir control. International Conference on Hydrology: Science & Practice for the 21st century, 12-16 July 2004, London.

[3] Mikulecký, P.: On the Way to Knowledge-Based Water Management. In: The BHS 2004 Intl. Conf. on Hydrology: Science and Practice for the 21st Century. 12-16 July 2004, London.
[4] Toman, J. & Toman, M.: Water Balance of the Reservoir Nýrsko. Design Material for Operating Regulations Actualization. Prague, March 2004, 53 pp.
[5] Mikulecký, P.: A Knowledge Management Application in Decision Making. In: Proc. of the 2nd Int. Conference m-ICTE 2003, Badajoz, 3.-6.12.2003, pp. 308-311, ISBN 84-96212-10-6.
[6] Toman, M. et al.: Project AQUIN – Annual Report for a Year 2003, CTU in Prague, Faculty of Civil Engineering, November 2003, Research report of the project GAČR - no. 103/01/0036, 117 pp.
[7] Mikulecký, P., Ponce, D. & Toman, M.: A knowledge-based solution for river water resources management. In: Water Resources Management II. Southampton, WIT PRESS 2003, pp. 451-458, ISBN 1-85312-967-4.
[8] Mikulecký, P., Ponce, D. & Toman, M.: A knowledge-based decision support system for river basin management. In: River Basin Management II. Southampton, WIT PRESS 2003, pp. 177-185, ISBN 1-85312-966-6.
[9] Toman, M. et al.: Project AQUIN – Annual Report for a Year 2002, CTU in Prague, Faculty of Civil Engineering, January 2003, Research report of the project GAČR - no. 103/01/0036, 82 pp.
[10] Hynek, J. & Slabý, A.: Why Evolutionary Algorithms Fail? The Contribution at International Conference Learning '02. Universidad Carlos III de Madrid, October 23 – 25, 2002, Madrid, Spain, pp. 47 – 52.
[11] Ponce, D., Mikulecký, P. & Toman, M.: Application of Qualitative Reasoning in Hydropower Plant Control. In: International Water Association Conference "Management of Productivity at Water Utilities", 12.6. – 14.6.2002, Prague, p. 191.
[12] Mikulecký, P., Ponce, D. & Toman, M.: Possibilities for Knowledge Management Solutions in Water Management. In: International Water Association Conference "Management of Productivity at Water Utilities", 12.6. – 14.6.2002, Prague, p. 96.
[13] Toman, M., Mikulecký, P. & Ponce, D.: Knowledge-based and Expert Systems Utilization to Improve the Control and Optimization Processes in Water Management Dispatch Centers in Real Time. In: International Water Association Conference "Management of Productivity at Water Utilities", 12.6. – 14.6.2002, Prague, pp. 107 – 112.
[14] Toman, M. et al.: Project AQUIN – Annual Report for a Year 2001, CTU in Prague, Faculty of Civil Engineering, January 2002, Research report of the project GAČR - no. 103/01/0036, 63 pp.
[15] Toman, M. & Mikulecký, P.: Expert Systems and Its Perspective in Water Resources Management. Water Management, 50, 2000, No.4, pp. 72 – 74.

Section 12
Coastal and estuarial problems

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Integrated coastal zone management in the Patos Lagoon Estuary (South Brazil): state of art

P. R. A. Tagliani, M. L. Asmus, C. R. A. Tagliani, M. Polette, C. S. B. Costa & E. Salas
Department of Oceanography, Federal University of Rio Grande, Brazil

Abstract

In the context of the extensive Brazilian coastal zone, the Patos Lagoon estuary comes across as the contemporary dilemma between the economic development and the environmental conservation in a conspicuous way. Deep changes in the environmental and socioeconomic scenario are expected in the near future, due to the big projects of economic development planned for the region, such as a paper mill industrial complex, a deepening of the navigation channel of the estuary from 15 to 18 m and the implantation of a shipyard to build up petroleum marine platforms.

Taking into consideration such trends, a program of Integrated Coastal Zone Management at Patos Lagoon estuary – Costa Sul Program, has been conducted since 2005 with the purpose of promoting sustainable development in this estuary. The project is oriented to four main strategies: 1) building capacity in the local government and empowerment of the local community; 2) restoration of coastal habitat and fishery resources; 3) to provide alternatives of rental for the rural coastal communities; and 4) to develop a comprehensive coastal management plan with active participation of the civil society, integrating the federal and state environmental policies, and programs and projects with the local ones. This paper will describe some results obtained at this time and will present some lessons learned.
Keywords: coastal zone, integrated management, estuaries.

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1 Introduction

One of the biggest challenges to be faced for Latin America in this 21st century is the overcoming of the historical barriers that hinder full economic development, promoting the technological transition with social justice and ecological equilibrium. Such a challenge is particularly aggravated in the coastal zones that constitute ecozones of high biodiversity, where there is the concentration of 60 of the 77 largest cities and approximately 60% of the population of 475 million people (Hinrichsen, In: IDB, [1]), becoming a focus of conflicting interests related to the use and the conservation of the coastal resources. The fragmentation of institutions, public policies, and human/financial resources, is pointed out by some authors as the main difficulty with respect to an effective Integrated Coastal Zone Management (ICZM), and consequently lead to a maintenance and/or aggravation of the conflicts of uses. Such is the situation of the Patos Lagoon estuary, south Brazilian coastal zone (between the parallels 31° 47’ 0 and 32° 39’ 45’’ S and meridians 52° 03’ 10’’ / 52° 44’ 10’’ W) where the worldwide trend of reduction in the fisheries productivity [FAO, 2], and the absence of income alternatives in the fishing communities, produce a vicious circle of reduction of the fishery resources, and increase of the poverty in such communities. On the other hand, the peripheral urban communities have suffered the consequences of uncontrolled industrial and urban development during the last three decades, what has been reflected in environmental deterioration, as well as reduction of their quality of life [3–5].

The region of the low Estuary of the Patos Lagoon is shared by the cities of São José do Norte, with 24,877 inhabitants and Rio Grande, with 193,789 thousand inhabitants, occupying the 6th position in the economic ranking of the Rio Grande do Sul state. This economic development, however did not cope with social development, as testified by the Human Development Index of the city that is relatively low in comparison with other cities of the state (Tagliani et al. [4]); The main stakeholder in the local economy has been the port of Rio Grande whose importance will tend to increase in the near future due to a number of investments that are being applied in infrastructure. The entrance of new stakeholders in the areas of forestry and production of cellulose allows one to foresee the deepening of the transformations that are operating in the economic, territorial and environmental scenario at a regional level.

Aiming to promote sustainable development in the low estuary of the Patos Lagoon, a Plan of Integrated Management in the region – Costa Sul Program – has been implemented since 2005 with the technical support of the local university, with the partnership of public and private institutions, as well as NGOs. The program was conceived on four strategic and integrated lines: 1) building capacity in the local government and empowerment of the local community; 2) restoration of coastal habitat and fishery resources; 3) to provide alternatives of rent for the rural coastal communities, and 4) to develop a comprehensive coastal management plan with active participation of the civil society, integrating the federal and state environmental policies, programs and projects with the local ones. The main activities in development include: a) the development of an integrated environmental monitoring system of the estuary, b) the development of a Geographic Information System; c) the development of hydrodynamic and of quality of water patterns; d) the development of sustainable aquaculture of shrimp e) the recuperation of the city garbage deposit, the restoration of salt marshes, of estuarine coves, of the system of coastal dunes, f) the development of agroecology for the rural communities and urban peripheries and the development of ecotourism; g) the elaboration of a plan of fishery management and the Portuary Environmental Agenda among others. All these actions constitute instruments of an Integrated Management Plan (h), elaborated with the participation of the organized civil society, which is presently in its adoption phase (table 1). The program has not yet reached its evaluation phase.

Table 1: Strategic lines, activities and steps in the ICZM Program according to the cycles identified by Olsen and Uchoa [6] for the Integrated Costal Zone Management: 1) Diagnosis; 2) Planning; 3) Adoption; 4) Implementation; 5) Evaluation.

Strategic lines	Activities	steps
Building capacities	A,B,C,G,H	1,2
Habitat and resource recovery	D, G,E	1,2,3,4
Generation of economic alternatives	D, F	1,2,3
Management Plan	H	1,2,3,4

2 Main results

2.1 Integrated management plan

Similarly to many Latin American countries, the environmental management in the Patos Lagoon estuary has been characterized by the adoption of fragmented policies and located and unarticulated sectorial focus. Consequently, the decisive processes have been fragmented, reactive and with few expressive results concerning the point of view of behavioral changes in the society and in the environmental quality of the coastal ecosystem.

The Costa Sul Program has adopted the cooperative planning model, opposing the models that Müller [7] called DeAD (Decides, Approves, Defends) prevailing in the local structures of planning (Salas [8]).

The cooperative model is being initially tested in small scale, in the Marinheiros Island, the largest island of the estuary, where a community of 1,400 people, among them fishermen and farmers, lives. After a number of workshops through the year of 2006, with the intense involvement of the local community, university and city government, an Environmental Management Plan for this island was elaborated. The main results obtained to date can be considered: 1) the preparation of an agenda of priority themes to be worked on in the next five years; 2) the organization of the social participants due to the existing conflicts and problems; 3) the process of construction of an empowerment agenda of the communities before the decision making organs; 4) the implementation of a local management committee and of some goals established by the committee.

Once the Management of Marinheiro Island was adopted and implemented, the Costa Sul Program carried out the Management Plan of Patos Lagoon estuary on a broader scale. This plan has concentrated efforts in order to promote a transition of a present situation, of weak sustainability, marked by the disarticulation of policies and programs, institutions/resources and civil society, for a situation of strong sustainability where these start integrating coordinated strategic actions (Gómez [9]) (figure 1). Currently the integrated management plan is concluded in a participative way with the civil society and is now in the adoption phase.

2.2 Income alternatives creation

2.2.1 The pilot project sustainable shrimp aquaculture

The estuary of the Lagoa dos Patos is a natural habitat for the *Farfantepenaus paulensis*, therefore, this species is perfectly adapted to the environment. Since 2005 the local university has carried on efforts to develop small scale projects of shrimp aquaculture, on a familiar basis, aiming to provide to the fishermen an income alternative to the fishery. The structure's cultivation consists of low cost technology, using fences set up with bamboos and fine mash nets in the shallow estuarine waters. The fishermen and their family members are assisted with training courses in the fishing, associations and aquaculture areas.

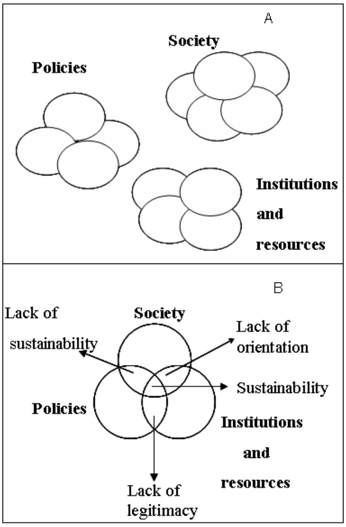

Figure 1: Conceptual framework of the strategy of the Integrated Management Plan of Patos Lagoon Estuary based on Gómez [9]; A) Scenery of weak sustainability; B) The common area between the three circles promotes the best conditions to the sustainability.

In parallel, the university carries on the technical support with the monitoring of water quality and researches applied in the field of physiology, ecology, land use planning and numerical modeling of the hydrodynamic and water quality in order to implement the activity of aquaculture in a sustainable way. At the end of January 2006, four structures of fences were installed and monitored. The product of the cultivation was identified with a stamp of the university, in order to differentiate the cultivated shrimp from the caught one. This measure made it possible to add value to the product, which was commercialized for US$ 5.5 / kg, a value four times superior to the one charged for the caught shrimp. Nevertheless the efforts of the local university, which each year has subsidized the implantation of pilot projects, the involvement of the fishermen is still incipient and a great difficulty persists for the effective implantation of the cultivation of the native shrimp in the estuary.

2.3 Habitat recovery

2.3.1 Salt marsh restoration

In sedimentary coastal regions, the water quality in shallow water bodies depends on the rooted vegetation that constitutes most part of the substrate for the settling of microorganisms which purify the water (processes of mineralization, nitrification, denitrification, etc.) (Hammer [10]). The increase of plant covering on the margins of lakes and lagoons results in the increment of the number of invertebrates and bigger reproductive success of aquatic birds (Chambers and McComb [11]). Consequently, rapid improvement in the quality of water and the trophic structure of anthropogenic impacted lakes and lagoons can be active by a plan of propagation and establishment of plants adapted to periodic flooding, such as grasses and sedges.

Over the last two centuries landfilling for urban and industrial development has destroyed several hectares of salt marshes in the main estuarine areas of Patos Lagoon. Creation and restoration of coastal marshes is nowadays a critical conservation issue facing the cumulative loss of wetlands and depreciation of remaining environments. The demonstrative project of the restoration of salt marshes which is being conducted in the ambit of the Costa Sul Program aims at:

- reverting the cumulative losses of vital areas of salt marshes in the Patos Lagoon estuary, through the restoration, creation and improvement of some selected units of salt marsh;
- intercepting, temporary stocking and/or breakdown part of the several contaminants normally present in the urban runoff of Rio Grande city through the transposition of dense vegetated stands of salt marshes between effluent damping points and the main body of the lagoon.
- mitigating erosive margins, exposed to dominant winds from the northeast and southeast, through the re-vegetation with native species.
- using the restoration activities of the project to promote the public awareness of the importance of salt marsh environments for the quality of life in the coastal zone.

The recuperation of salt marsh involves the activities of production of seedling and vegetative propagules, establishment of the transplants, monitoring

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of the created/restored salt marshes, distribution of advertising material and actions of environmental education. In the first year of the project a marsh area of 10,000 m^2 was recuperated by planting cordgrass *Spartina alterniflora* propagules, spaced 1 m apart, along slightly erosive margins and across urban runoff drains (figure 2).

Figure 2: a) Salt marsh restoration in the margins of the urban area of Patos Lagoon estuary, using *Spartina alterniflora*. The inserted figure shows the same estuarine embayment before the plantation of *Spartina*; b) The “Spartinário” installed at the university to produce the plants from seeds; the insert shows the external area of the greenhouse; c) The removal of debris to prepare the terrain; d) The educative panel installed nearby a restoration site.

2.3.2 Sand dune restoration

Among the actions of recuperation of coastal resources, the Costa Sul Program has worked towards the strengthening of the project of recuperation of coastal dunes which is being conducted by Rio Grande City government through the efforts of a local NGO. This effort resulted in the elaboration of a Management Plan of the coastal dunes which includes among other aspects a plan of territorial ordering on the beach zone. The management plan of the coastal dunes started to constitute one of the management instruments of the ICZM.

2.4 Empowerment of the artisanal fishermen communities

Currently, common sense exists in the ambit of Costa Sul Program that any effort applied to solve the crisis in the fishery sector will not be effective without the participation of the fishermen in the decision making instances. The artisanal fishermen community of Patos Lagoon estuary are politically organized in a sectorial Forum (Patos Lagoon Forum) formed by 15 public entities. The Costa Sul Program have carried out efforts to improve the role of participation of the fishermen in such forum, and building up a collaborative management plan for the artisanal fishery of Patos Lagoon estuary through out a series of workshops with an effective participation of the artisanal fishermen community.

3 Lessons learned

The superposition and the antagonism of interests of the public and private organizations, associated to the lack of capable human resources, constitute major obstacles for an effective integrated coastal management. The integration of the institutional matrix and the social participants strengthens and enlarges the governance, establishing the basic conditions for the promotion of sustainable development. The integration needs before hand, a correct identification of the stakeholders, and of a diagnosis of the institutional structure which enables identifying the responsibilities of each organization in the hierarchical structure of the public administration, its strengths, fragilities, opportunities and threats for the integrated coastal management,

On the other hand, the development of demonstrative projects of low scale, has permitted reaching concrete results in a short term. Such strategy has demonstrated being a useful instrument of learning and to convince and induce positive behavioral changes both in the public as well as in the private sector. In the case of the Environmental Plan of Marinheiro Island, the most expressive result was the process of building up the environmental agenda, which resulted in the empowerment of the local community and strengthened its group identity.

Acknowledgements

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References

[1] IDB Strategy Paper. Coastal and marine resources management in Latin America and the Caribbean. Inter-American Development Bank Document. Number ENV 129.Washington, D.C. 1998

[2] FAO (Food and Agriculture Organisation of the United Nations). The state of world fisheries and aquaculture. Rome: FAO; 2002

[3] Niencheski,F. & Baungartem.M.G.Z. Hidroquímica. *In:* : Tagliani, P. R. A. & Asmus M.L. (Coord.). *Estudo de Impacto Ambiental do Porto de Rio Grande*. FURG. Documento Técnico. pp.491-545. In, 850 pp. 1997
[4] Tagliani, P.R.A., Landázuri, H., Reis ,E.,Tagliani, C.R.; Asmus, M. & Arcilla, A.S. Integrated coastal zone management in the Patos Lagoon estuary: Perspectives in context of developing countries. Ocean and Coastal Management, Elsevier 46 .807-822. 2003)
[5] Almeida, M.T.A; Baungarten,M.G.Z.; Rodrigues R.M.S. Identificação das Possíveis Fontes de Contaminação das Águas que margeam a cidade de Rio Grande, Documentos Técnicos, Editora da FURG, 29p. 1993
[6] Olsen, S. B.; Ochoa, E. 2004. Folhas de Aplicação do Caderno de Trabalho. Marco metodológico e conceitual para o Planejamento e Implementação do Gerenciamento de Ecossistemas Costeiros. 33p
[7] Müller, B.: Redes de Cooperación y el Dilema de la Participación. Ponencia en la conferencia "Ordenamiento Territorial y Participación". Concepción. 2002
[8] Salas, E. Diálogos políticos: poniendo en práctica la gobernanza costera. En: Cooperación en el Espacio Costero. Ecoplata (en proceso de publicación).
[9] Gómez, R. Personal communication. Conference title: "Good Governance". GTZ workshop Recife, Brasil. 2005
[10] Hammer, D.A. Designing constructed wetlands systems to treat agricultural nonpoint source pollution. Ecological Engineering, 1: 49-82. 1992
[11] Chambers, J.M. & McComb A.J. Establishment of wetland ecosystems in lakes created by mining in Western Australia. In: Mitsch, W.J. (ed.). Global Wetlands: Old World and New. Elsevier, Amsterdam, pp. 431-441. 1994.

Biotic response to altered freshwater inflow patterns to the Kromme River Estuary, South Africa

T. H. Wooldridge
Department of Zoology, Nelson Mandela Metropolitan University, Port Elizabeth, South Africa

Abstract

The Kromme is an example of an estuary where freshwater inflow is significantly attenuated. Completion of a second major dam in 1983 increased total dam storage capacity above the Mean Annual Runoff (MAR) from the catchment. Currently, less than 2% of MAR reaches the estuary. Marine dominance is now persistent with little upstream variation in water salinity. Amplitude and frequency of flood events have also declined and most of the smaller floods and freshettes no longer arrive at the estuary. Sediment dynamics have also changed, particularly in the lower reaches where less frequent scouring enables disproportionately large sandbanks to develop. These sandbanks are only removed by very large floods. Sediment accumulation in the lower estuary has reduced the efficiency of tidal exchange and this exacerbates high salinity in the upper reaches. Because of changes to natural patterns in the physico-chemical environment, floral and faunal characteristics of the estuary have shifted. Eelgrass (*Zostera capensis*) particularly, has become more extensive (a four-fold increase in biomass compared to the natural state) due to less frequent floods. These eelgrass beds constitute the most important habitat with respect to invertebrate production. Invertebrate species that favour vegetated areas (e.g. the shrimp *Palaemon perengueyi*) currently dominate the benthic community while species that favour non-vegetated sediments have probably declined (e.g. the mudprawn *Upogebia africana*).
Keywords: freshwater attenuation, dam, estuary, community changes.

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1 Introduction

The completion in 1983 of a second major dam (Mpofu) on the Kromme River (Eastern Cape, South Africa), increased total storage capacity (133 x 10^6 m^3) above Mean Annual Runoff (MAR) from the catchment (Bickerton and Pierce [1]). The dam is close to the coast (18 km and only 4 km above the estuary) and now receives <2% of MAR. Average salinity range along the estuary is <5 psu and the upper reaches become hypersaline in summer with salinity seldom falling below 30 psu in any part (Whitfield and Bruton [2], Baird and Heymans [3], Wooldridge [4]). The impoundments also reduce frequency and amplitude of flood events and this has modified sediment dynamics. Less river sediment now reaches the estuary; in contrast to the lower reaches where infrequent flood scouring enabled disproportionately large sand banks to develop (Reddering [5]). These sandbanks are only removed by very large floods. Increased sedimentation in the lower estuary has also reduced the efficiency of tidal exchange and this exacerbates hypersalinity in the upper reaches.

Although post-1983 conditions reflect strong marine dominance, research undertaken prior to 1983 suggests that widespread euhalinity was already prevalent in the Kromme estuary. These earlier studies documented increased marine influence in summer when hypersaline conditions developed in the upper reaches (Hecht [6], Baird *et al.* [7], Marais [8], Bickerton and Pierce [1]). Hanekom [9] recorded salinity values close to 35 during two consecutive summers, also noting the effects of four floods (July-August 1979 and March and June 1981) that flushed the estuary on each occasion. In 1979, surface salinity values in the upper reaches were still below 10 psu two months after the August flood (Hanekom [9]).

Available salinity data therefore suggest that in the 1970s, euhalinity was already an intermittent feature of the Kromme estuary. The Churchill Dam (maximum capacity 33.3 × 10^6 m^3) higher up in the catchment would also have exacerbated euhaline conditions, but there are no empirical data to evaluate its impact. However, recent hydrological modelling of the estuary provides some potential answers with respect to the salinity regime in the estuary prior to the construction of the dams. Information generated by the model suggests that the system naturally fluctuated between a marine dominated state and a situation where a salinity gradient existed under median flow conditions (Department of Water Affairs and Forestry [10]). In addition, the report [10] stated that 'major floods have been dramatically modified and most of the freshets and smaller floods do not reach the estuary'. Integration of empirical data and information from the model would suggest that marine dominance was not an unnatural feature of the estuary, although the persistence of high salinity has probably increased as a consequence of the larger downstream dam in particular. Reductions in estuarine water salinity are now of shorter duration and occur less frequently because of impoundment effects.

The current paper reviews historical research records on the biota of the Kromme estuary and attempts to evaluate changes in community composition and structure brought about by the construction of the Mpofu impoundment completed in 1983.

2 Characteristics of the Kromme River and Estuary

The Kromme River rises some 95 kilometres from the sea in the Langkloof Valley between the coastal Tsitsikamma Mountains and the Kouga range. Quartzite forms the largest part of the geological substrate in the catchment basin; the total area varying between 936 km^2 and 1125 km^2, depending on the source reference (Bickerton and Pierce [1]).

Rainfall in the Kromme catchment is distributed throughout the year with spring and autumn peaks. Mean annual precipitation varies from 700 to 1200 mm. Flow patterns in the Kromme are erratic and floods have occurred in almost every month of the year (Bickerton and Pierce [1]).

The 13.7 km long estuary has an approximate surface area of 300 ha. The estuary has one major tributary, the Geelhoutboom (fig. 1). Although constricted, the tidal inlet remains permanently open to the sea. Tides are semi-diurnal with a small diurnal inequality. Mean spring tide range outside the inlet is about 1.75 m, while neap tides average 0.57 m. A flood tidal delta extends 5 km from the mouth, but additional sand is derived from an adjacent dunefield. In the upper reaches the estuary in narrow and incised into bedrock. The lower estuary is shallow (<2 m depth); upstream water depth averages 3-4 m in channel areas. Other physical characteristics of the estuary are given in Table 1.

Table 1: Physical characteristics of the Kromme estuary. Data from Baird and Ulanowicz [11] and Bickerton and Pierce [1].

Length (km)	Width (m)	Depth (m)	Tidal prism
13.7	Maximum 175 Average 80	Maximum 5 Average 2.5	1.87 x 10^6 m^3

The Geelhoutboom tributary rises in the Humansdorp area where the underlying geological formation is Bokkeveld slate that is readily eroded (Reddering and Esterhuysen [12]). Consequently, the Geelhoutboom River carries a relatively high sediment load when it enters the estuary about 8 km from the mouth. On occasions, fine sediment loading of the Geelhout tributary is exacerbated by runoff from farmlands adjacent to the river and estuary (fig. 1).

Sediments in the Kromme estuary become progressively finer-grained in an upstream direction, mainly because of the decreasing velocity of tidal currents (Reddering and Esterhuysen [12]). The fine muddy sediments in the middle and upper estuary have a fluvial origin, although the two large storage reservoirs probably stop most fluvial sediment input via the main tributary. Sediment input via the Geelhoutboom tributary will not be influenced by the reservoirs and fine material continues to be deposited into the estuary. According to Reddering and Esterhuysen [12], the mixing of marine sand and mud results in a compact mass that is not easily removed by floods.

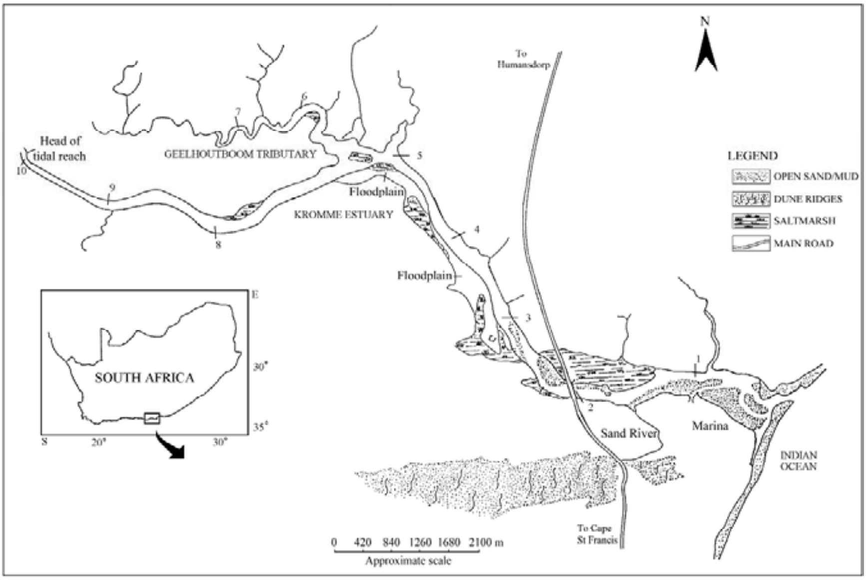

Figure 1: Map of the Kromme River Estuary and Geelhout tributary.

The lower reach of the estuary (ca 5 km) is relatively shallow and sandy with well-developed intertidal flats (fig. 1). This section of the estuary is very dynamic and channels continually change their position. Most of this sand is of marine origin. A further source of sand influx is via the Sand River that enters the estuary approximately 2 km from the mouth (fig. 1). The Sand River drains an extensive dune field to the southwest and carries sand into the lower estuary during occasional floods.

3 Present freshwater supply to the estuary

Present management policy provides for a total annual freshwater allocation of 2×10^6 m^3 for the estuary, unless natural overtopping of the dam occurs. However, overtopping is infrequent, and years may pass between overspill events. Severe drought at the end of the 1980s and early 1990s resulted in the reservoir levels falling below 30% of capacity (Jury and Levey [13]). Freshwater was then released on a monthly basis ($1/12 \times 2 \times 10^6$ m^3) in order to prevent hypersalinity developing in the upper estuary.

During the latter part of the drought (early 1990s) and up to the present time, no regular freshwater releases are made for environmental purposes. Consequently, river flow below the Mpofu dam is erratic and the estuary received little or no freshwater, except for local runoff after very heavy rains. Because the Mpofu Dam reduces natural runoff from the catchment, marine conditions in the estuary now persist for extended periods (years).

4 The influence of river impoundment on the macrophyte *Zostera capensis*

No spatial separation of macrophyte communities along the length of the Kromme estuary is apparent, although some variation occurs. This is due to the absence of a salinity gradient that normally structures the species composition as salinity decreases upstream (Adams *et al*. [14]). For example, *Zostera capensis* (eelgrass) currently extends into the upper reaches in comparison to its usual association with lower reaches of estuaries. During the period 1983 to 1992, Adams and Talbot [15] registered a four-fold increase in the standing biomass of eelgrass. This was primarily ascribed to reduced inflow of freshwater after construction of the dam, lack of sedimentary disturbances, stable salinity values and reduced turditities. Talbot *et al*. [16] consider flooding and associated sedimentary effects to be the over-riding forcing function determining the state of submerged macrophytes in small estuaries subject to occasional floods.

5 Changes in the macrobenthic community after the construction of the Mpofu dam 4 km above the estuary

Fifty-six macrobenthic species are listed by Bickerton and Pierce [1] in the Kromme estuary. These results are based on earlier studies by numerous researchers. The sandprawn *Callianassa kraussi* is one of the most widespread species, attaining densities of over 100 individuals per m^2 of substrate above the roadbridge (Day [17]). This species is relatively scarce in the sands at the mouth where it is replaced by a high density (136 per m^2) of *Loripes clausus* (Hecht 1973, quoted in Emmerson *et al*. [18]).
The mudprawn *Upogebia africana,* occurs upstream of the bridge where a muddy substratum is present. In places, density exceeds 100 individuals per m^2 (Day [17]). The crab *Sesarma catenata* is common in areas where saltmarsh occurs. Submerged *Zostera* beds (eelgrass) along the main estuary also harbour a rich and abundant fauna (Emmerson *et al*. [18], Hanekom [9]). Emmerson *et al*. [18] listed dominant species present in eelgrass beds that included the molluscs *Arcuatula capensis* (max. 469 individuals per m^2), *Macoma litoralis* (max. 181 individuals per m^2) and *Nassarius kraussianus* (max. 241 individuals per m^2), as well as crustaceans such as *Cleistostoma edwardsii* and *C. algoense* (max. 296 and 156 individuals per m^2 respectively).

Hanekom [9] recorded 29 macrobenthic species using a 1 mm mesh sieve in intertidal *Zostera* beds. Thirteen of the species were crustaceans and 12 were molluscs. In addition, Hanekom [9] sampled non-vegetated areas adjacent to *Zostera* beds. Most species occurred in both habitats, but the isopod *Exosphaeroma hylocoetes*, the molluscs *Arcuatula capensis*, *Haminea alfredensis* and *Natica tecta* were found only at sites covered with *Zostera*. Other species such as the polychaete worm *Ceratonereis erythraeensis*, the crabs *Cleistostoma edwardsii* and *Hymenosoma orbiculare* were also more abundant in vegetated areas. The mudprawn *Upogebia africana* on the other hand, was more common in open areas.

The general distribution of the 29 species recorded by Hanekom [9] along the length of the Kromme estuary was similar to that recorded by Hecht [6] who sampled in *Zostera* and non-*Zostera* areas. No one species was present at all sites, although the crown-crab *Hymenosoma orbiculare*, the bivalve *Macoma littoralis* and the crab *Cleistostoma edwardsii* occurred at most sites. Species such as *Alpheus crassimanus*, *Betaeus jucundus*, *Loripes clausus, Polybranchiorhynchus dayi* and *Solen capensis* were relatively rare (<10% of sites sampled).

Decapod crustaceans *Cleistostoma edwardsii* and *Upogebia africana* tended to dominate lower and lower-middle reaches, but upper reaches were dominated by a mollusc community *(Arcuatula capensis, Macoma litoralis, Solen cylindraceus* and *Nassarius kraussianus*). The crown crab *Hymenosoma orbiculare* was more or less evenly distributed along the estuary. Bivalves such as *Loripes clauses* and *Solen capensis* were limited to the mouth region.

Winter and Baird [19] have underlined the importance of anomuran, brachyuran and macruran crustaceans in controlling energy flow in many Eastern Cape estuaries, including the Kromme. The most important of these species from an energy flow perspective is the shrimp *Palaemon perengueyi* (Table 2). *Palaemon peringueyi* occurs primarily in subtidal eelgrass habitat (Emmerson [20]) and attains extremely high densities (max. 1016 individuals per m^2) (Emmerson *et al*. [21]). This species breeds in nearshore oceanic waters and postlarvae then migrate into estuaries or pools along the intertidal rocky shore where they utilize available resources. Table 2 below ranks the five most important species with respect to energy flow in the Kromme estuary.

Table 2: Contribution to annual production of the five most important invertebrates present in the Kromme estuary prior to the construction of the 2nd dam in 1983 (extracted from Winter and Baird [19]).

Species	Contribution to macroinvertebrate production (%)
Palaemon perengueyi	32
Sesarma catenata	22
Upogebia africana	13
Macoma litoralis	10
Callianassa kraussi	8

The five major producers contributed 85% to total production. Although the total number of species recorded was much higher, the data reflected the importance of a few key species. In terms of habitat, benthic macroinvetebrate production was greatest in *Zostera capensis* beds (35.2%), followed by saltmarsh and mud habitats (each 25.2%) (Winter and Baird [19]). Thus, 85% of macroinvertebrate production in the Kromme estuary was associated with non-sandy habitats upstream of the mouth area where *Zostera capensis* was most prevalent.

In terms of freshwater reduction to estuaries, changes in the biota do not lead to changes in trophic levels, but rather to major shifts in the primary producers and trophic pathways (Grange *et al.* [22]). In the Kromme estuary, Baird and Heymans [3], report that there was a major decline in the zooplankton standing stock as a consequence of reduced phytoplankton stock after 1983 when the dam was built. Submerged macrophyte biomass on the other hand increased from 60 to 125 g C m^{-2}, leading to changes in the balance between primary producers. This is supported by Adams and Talbot [15] who recorded a four-fold increase in standing biomass of *Zostera capensis* in the nine-year period after construction of the Mpofu dam. This included a 2.4 fold increase in density and a 1.6 fold increase in aerial coverage.

Table 3: Predicted change in the standing stock of key macrobenthic species after the construction of the Mpofu dam in 1983. The predicted change is based on habitat preference of the species (see text) and the four-fold increase in *Zostera capensis* standing stock in the estuary. (+) indicates an increase in standing stock, (-) indicates a decrease in standing stock.

Species	**Predicted change**	**Reference source**
Upogebia africana	-	Hanekom [9]
Exosphaeroma hylecoetes	+	Hanekom [9]
Palaemon perengueyi	+	Emmerson *et al*. [21]
Cleistostoma edwardsii	+	Hanekom [9]
Hymenosoma orbiculare	+	Hanekom [9]
Arcuatula capensis	+	Hanekom [9] Emmerson *et al*. [21]
Haminea alfredensis	+	Hanekom [9]
Natica tecta	+	Hanekom [9]

Increase in *Z. capensis* density and coverage after dam construction would also have led to changes in zoobenthic community structure. Predicted shifts in population abundance levels and contribution to invertebrate production for some of the important benthic species are listed in Table 4. These predicted shifts are based on habitat preference already discussed. *Palaemon perengueyi* for example, occurs primarily in eelgrass beds (Emmerson [20]) and the increase in *Z. capensis* density and coverage would therefore have favoured an increase in *P. perengueyi* biomass, a species responsible for about 32% of invertebrate annual production (Table 2) in the Kromme estuary before 1983. Thus, abundance and proportional contribution of *P. perengueyi* to production is also likely to have increased. Similarly species that prefer non-vegetated habitats along the intertidal and marginal fringes would have responded negatively to an increase in *Zostera capensis* biomass (e.g. the mudprawn *Upogebia Africana*).

6 Conclusion

Construction of a second major dam in 1983 on the Kromme River increased total reservoir storage capacity above Mean Annual Runoff (MAR) from the catchment. Although marine dominance was not an unnatural feature of the estuary, the persistence of high salinity has increased. Reductions in estuarine water salinity are now of shorter duration and occur less frequently because of impoundment effects. These impoundments also reduce frequency and amplitude of flood events. As a consequence of a less variable salinity regime, reduced sedimentary disturbance and reduced turbidity, eelgrass (*Zostera capensis*) distribution and coverage has expanded, with biomass increasing four-fold. This has led to structural and functional changes in the invertebrate community. Zooplankton biomass has declined while biomass of benthic species and their relative importance as contributors to invertebrate production in the estuary has probably increased significantly (e.g. *Palaemon perengueyi*). By contrast, intertidal mudbanks have decreased in area (increased *Zostera capensis* coverage) and this has probably led to a decline in biomass of species inhabiting these open habitats (e.g. *Upogebia africana*).

References

[1] Bickerton, I.B. & Pierce, S. M., Estuaries of the Cape. PartII. Synopsis of available information on individual systems, Report No. 33: Krom (CMS 45), Seekoei (CMS 46) and Kabeljous (CMS 47), eds. A.E.F. Heydorn. & P.D. Morant, CSIR Research Report432, Stellenbosch, pp. 1-109, 1988.

[2] Whitfield, A.K. & Bruton, M.N., Some biological implications of reduced fresh water inflow into Eastern Cape estuaries: a preliminary assessment. South African Journal of Science, **85**, pp. 691-694, 1989.

[3] Baird, D. & Heymans, J.J., Assessment of ecosystem changes in response to freshwater inflow of the Kromme estuary, St Francis Bay, South Africa: a network analysis approach. WaterSA, **22**, pp. 307-317, 1996.

[4] Wooldridge, T.H., Estuarine zooplankton community structure and dynamics (Chapter 7). Ecology of South African Estuaries, eds. B. Allanson & D. Baird, Cambridge University Press, UK, pp. 141-166, 1999.

[5] Reddering, J.S.V., Prediction of the effects of reduced river discharge on the estuaries of the south-eastern Cape Province, South Africa. South African Journal of Science, **84**, pp. 726-730, 1988.

[6] Hecht, T., The ecology of the Kromme River Estuary with special reference to Sesarma catenata. Unpublished MSc dissertation, University of Port Elizabeth, pp. 1-150, 1973.
[7] Baird, D., Marais, J.F.K. & Wooldridge, T.H., The influence of a marina canal system on the ecology of the Kromme Estuary, St Francis Bay. South African Journal of Zoology, **16**, pp. 21-24, 1981.
[8] Marais, J.F.K., Seasonal abundance, distribution and catch-per-unit-effort of fishes in the Kromme estuary, South Africa. South African Journal of Zoology, **18**, pp. 96-102, 1983.
[9] Hanekom, N., An ecological study of the Zostera beds in the Kromme Estuary. University of Port Elizabeth Report No. 18, pp. 1-163, 1982.
[10] Department of Water Affairs and Forestry, South Africa, Kromme/Seekoei Catchments Reserve Determination study – technical component. Kromme Estuary. Prepared by CSIR for Coastal and Environmental Services, Report No. RDM/EWR001/ER0005/CON/CES/1105, pp. 1-135, 2005.
[11] Baird, D. & Ulanowicz, R.E., A comparative study of the trophic structure, cycling and ecosystem properties of four tidal estuaries. Marine Ecology Progress Series, **99**, 221-237, 1993.
[12] Reddering, J.S.V. & Esterhuysen, K., Sedimentation in the Kromme estuary. University of Port Elizabeth, ROSIE Report No. 6, pp. 1-92. 1983.
[13] Jury, M.R. & Levey, K., The Eastern Cape drought. Water SA, **19**, pp. 133-137, 1993.
[14] Adams, J.B., Knoop, W.T. & Bate, G.C., The distribution of estuarine macrophytes in relation to freshwater. Botanica Marina, **35**, pp. 215-226, 1992.
[15] Adams, J.B. & Talbot, M.M.B., The influence of river impoundment on the estuarine seagrass Zostera capensis Setchell. Botanica Marina, **35**, pp. 69-75, 1992.
[16] Talbot, M.M.B., Knoop, W.T. & Bate, G.C., The dynamics of estuarine macrophytes in relation to flood/siltation cycles. Botanica Marina, **33**, pp. 159-164, 1990.
[17] Day, J.H., Summaries of current knowledge of 43 estuaries in southern Africa (Chapter 14). Estuarine ecology with particular reference to southern Africa, ed. J.H. Day, A.A. Balkema, Cape Town, pp. 251-329, 1981.
[18] Emmerson, W.D., Watling, H.R. & Watling R.J., Community analyses in the Kromme and Swartkops estuaries and in the Algoa Region. University of Port Elizabeth Zoology Department Report Series No. **16**, pp. 1-128, 1982.
[19] Winter, P.E.D. & Baird, D., Diversity, productivity, and ecological importance of macrobenthic invertebrates in selected Eastern Cape Estuaries (Chapter 15), Towards an environmental plan for the Eastern Cape, ed. M.N. Bruton & F.W. Gess, Grocott & Sherry, Grahamstown, pp. 149-154, 1988.

[20] Emmerson, W.D., The ecology of Palaemon pacificus (Stimpson) associated with Zostera capensis Setchell. Transactions of the Royal Society of South Africa, **46**, pp. 79-97, 1986.
[21] Emmerson, W.D., Watling, H.R. & Watling R.J., Community analyses in the Kromme and Swartkops estuaries and in the Algoa region, University of Port Elizabeth Zoology Department Report Series, **No. 16**, pp. 1-128, 1982.
[22] Grange, N., Whitfield, A.K., De Villiers, C.J. & Allanson, B.R., The response of two South African east coast estuaries to altered river flow regimes, Aquatic Conservation: Marine and Freshwater Ecosystems, **10**, pp. 155-177, 2000.
[23] Jerling, H.L. & Wooldridge, T.H., The mesozooplankton of a freshwater-starved estuary, Changes in fluxes in estuaries: implications from Science to Management, eds K.R. Dyer & R.J. Orth, Olsen & Olsen, Fredensborg, Denmark, pp. 301-306, 1994.

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