APUSS

Assessing Infiltration and Exfiltration on the Performance of Urban Sewer Systems

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DELIVERABLE 10

Proposal of a methodology to compare investment strategies accounting for infiltration / exfiltration problems and application to a semi-virtual case-study

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Table of contents

1 Objectives **3**
2 Description of the methodology **4**
2.1 General scheme 4
2.2 Evaluation of the impacts of I/E on the drainage system (step 1) 5
2.3 Definition of criteria to be used for the comparison (step 2) 6
2.4 Definition of investment strategies (step 3) and evaluation of their impacts on the drainage system (step 4) 7
2.5 Possible additional criteria (step 5) 8
2.6 Calculations of the criteria (step 6), analysis using multi-criteria approach (step 7) and final decision (step 8) 10
2.7 Remarks for application of the methodology 12
3 Description of the semi-virtual case study **13**
3.1 Presentation of the components 13
3.2 Simulation results 16
3.2.1 Annual rainfall variability 16
3.2.2 Foul sewage and total wastewater flows and pollutant loads 16
3.2.3 CSO tank 18
3.2.4 Wastewater treatment plant (WWTP) 19
4 Impacts of infiltration **21**
4.1 Impacts on the CSO tank 21
4.2 Impacts on the WWTP 22
4.2.1 Pollutant loads 22
4.2.2 Operation costs 26
5 Definition of rehabilitation strategies **28**
5.1 Investment in higher treatment plant capacities 28
5.1.1 Impacts on pollution 28
5.1.2 Impacts on operation costs 29
5.2 Investment in a larger CSO tank volume 30
5.3 Rehabilitation of the sewer system 32
5.4 Mixed investment: renovation of 50 % of the pipes and investment in upgrading the WWTP 34
6 Comparison of the strategies **35**
6.1 Impacts of the investment strategies on each selected criterion 35
6.2 Multi-criteria analysis of each IR scenario 38
6.3 Discussion of the results – influence of hypotheses 42
7 Conclusion **44**
8 Acknowledgements **44**
9 References **45**
10 Annexes **46**

1 Objectives

For sewer systems where infiltration and/or exfiltration (I/E) problems have been observed, end-users frequently have to make decisions and choices between some possible investment strategies aiming to solve the observed problems. In most frequent cases, the comparison of the possible strategies is based on a traditional cost-benefit approach, which consists to minimise the costs of investment and operation for an expected given benefit. Two of the main limitations in such an approach are : i) the difficulty to evaluate all criteria in monetary units (e.g. what is the cost in Euros of any kg of COD discharged into a river by a wastewater treatment plant (WWTP) overloaded by infiltration ?), ii) the ignorance of some criteria that can not be easily calculated, like environmental aspects.

It is obvious that the choice of any investment strategy regarding infiltration or exfiltration phenomena should include environmental, technical and financial aspects. The two main objectives of the APUSS WP 10 consist :

- to propose a general methodology based on an integrated and multi-criteria approach to compare and rank investment strategies to remedy I/E problems ;
- to illustrate the application of this methodology for a semi-virtual case study.

This report contains five chapters :

- description of the methodology (chapter 2)
- description of the semi-virtual case study (chapter 3)
- application of the methodology including:
    - evaluation of the impacts of infiltration on the drainage system (chapter 4)
    - definition of the rehabilitation strategies (chapter 5)
    - comparison of the strategies and discussion of the results (chapter 6).

The application of the proposed methodology requires various basic elements (methods, tools, models and software) which will not be fully described in this final report. Appropriate literature references are given for the readers who would like to have fully detailed information on these elements.

Finally, it should also be clearly emphasised here that the most important point in this report is the general methodology, that can be applied by any end-user with his/her own information and necessary adaptations. The semi-virtual case study is given only to illustrate how the methodology can be applied for a specific case, under some specific simplifications and hypotheses. Consequently, the conclusions drawn from this example should not be neither generalised nor extrapolated.

2 Description of the methodology

2.1 General scheme

The proposed methodology is represented in Figure 1. The 8 steps are briefly described in the following paragraphs.

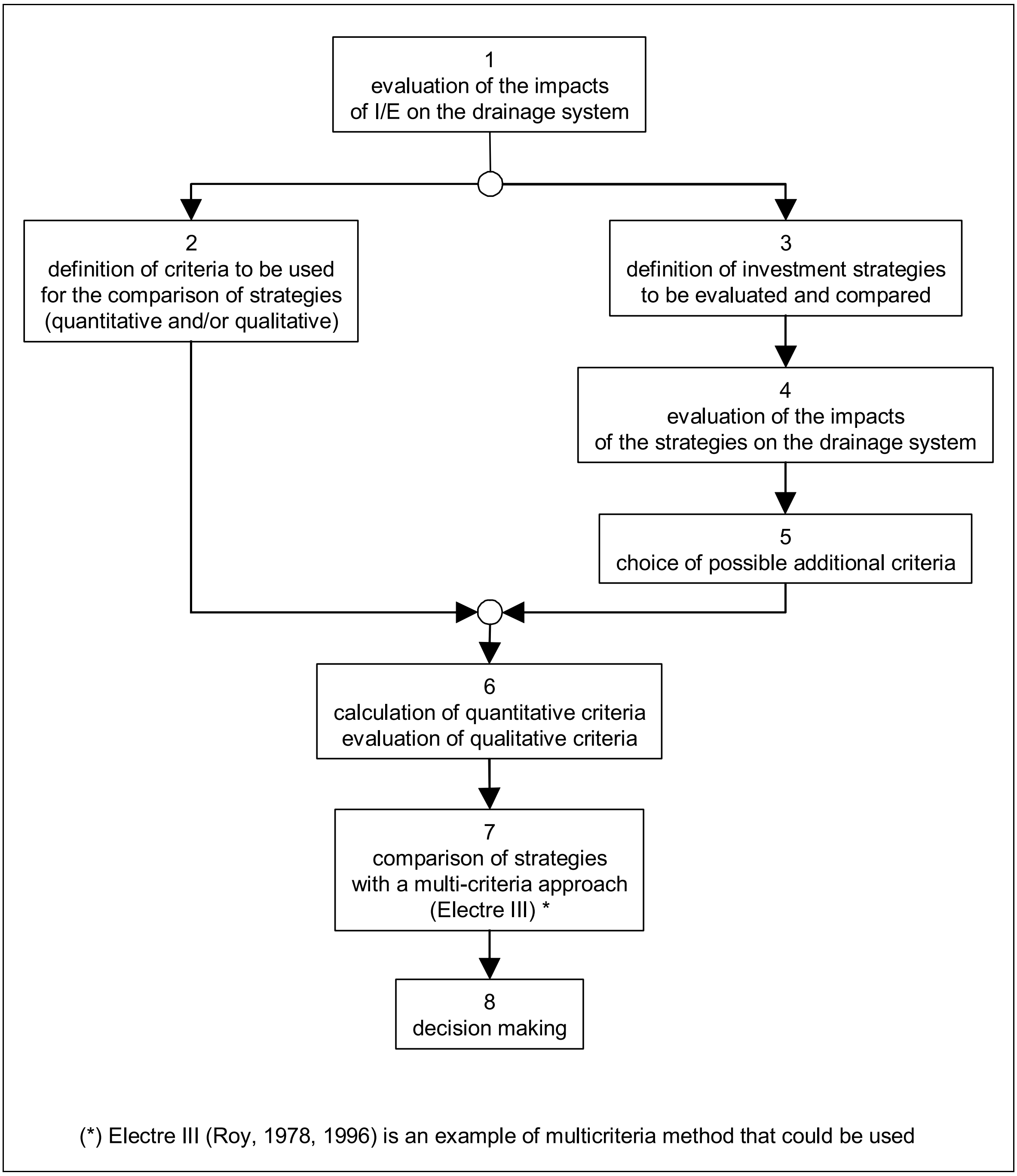

Figure 1 : Methodology for definition, comparison and choice of investment strategies regarding infiltration / exfiltration based on a multi-criteria analysis.

2.2 Evaluation of the impacts of I/E on the drainage system (step 1)

Infiltration and exfiltration may have numerous impacts on the urban drainage system. The inventory and the evaluation of these impacts are fundamental in order to get an exhaustive view of the different aspects and to avoid neglecting any aspect. Based on previous literature review carried out in the APUSS project (Neitzke, 2002), the impacts of infiltration and exfiltration are summarized respectively in Figure 2 and Figure 3.

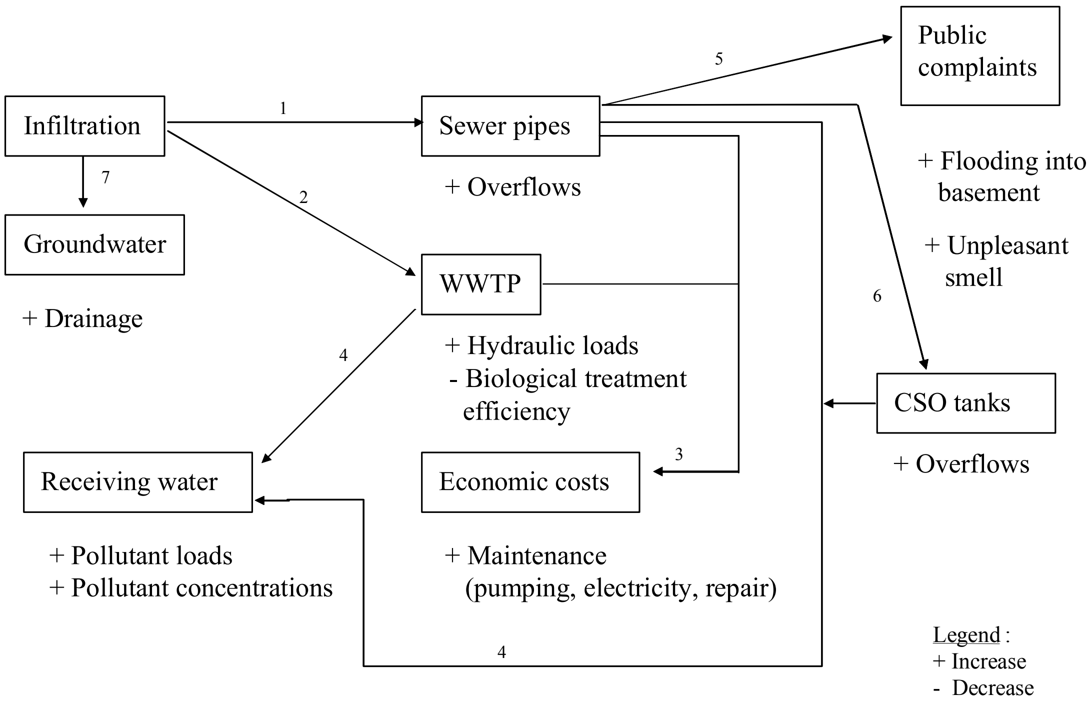

Figure 2 : Impacts of infiltration on urban drainage systems (from Neitzke, 2002).

Infiltration increases the hydraulic loads in sewer pipes, which lead to more frequent and premature combined sewer overflows CSOs (1), which can be partly avoided by storage of excess water in CSO tanks (6). Emptying CSO tanks increases pumping times and energy consumption, leading to higher energetic costs (3). CSOs and CSO tanks both increase the pollutant loads discharged into the receiving waters (4). Overflows may also lead to flooding in house basements, unpleasant smells and population claiming (5).

Infiltration also impacts the WWTP operation. An increased hydraulic load leads to higher pumping costs (3). The biological treatment efficiency decreases because of the dilution of influent concentrations (2). Due to permanent infiltration, the capacity of the WWTP is also reduced during storm events, which results in i) more frequent discharges of untreated water and ii) higher risk of sludge losses from the secondary clarifier. As a consequence, the pollutant loads discharged into the receiving waters increase (4).

At last, drainage of groundwater into sewers by infiltration may lead to the lowering of groundwater levels, with potential geotechnical risks for building and urban infrastructures (7).

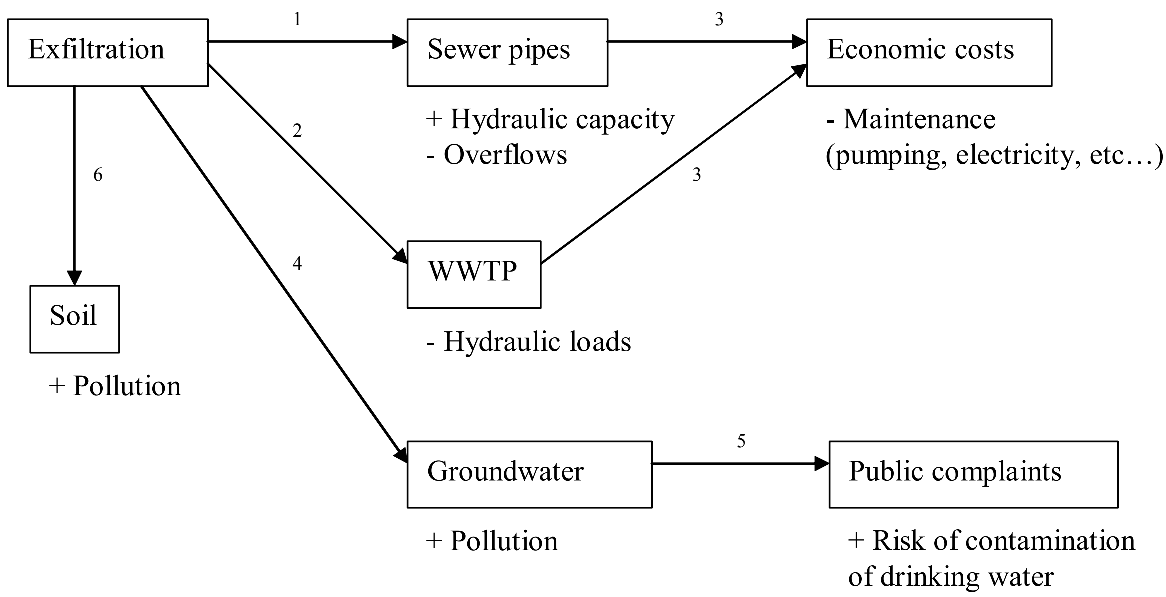

Figure 3 : Impacts of exfiltration on urban drainage systems (from Neitzke, 2002).

Exfiltration decreases the hydraulic loads in the sewer pipes (1) and the hydraulic loads entering the WWTP (2). This leads to economic savings because less electricity is needed for pumping and for operation of the WWTP (3). However, the main risk linked to exfiltration is the pollution of soil (6) and of the groundwater (4), which may lead to the contamination of drinking water resources (5).

For any specific catchment, all above impacts have to be checked (are they all pertinent in this particular catchment ?) and evaluated (flows, volumes, transfer of pollutants, etc.) by means of appropriate measurements and modelling. The proposed methodology itself (Figure 1) is not prescriptive : all methods, tools, software and measurements usually applied by the end-user or by his/her sub-contractors (e.g. consulting companies) can/should be used to evaluate the impacts. Of course, the quality of this evaluation is a key element for the quality of the following steps in the proposed methodology. This point has to be clearly emphasised, and the necessary efforts shall be devoted to get and elaborate the requested knowledge of the phenomena and of the impacts.

2.3 Definition of criteria to be used for the comparison (step 2)

The above impacts shall be expressed through criteria that can be used for their evaluation and for the comparison of investment strategies. In the multi-criteria approach Electre III (see Roy, 1978, 1996; Vincke, 1992 for detailed information on this approach) to be used in step 7, it is useless to express all criteria with a common unit (like e.g. Euros in traditional cost-benefit approaches). The units used to evaluate the impacts can be kept as they are. For example, pollutant loads are expressed in kg, costs in Euros, overflow volumes in m^3, etc.

Some impacts may be difficult to evaluate, such as groundwater drainage, flooding in basements, groundwater pollution risk, etc., because of the lack of knowledge regarding the phenomena involved in the corresponding processes, the lack of data and information to calculate the criteria, etc. This is why, when criteria can not be calculated with numerical values, they can be expressed in Electre III by evaluation grades based on end-user's expertise and estimation.

Some impacts could also be considered as not really important for a given case, and could be intentionally neglected after exhaustive review and evaluation have been completed. This choice made by the end-user should always be site-specific. In the following case-study (chapter 3), only pollutant loads discharged into receiving water and economic costs have been considered. Individual sewer pipe overflows, flooding in basements and groundwater drainage have not been modelled as they are not relevant in this particular case.

2.4 Definition of investment strategies (step 3) and evaluation of their impacts on the drainage system (step 4)

At first, possible investment strategies have to be reviewed. Regarding infiltration, investment strategies may concern sewer pipes (rehabilitation), the WWTP (higher treatment capacities) and CSO storage tanks (higher tank capacities). Regarding exfiltration, only rehabilitation of sewer pipes may solve the negative impacts induced by exfiltration. Then each possible strategy should be defined in detail and evaluated.

Regarding the selection of rehabilitation technologies (RTs), many reviews exist in journals, conferences, textbooks and on the web. In France, the French National research project RERAU (RERAU, 2004) is a very useful source of information. At European level, the FP5 project CARE-S (part of the CityNet cluster) has made an inventory of 73 techniques (Villanueva *et al.*, 2004). Rehabilitation technologies could be distributed in three categories : i) repair, ii) renovation and iii) replacement, depending on the importance of the planned rehabilitation. Within the CARE-S project, CLABSA s.a. and SINTEF s.a. developed both a database for RTs (Villanueva *et al.*, 2004) and a cost tool (Freni *et al.*, 2005). The database allows the user to select appropriate RTs based on the observed defects, on the pipe characteristics (diameter, material, accessibility, etc.) and on the technical function expected from the RT (structural improvement, sealing, etc…).

O'Brien and Gere Engineers (O'Brien and Gere Companies, 2001) have compared different RTs (mostly trenchless technologies) based on their impacts, i.e. costs, pumping requirements, surface disruption, lateral interruption and upsizing possibilities (Figure 4).

The two main impacts are costs and traffic disruption. Costs may vary a lot from one technology to another (see the case-study in chapter 3). Investment costs also depend on the context (traffic, accessibility) and on the characteristics of the damaged pipes such as diameter, depth (see Frehmann, 2004 : APUSS deliverable D11.2). Consequently, it is difficult (and even not pertinent) to give an average cost of the different RTs. Any evaluation should be site specific and based on the local circumstances.

Surface disruption appears frequently as the main reason for the non applicability of a given RT because the disturbance of traffic is considered as a very critical negative impact. Usually, taking into account the environment and the characteristics of the sewers, the number of RTs potentially applicable to a given catchment is rather limited.

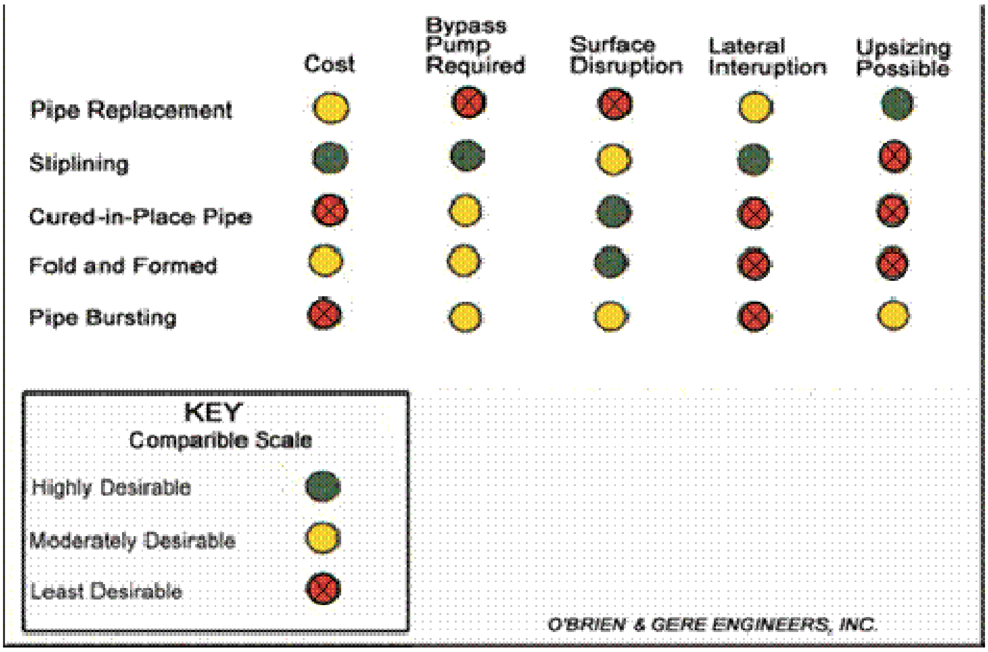

Figure 4 : Comparison of different RTs. Sliplining, CIPP, Fold and Formed, and pipe bursting are trenchless technologies (adapted from O'Brien and Gere companies, 2001)

2.5 Possible additional criteria (step 5)

In addition to the usual criteria selected in step 2, it might be interesting to have a long-term view and to integrate the expected life time (life cycle) of rehabilitation in the comparison of the investment strategies. Indeed, the expected life time vary dramatically among the various RTs that may be used. E.g. Frehmann (2004) gives orders of magnitude of the expected life time of sewer pipes depending on the RT used (Table 1).

	threshold value	**cover depth H = 4 m**	**cover depth H = 10 m**	
Replacement	80 to 100 years	45 years to 61 years to 88 years	65 years to 103 years to 141 years	minimum expected value maximum
Rehabilitation	50 to 60 years	47 years to 65 years	19 years to 31 years	minimum maximum
Interest calculation is not considered.				

Table 1 : Amortisation – life cycle of sewer pipes (from Frehmann, 2004).

Delleur *et al.* (1998) propose to use the RT expected life time to calculate the annual unit construction costs AUCC as follows :

$$AUCC_j = \left[\frac{r(1+r)^{n_j}}{(1+r)^{n_j} - 1} \right] UCC_j$$

where r is the interest rate, n_j is the expected life time of the rehabilitation technology j and UCC_j the unit construction cost of the rehabilitation technology j.

The higher the expected life time of the RT is, the lower the AUCC value will be. An investment strategy i could have a higher UCC than a strategy k, but taking into account its longer life time, its AUCC could be lower. This expected life time is difficult to estimate because it varies according to the context (traffic, depth, etc.). A study carried out by Orditz (2004) on the durability of rehabilitations in sewer systems showed that very few studies have been produced on this topic. Moreover, most of rehabilitation technologies have been applied too recently (up to maximum 25 years) to have long-term knowledge on their durability.

Contrary to investment strategies in WWTPs or in CSO tanks, rehabilitation of sewer systems may also increase the asset serviceability of the sewer system. The concept of asset serviceability has been defined by Ewan Associates and Mac Donald (2001) as "the ability of an asset to deliver a defined service to customers and safeguard the environment". Increasing the serviceability implies minimizing the dysfunctions of the considered system. For sewer systems, a list of all possible dysfunctions has been established in the European project CARE-S according to the European Standards EN 752-2 (1996) and EN 13508-1 (2003):

- ongoing degradation from abrasion
- blockage
- risk of collapse
- ongoing corrosion
- excessive spillage
- exfiltration (seepage loss)
- flooding
- decrease in hydraulic capacity
- infiltration
- ongoing degradation from root intrusion
- sand silting
- destabilization of ground-pipe system.

The sewer serviceability may be an important criterion because it expresses that sewer rehabilitated for infiltration purposes could contribute to the improvement of pipes which should have been rehabilitated for other dysfunction purposes such as risk of collapse, ongoing corrosion, etc. The increase of the asset serviceability following the sewer rehabilitation could be expressed in quantitative criteria such as reduction of number of flooding events. However, for most of dysfunctions, it may be hard to estimate the impact of the sewer rehabilitation on them. Consequently, only qualitative notes can be used which are based on end-user's knowledge and expertise.

Other additional criteria could be defined by the end-user depending on his/her objectives, on available knowledge, data and models. The advantage of the proposed methodology is to be adaptable to the each specific case. All information, impacts and phenomena that should be taken into account can be introduced as evaluated either quantitatively or qualitatively by means of grades.

2.6 Calculations of the criteria (step 6), analysis using multi-criteria approach (step 7) and final decision (step 8)

After calculation of quantitative criteria and estimation of qualitative ones (step 6), the analysis of the results will be made by using a multi-criteria approach.

Among the several existing multi-criteria methods, the Electre II and III methods are well known and widely used. Contrary to traditional cost-benefit approaches, all criteria are expressed in their own natural units, which avoids the problems of averaging of or compensating for the criteria. Contrary to Electre II (Roy and Bertier, 1971; Roy, 1996), Electre III (Roy, 1978; Vincke, 1992) accounts for uncertainties in the evaluation of the performance indicators or of criteria. It is then more appropriate for studies on environmental aspects, because of the high uncertainties in data and results (field measurements, variability from one year to another one, etc.). An example of application to stormwater storage tank cleaning maintenance, with a brief summary of the Electre III method, has been given e.g. by Bertrand-Krajewski *et al.* (2002). Electre III is a ranking method (Roy, 1978 ; Vincke, 1992) using "pseudo criteria" that explicitly take into account an indifference threshold q_j and a strict preference threshold sp_j per criterion j. The indifference threshold is particularly interesting, as it can be used to integrate the specific uncertainties in each criterion : Electre III is then less sensitive to variations of the input data and parameters than Electre II. The Electre III method requires to define a weight w_j for each criterion j. These weights are chosen by the end-user, according to his/her own hierarchy and preferences. Then, the actions or strategies are compared in ordered pairs (S_i, S_k). Each pair of actions is characterised by an outranking degree $S(S_i, S_k)$ expressing the credibility of the statement "S_i outranks S_k". The outranking degree is calculated according to two types of index : a concordance index and discordance indexes.

One defines the following variables :

w_j weight of criterion j
p_{ij} performance of a strategy i according to the performance indicator or criterion j
$q_j(p_{ij})$ indifference threshold which is either a constant or expressed as a percentage of p_{ij}
$sp(p_{ij})$ strict preference threshold which is either a constant or expressed as a percentage of p_{ij}

The concordance index is expressed as follows (see Figure 5) :

$$C(S_i, S_k) = \frac{\sum_{j \in J} w_j c_j(S_i, S_k)}{\sum_j w_j}$$

where

$$c_j(S_i, S_k) = \begin{cases} 1 & \text{if } p_{ij} + q_j(p_{ij}) \geq p_{kj} \\ 0 & \text{if } p_{ij} + sp_j(p_{ij}) \leq p_{kj} \\ & \text{linear in between} \end{cases}$$

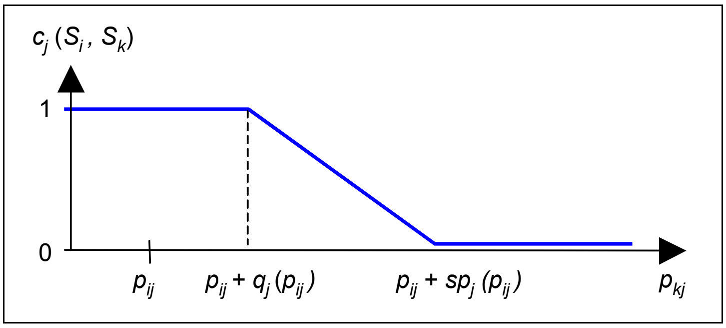

Figure 5 : Calculation of a partial concordance index c_j (S_i, S_k) for an indicator j.

The definition of the discordance index for each criterion j requires the introduction of a veto threshold $v_j(p_{ij})$ refusing the outranking of S_i by S_k when S_i is not preferred to S_k on criterion j and when the difference between the two performances is greater than $v_j(p_{ij})$. The discordance index for each criterion j is expressed as follows (see Figure 6) :

$$D_j(S_i,S_k) = \begin{cases} 0 & \text{if } p_{kj} \le p_{ij} + sp_j(p_{ij}) \\ 1 & \text{if } p_{kj} \ge p_{ij} + v_j(p_{ij}) \\ & \text{linear in between} \end{cases}$$

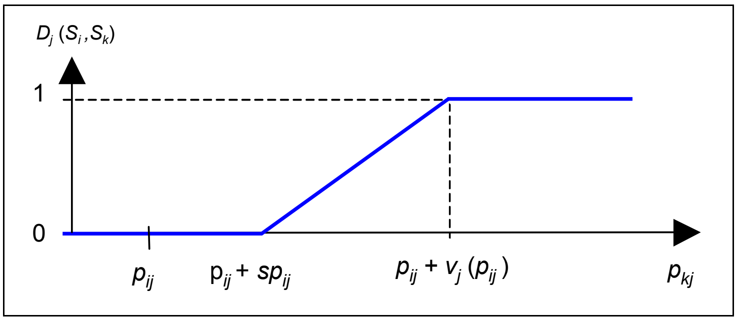

Figure 6 : Calculation of a partial discordance index D_j (S_i, S_k) for an indicator j.

The degree of outranking $S(S_i, S_k)$ is then defined by :

$$S(S_i,S_k) = \begin{cases} C(S_i,S_k) & \text{if } \forall j\ D_j(S_i,S_k) \le C(S_i,S_k) \\ C(S_i,S_k) \cdot \prod\limits_{j \in J} \dfrac{1 - D_j(S_i,S_k)}{1 - C(S_i,S_k)} & \text{in other cases} \end{cases}$$

The valued outranking relationships are then used in order to build a simple outranking graph. An outranking arc between two actions S_i and S_k is defined if $S(S_i, S_k)$ is close to the maximum λ of all outranking degrees and if the difference between $S(S_i, S_k)$ and $S(S_k, S_i)$ is significant. For that purpose, a threshold $s(\lambda)$ has to be determined.

There is an arc in the graph between S_i and S_k if $S(S_i, S_k) \ge \lambda - s(\lambda)$ and if $|S(S_i, S_k) - S(S_k, S_i)| \le s(\lambda)$. For each action S_i a "qualification" $Q(S_i)$ is calculated according to the difference between the number of actions which are outranked by S_i and the number of actions which outranks S_i. The solutions that present the best qualification receive a rank equal to 1. These actions are then removed from the initial set of actions and the same procedure is started again in the remaining set (new λ, new graph, new rank, etc).

A similar procedure is carried out but from the worst actions (worst qualification) to the best ones. These two ways of ranking are called “distillations”. The final ranking is obtained through the average of the ranks of the two intermediate distillations.

Multi-criteria analysis helps the end-user to rank strategies, and consequently to make decisions. However, it should be clearly repeated that the weights and the thresholds involved in the use of the method remain partly subjective (i.e. depend on the end-user’s priorities, perception, etc.) and could be of great influence in the final results. In consequence, the proposed solutions have to be reviewed critically before making the final decision.

2.7 Remarks for application of the methodology

In the following chapters, the above methodology will be applied to the semi-virtual case study, with additional information about the methodology, in order to illustrate how it can be applied practically. It will show also that the availability of information, data, measurements and calibrated models, which is independent from the methodology itself, is a crucial aspect. If weak or incomplete information is used, weak and biased results and outputs will be provided. In other words, and this is not new, “garbage in, garbage out”. It is not meaningful to apply elaborate multi-criteria methods if the necessary and correct information has not been collected and validated before.

3 Description of the semi-virtual case study

The above methodology has been applied, for demonstration purposes, on a semi-virtual case study inspired from the catchment of Ecully, in Lyon, France. It is important here to repeat again that the conclusions from this case study shall not be considered as representative of the true reality, but only as an example of application. Only the methodology can be transposed and adapted to other cases.

3.1 Presentation of the components

Ecully is an urban catchment of 245 hectares located in the western suburbs of Lyon (De Bénédittis, 2004). The habitat is mostly residential and the imperviousness of the catchment is evaluated to 46 %.

The number of inhabitants in the territorial division of Ecully has been estimated to 7700 inhabitants during the last census in 1999. However, the exact number of inhabitants in the experimental catchment is unknown because i) it does not cover the entire territorial division of Ecully and ii) daily fluctuations are observed due to the presence of universities on the experimental catchment.

The drainage system is a combined sewer system with a 0.027 m/m mean slope. This catchment is intensively studied as part of the OTHU project (field observatory in urban hydrology), which has been created in 1999 in order to acquire field data for research purposes in urban hydrology (Bertrand-Krajewski *et al*., 2000). This catchment has also been used for other aspects of the APUSS project. Flowmeters, sensors and automatic samplers to measure pollutant concentrations and loads are installed at the outlet of the catchment, which is connected to one of the main trunk sewers of Lyon.

Consequently, there is no wastewater treatment plant at the outlet of the catchment. For the purpose of this case study, a virtual WWTP has been introduced, designed and simulated. In order to limit the impact of CSOs, a virtual CSO storage tank has also been introduced in the case study. All components are schematized in Figure 7, with some basic elements of information.

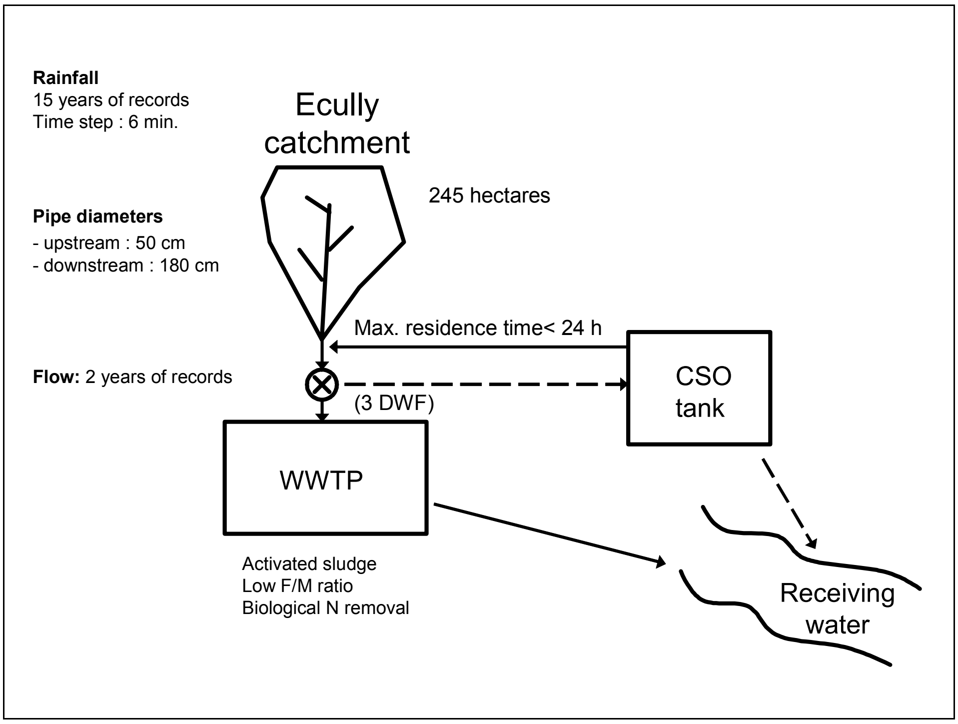

Figure 7 : Components of the semi-virtual case-study.

The case-study may appear as quite simple. However, its complete description and its modelling to illustrate the application of the proposed methodology require a lot of information. When adequate or detailed information is not available, simplifications and hypotheses have been made, which do not affect the application of the methodology itself but which may lead to some possible bias in the numerical values and in the conclusions.

Available data from the OTHU monitoring station and from measurement campaigns are :

- 2 years of flow data at the outlet of the catchment, with a 2 minutes time step.
- 15 years of rainfall data from the nearest raingauge located in Champagne-au-Mont-d'Or, with a 6 minutes time step.
- 8 measurement campaigns carried out during dry weather, with hourly data on suspended solids, COD and TKN concentrations.
- 15 measurement campaigns carried out during storm weather (Musso, 1997), with data on suspended solids, COD and BOD concentrations.

Both the rainfall – runoff process over the catchment and the flow propagation in the sewer network are simulated by initial and continuous rainfall losses functions and by means of a single linear reservoir, with a 6 minutes time step. The model has been calibrated with 10 rainfall events and validated with 5 other events; initial and continuous losses are quite important: respectively 1 mm and 60 % of the total cumulative rainfall height, which means that many impervious surfaces are not connected to the sewer system (see Annex 1 for details of the calibration). These values are specific for the Ecully catchment but are quite high for the purposes of a demonstration case-study. The continuous losses have been finally set to 30 % of the total cumulative rainfall height in order to work with a more usual case-study.

The typical mean foul sewage flow Q_{fs} (i.e. without any infiltration) has been calculated based on observed dry weather days and infiltration measurement campaigns carried out in year 2003. Q_{fs} corresponds to a volume of 1901 m^3/day (see Annex 3 for details).

The WWTP design and its functioning have been simulated by means of the ASIM software developed by EAWAG (Gujer and Larsen, 1995). It is an activated sludge plant with biological nitrogen removal (nitrification-denitrification). The design has been calculated for a maximum inflow Q_{max} equal to 3 times the mean dry weather flow DWF, for an influent design standard temperature of 13 °C and for a maximum nitrogen concentration in the effluent equal to 10 mg N/L (i.e. 83 % removal) which leads to a COD removal rate equal to 94 % (see Annex 2 for details). The nitrate recirculation flowrate is limited to maximum 4 times the dry weather flow as recommended in the French design standards.

During dry weather days, simulations are carried out with a 1 hour time step and then recalculated for daily time step. During wet weather days (i.e. days during which at least one storm event occurs), simulations are carried out with a 6 minutes time step and results are then recalculated for hourly and daily time steps. Annual influent temperature was assumed to follow a sinusoidal function over time, which was calibrated from the mean daily effluent temperatures of 80 different days. Mean annual effluent temperature was found to be equal to 17.4°C. The amplitude of the sinusoidal function is 10.4°C (minimum temperature of 12.2°C in February and maximum temperature of 22.6°C in August). As the observed mean temperature is higher than the design temperature, one may expect higher nitrogen removal rates than design removal rates.

In case of overflow (i.e. when the total flow during storm events from the catchment exceeds 3 DWF), the excess flow is temporarily stored in the CSO tank. If the CSO tank is full, the effluents are discharged directly into the receiving water. The water stored in the tank is then pumped towards the WWTP, provided its storage time has been less than 24 hours, in order to avoid septicity problems and related dysfunctioning in the WWTP. If the storage time is longer than 24 h, the water is discharged into the receiving water. These rules have been defined to simplify the case study. For real systems, other and more complex rules may be defined. All simulations are carried out with a 6 minutes time step and results are then recalculated for hourly and daily time steps.

Considering the available data and the models used, COD loads and total nitrogen have been accounted for in simulations for all components.

Seven simulation series have been carried out over the complete 15 years period for infiltration ratios IRs equal to 0, 25, 50, 75, 100, 150 and 200 % of the foul sewage flow. The reference simulation series (or scenario A) is the series calculated for IR = 0 % and with the corresponding designed WWTP.

The results for each simulation series are :
- financial criteria : investment costs (Euros/year), operation and maintenance costs (Euros/year).
- environmental criteria : COD and total nitrogen loads discharged into the receiving water from both the WWTP and the CSO tank (kg/year).

3.2 Simulation results

In this paragraph, simulation results for the reference situation A are summarized.

3.2.1 Annual rainfall variability

The annual rainfall heights corresponding to all rainfall events in the 15 years data series are given in Figure 8. The mean annual height is equal to 500 mm, which was significantly lower than the average annual precipitation height of 850 mm calculated by the Water Department of the Urban Community of Lyon (named hereafter the Greater Lyon) according to its 27 rain gauges between 1987 and 2001.

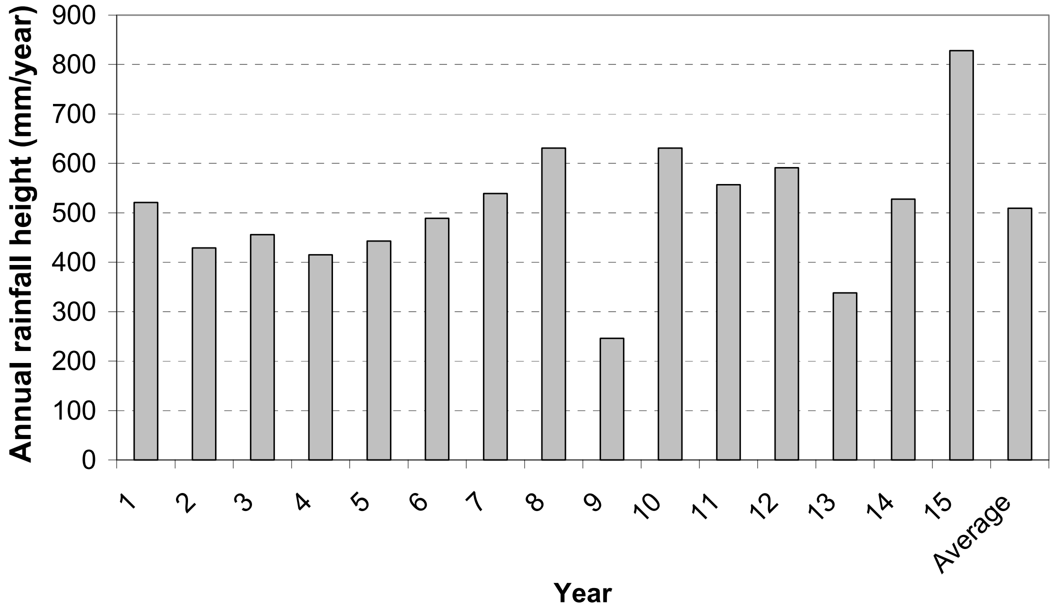

Figure 8 : Annual rainfall heights in mm used for simulations.

This large difference is mainly due to the fact that the 15 years series of rainfall records used for our simulations has been elaborated by the Greater Lyon by removing many rainfall events which were considered as not strong enough to generate overflows in the sewer system. In other words, the series has not been elaborated to simulate the rainfall events but to simulate only the events which may have clear hydraulic impacts on the sewer system. Consequently, this bias in the rainfall data series leads to an underestimation of the total volumes and overflow during wet weather periods, compared to the ideal situation where all storm events would have been kept in the data base.

3.2.2 Foul sewage and total wastewater flows and pollutant loads

The typical foul sewage flow Q_{fs} (i.e. without any infiltration) has been calculated as the average of observed dry weather days (see Annex 3 for details). Its variation during a 24 hours period is given in Figure 9: it corresponds to a volume of 1901 m^3/day. This daily foul sewage flow pattern has been kept constant for all 15 years of simulation. From this curve, hourly flow rate values are calculated for dry weather days simulations of the WWTP.

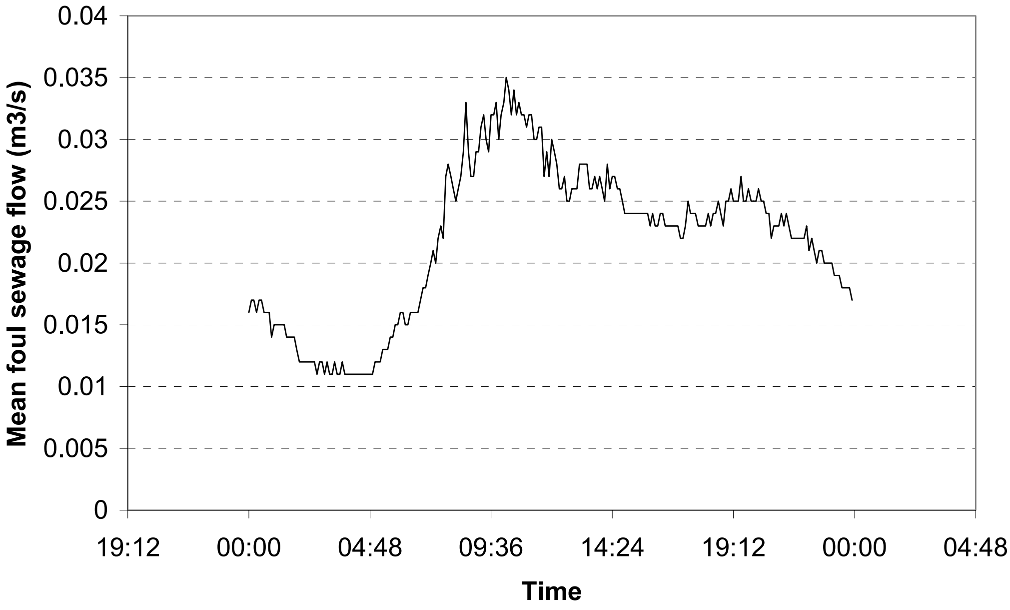

Figure 9 : Mean daily foul sewage flow pattern (m^3/s).

The total flow Q_T is the sum of the foul sewage flow Q_{fs} and of the runoff flow Q_{run} calculated from the rainfall data by means of the linear reservoir. The distribution of the volumes at the outlet of the catchment is shown in Figure 10.

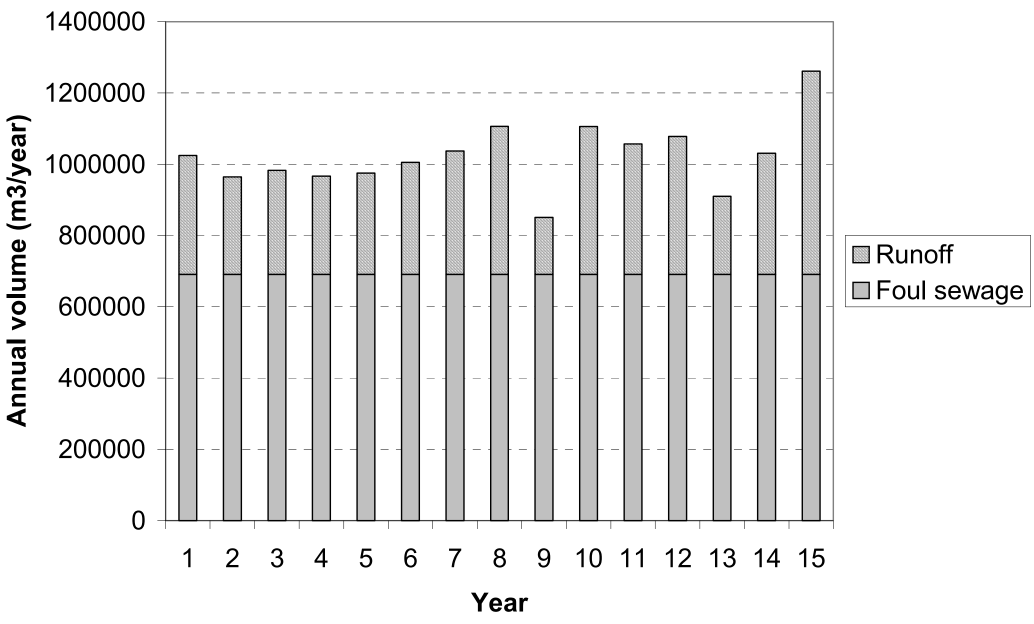

Figure 10 : Distribution of runoff and foul sewage annual volumes.

Annual volumes at the outlet of the catchment are varying from one year to another one depending on the annual rainfall height. Runoff volumes contribute from 19 to 45 % of the total annual volume.

The mean pollutant concentrations for dry weather periods have been calculated by averaging the results from 8 dry weather measurement campaigns. For wet weather periods, mean pollutant concentrations have been established from 15 measurement campaigns carried out in Ecully by Musso (1997). These mean concentrations are given in Table 2.

Mean concentrations	Foul sewage	Surface runoff
COD (mg O_2/L)	519	224
TKN (mg N/L)	59	19

Table 2 : Mean COD and TKN concentrations in foul sewage and runoff.

They have been kept constant for all simulations. This strong hypothesis is of course not realistic, as it is known and observed that pollutant concentrations widely vary from event to event and also within storm events. However, these variations were difficult to simulate adequately during this case-study as it is very difficult to calibrate a storm event pollutant model with limited data sets (there is an independent research program on this particular topic, see e.g. Mourad *et al.*, 2004). It has been decided here to keep the case study simple and to use constant values to avoid the introduction of many additional hypotheses that would have been hard to justify or validate. This may affect the final results, but very likely not their orders of magnitude.

Total pollutant loads are determined by summing foul sewage loads and surface runoff loads. Due to the low concentrations during wet weather, the foul sewage contributes annually to 74 to 91 % of the total COD loads and 79 to 93 % of the total nitrogen loads.

3.2.3 CSO tank

During dry weather, no overflow occurs because the total flow is lower than the maximum WWTP design flow (Q_{max} = 3 x DWF). As a consequence, on an annual basis, 64 to 87 % of the total volume is treated by the WWTP, as shown in Figure 11.

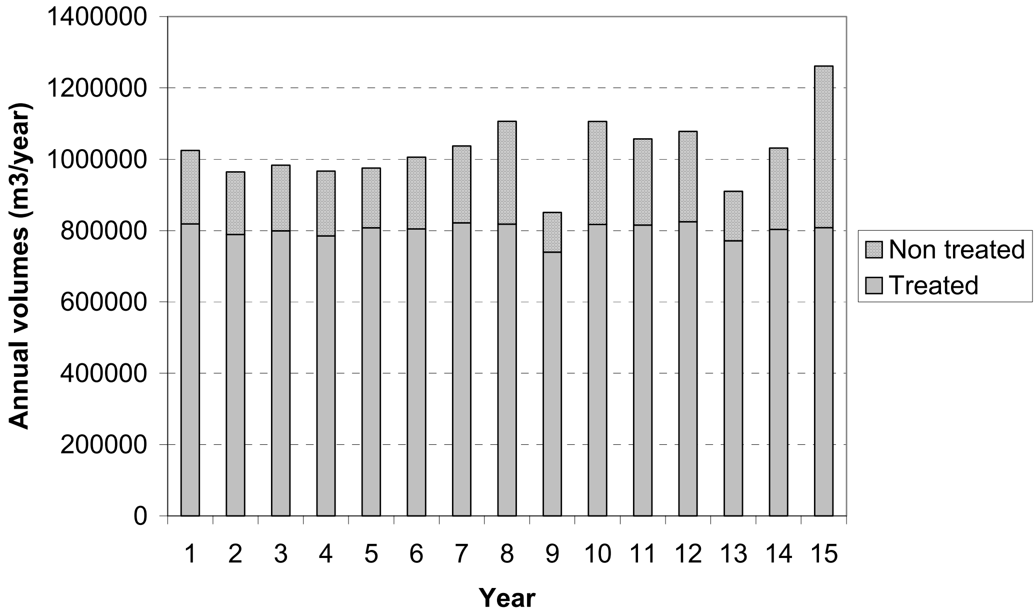

Figure 11 : Distribution of annual treated and non treated volumes.

The annual volume treated by the WWTP is stable from one year to another. The same pattern is observed for the pollutant loads. Consequently, the operation of the WWTP and the impact of the infiltration on the WWTP have been simulated only for one year, assumed to be representative of the 15 years period.

However, the total non treated pollutant load varies significantly from one year to another one : the non treated COD load discharged in the year 15 is four times higher than the load discharged in the year 9 and two times higher than the average year. In order to take into account this significant variability, three years have been analysed more specifically in the following paragraphs :
- year 1 as an average rainfall year (521 mm/year)
- year 9 as the minimum rainfall year (246 mm/year)
- year 15 as the maximum rainfall year (828 mm/year).

3.2.4 Wastewater treatment plant (WWTP)

The WWTP has been modelled using the ASIM software developed by EAWAG (Gujer and Larsen, 1995). The choice of an appropriate model for WWTP design and operation was made in close collaboration with EAWAG. There was no explicit modelling of the secondary clarifier, but it might be interesting to model it in a future study in order to observe its behaviour especially during storm events.

The analysis of all days over the year shows that the efficiency slightly decreases during wet weather days. The annual distribution and pathways of pollutant loads are shown in Figure 12 for COD and in Figure 13 for nitrogen.

Regarding COD loads, 14 to 27 % of the loads produced over the catchment are discharged into the receiving water. In average, two thirds are directly discharged by overflows without any treatment and one third is discharged by the WWTP.

Regarding nitrogen loads, 24 to 34 % are discharged into the river. These higher percentages are due to the lower efficiency of the WWTP regarding nitrogen removal. In average, one third is directly discharged by the overflows without any treatment and two thirds are discharged by the WWTP.

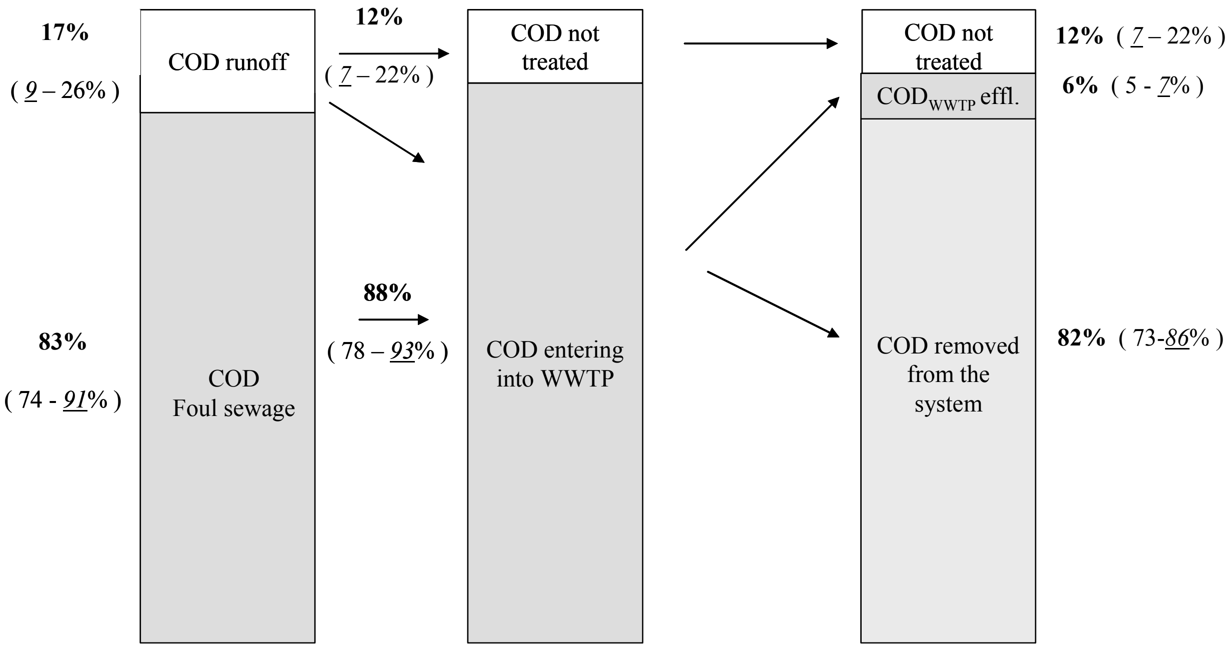

Figure 12 : Distribution and pathways of the annual COD load through the WWTP.
"COD$_{WWTP}$ effl." means the COD load discharged by the WWTP into the river after treatment.
In bold: year 1 (average); in italic: year 9 (minimum); in normal characters: year 15 (maximum).

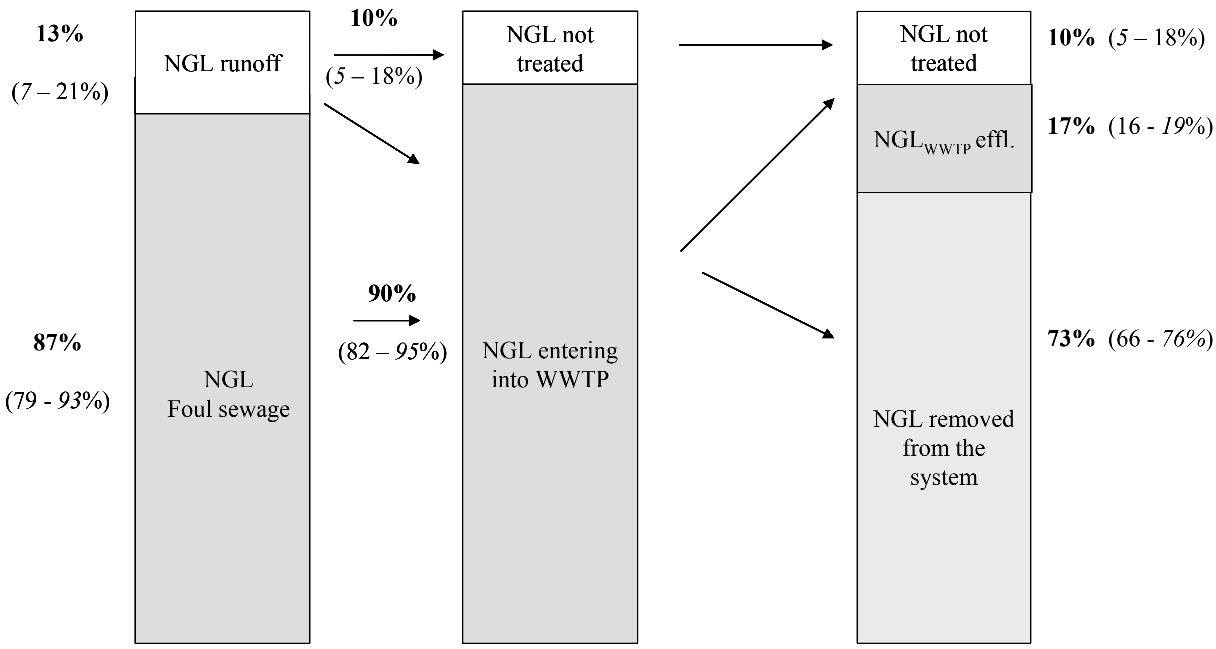

Figure 13 : Distribution and pathways of the annual NGL load through the WWTP.
"NGL$_{WWTP}$ effl." means the NGL load discharged by the WWTP into the river after treatment.
In bold: year 1 (average); in italic: year 9 (minimum); in normal characters: year 15 (maximum).

The next chapter describes of the effects of increasing the infiltration ratio IR on the above initial scenario without infiltration.

4 Impacts of infiltration

The main hypothesis is that infiltration water is clean water which does not contribute to the pollutant loads received by the WWTP. It only increases the total volume. For this study, it has been assumed that infiltration likely occurs only between January and July, which is corresponding to the high groundwater level period. The infiltration flow Q_{inf} is expressed as the infiltration ratio IR which corresponds to the percentage of the foul sewage flow Q_{fs} : IR = 100 Q_{inf}/Q_{fs}. The total flow Q_T at the WWTP inlet is equal to Q_{run} + (1+IR/100) Q_{fs}. Six series of simulations were carried out with IR values equal to 25, 50, 75, 100, 150 and 200 %.

In this case study, the catchment and the sewer system are simulated as a whole. All overflows are supposed to occur only at the outlet of the catchment, which is a rather realistic hypothesis for the Ecully catchment where the 5 small overflow structures located in the upstream parts of the sewer system are almost never in function, except for very exceptional storm events. Consequently, all pollutant loads (but of course not volumes) at the outlet of the catchment are the same as for scenario A without infiltration.

4.1 Impacts on the CSO tank

During dry weather, the total flow Q_T is lower than Q_{max} until IR = 100 % (Figure 14). For IR = 150 % and 200 %, Q_T is higher than Q_{max} for several hours per day.

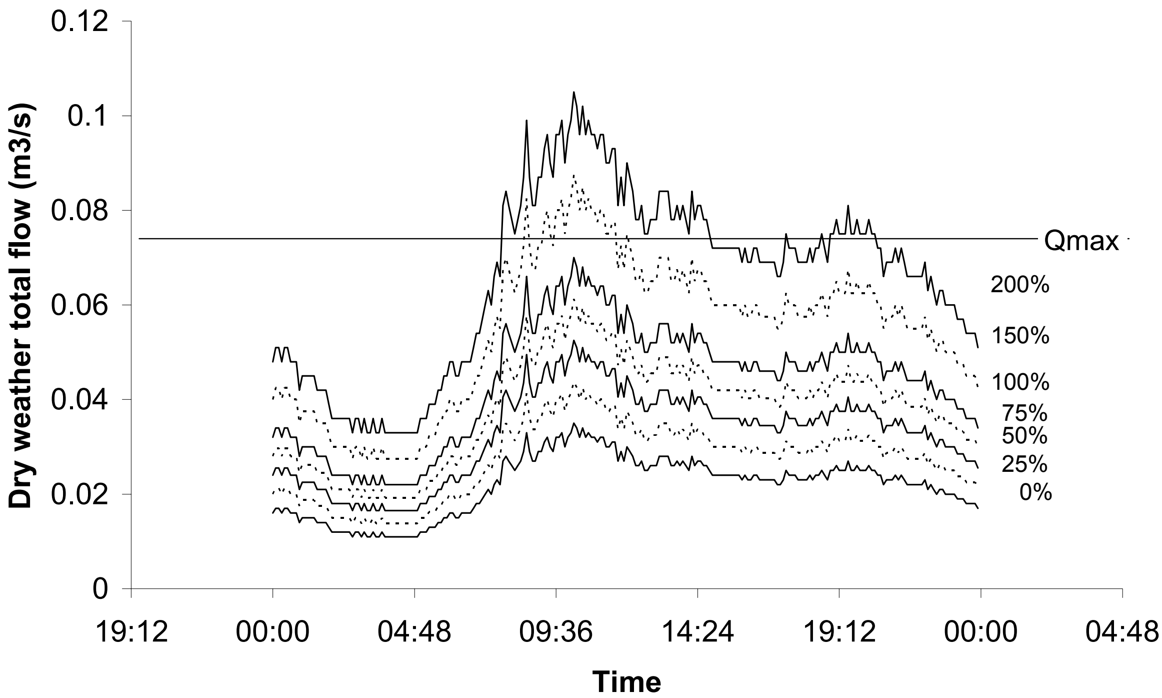

Figure 14 : Dry weather flow patterns for IR = 0, 25, 50, 75, 100, 150 and 200 %.

In case of IR = 150 %, the volume overflowed into the CSO tank during dry weather is redirected to the WWTP during lower inflow periods : all water is treated. But in case of IR = 200 %, the CSO tank can not be fully emptied and after 24 h, according to our simulation rules, the remaining (but here negligible) volume (typically 2 m^3/day) is discharged directly into the receiving water.

During wet weather, the time required to fully empty the tank is higher because both volumes transferred in sewer pipes and volumes overflowed into the tank are higher. The volume of non treated wastewater discharged into the receiving water is consequently increasing when IR is increasing. On an annual basis, the total non treated COD load is multiplied by a factor ranging from 1.08 (year 15) to 1.33 (year 1) in the case IR = 200 %, as shown in Figure 15. As expected, infiltration has less impact on the total non treated pollutant load discharged into the receiving water when the rainfall height is more important (year 15).

For all the different years, the non treated loads increase linearly until IR = 100 %, but exponentially from IR = 150 %. The same trend is observed for the total non treated nitrogen loads

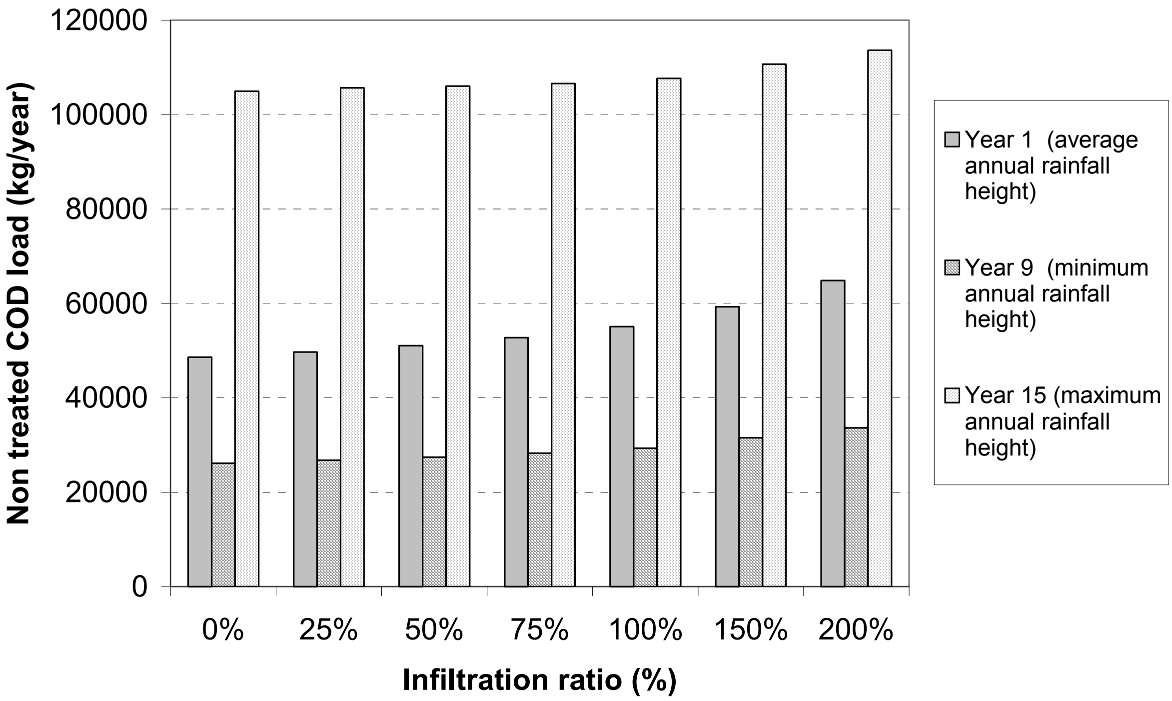

Figure 15 : Non treated COD load (kg/year) discharged into the receiving water vs. the infiltration ratio IR and different years.

4.2 Impacts on the WWTP

4.2.1 Pollutant loads

As it is assumed that infiltrated water does not bring any pollutant loads, only inflow volumes are modified by infiltration, which corresponds to dilution, as shown in Figure 16.

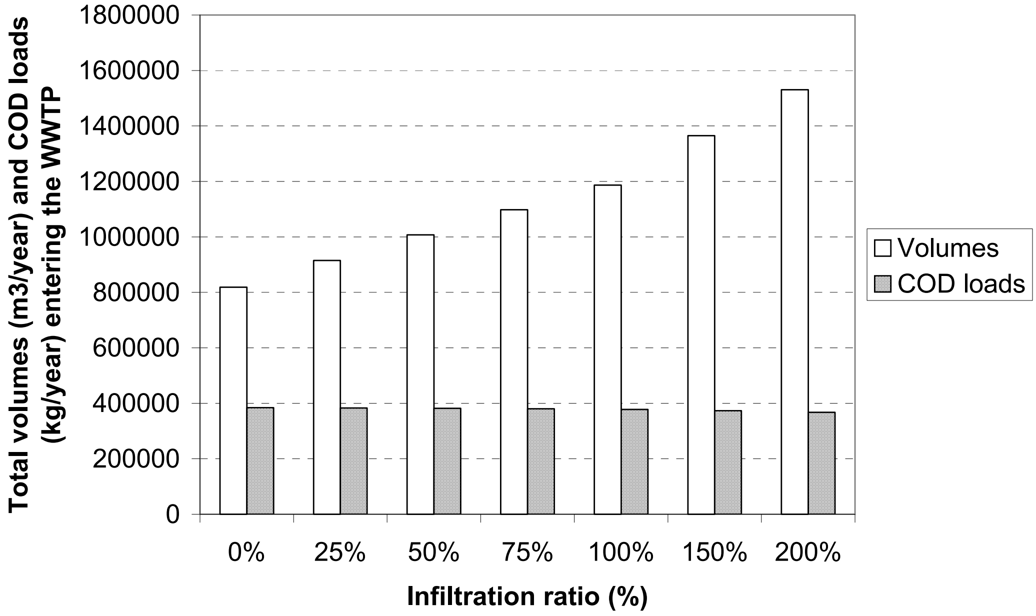

Figure 16 : Total volumes and total COD loads entering the WWTP vs. the infiltration ratio.

This dilution of the influent and the decrease of residence time in biological tanks significantly affect the efficiency of the WWTP, especially regarding nitrogen, as shown in Table 3. Consequently, the annual total pollutant loads (sum of non treated loads and loads discharged by the WWTP) discharged into the receiving water increase with infiltration (see Figure 17 and Figure 18).

IR (%)	Efficiency for COD (%)	Efficiency for N (%)
0%	94%	83%
25%	94%	81%
50%	94%	80%
75%	94%	78%
100%	94%	76%
150%	94%	75%
200%	94%	72%

Table 3 : WWTP removal efficiency for COD and nitrogen vs. the infiltration ratio.

As the WWTP has been designed for high nitrogen removal, the COD removal remains high and is not significantly affected by increasing infiltration ratios. The COD load discharged by the WWTP remains stable. However, the total COD load discharged into receiving water increases significantly due to the increase of the non treated loads (Figure 17).

Regarding nitrogen, there is a more pronounced decrease of the WWTP efficiency, but a less significant direct discharge of non treated nitrogen into the receiving water (Figure 18).

The corresponding total annual pollutant loads values are given in Table 4 for the three selected years.

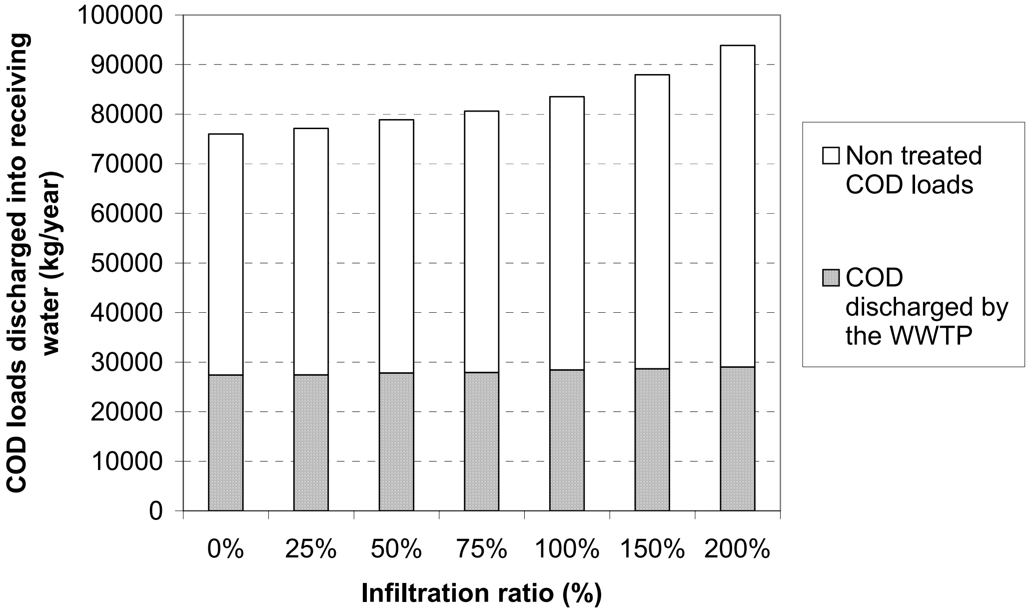

Figure 17 : COD loads discharged into the receiving water vs. the infiltration ratio (year 1).

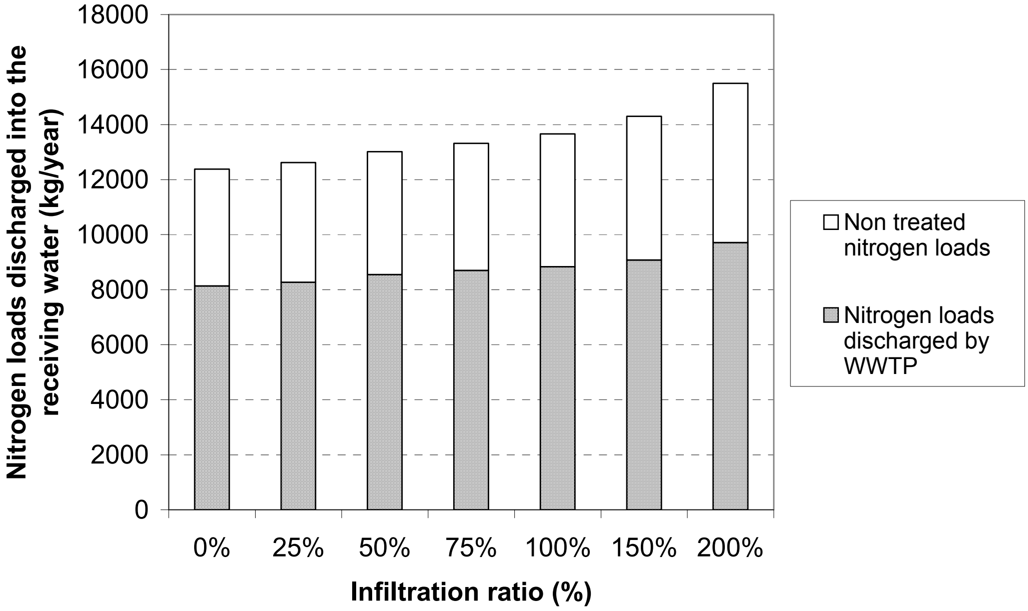

Figure 18 : Nitrogen loads discharged into the receiving water vs. the infiltration ratio (year 1).

YEAR 1				
IR	Total COD load emitted in the receiving water (kg COD/year)	Total NGL load emitted in the receiving water (kg N/year)	Total COD load with infiltration / total COD load without infiltration (scenario A)	Total NGL load with infiltration / total NGL load without infiltration (scenario A)
0%	76005	12385	1.00	1.00
25%	77129	12617	1.01	1.02
50%	78879	13012	1.04	1.05
75%	80626	13315	1.06	1.08
100%	83503	13662	1.10	1.10
150%	87949	14298	1.16	1.15
200%	93866	15498	1.23	1.25

YEAR 9				
IR	Total COD load emitted in the receiving water (kg COD/year)	Total NGL load emitted in the receiving water (kg N/year)	Total COD load with infiltration / total COD load without infiltration (scenario A)	Total NGL load with infiltration / total NGL load without infiltration (scenario A)
0%	53537	10416	1.00	1.00
25%	54194	10605	1.01	1.02
50%	55257	10938	1.03	1.05
75%	56146	11163	1.05	1.07
100%	57723	11392	1.08	1.09
150%	60134	11838	1.12	1.14
200%	62643	12699	1.17	1.22

YEAR 15				
IR	Total COD load emitted in the receiving water (kg COD/year)	Total NGL load emitted in the receiving water (kg N/year)	Total COD load with infiltration / total COD load without infiltration (scenario A)	Total NGL load with infiltration / total NGL load without infiltration (scenario A)
0%	132346	17217	1.00	1.00
25%	133100	17417	1.01	1.01
50%	133858	17723	1.01	1.03
75%	134493	17926	1.02	1.04
100%	136078	18157	1.03	1.05
150%	139318	18676	1.05	1.08
200%	142611	19612	1.08	1.14

Table 4 : Increase of annual pollutant loads discharged in the receiving water vs. the infiltration ratio.

Based on values given in Table 4 and by accounting for both measurement and model uncertainties, one may consider, for the multi-criteria analysis to be carried out later on, that differences in total annual pollutant loads are not significant until the difference reaches 5 %. In other words, for IR values lower than 75 % for year 1 for example, there is no meaningful change in total annual nitrogen and COD loads. Similarly, one assumes also that differences in total annual pollutant loads higher than 10 % have a detrimental impact on the environment.

The above threshold values, which will affect the final ranking in the Electre III method, have been chosen partly arbitrarily for this semi-virtual case study. Depending on the context, they should be modified in a real case, where uncertainties in data and modelling results should be better assessed and impacts on receiving water should also be further analysed. For example, 2000 kg/year of total nitrogen may have a negative impact on a small river whereas it can be negligible for a large river with high flows.

Nevertheless, it should remain clear that such a choice will always remain partly subjective and should be discussed between engineers, technicians and decision makers. In Annex 6, a brief analysis of the above thresholds is presented.

It also appears that impacts of infiltration vary from one year to another one depending on the rainfall variability. For example, the total annual COD loads discharged into the receiving water with IR = 200 % compared to the scenario A (without infiltration) is higher for the year 15 than for the year 1. Consequently, IR values to be exceeded to consider meaningful changes in total annual pollutant loads vary depending on the studied year, which will affect the final ranking by the Electre III method. Taking into account this variability, it may be interesting for the stakeholders to compare the results for several years before taking their decision regarding investment strategies.

4.2.2 Operation costs

Based on design requirements and pump manufacturers specifications, the pumping station at the WWTP inlet has been designed with a total head of 6 m, an efficiency of 55 % and a power of 13.5 kW. As the annual volume entering the WWTP is multiplied by 1.9 for IR = 200 % (infiltration is considered only from January to July, during the high groundwater level period), the annual energy consumption of the pumping station is also multiplied by 1.9 (Figure 19).

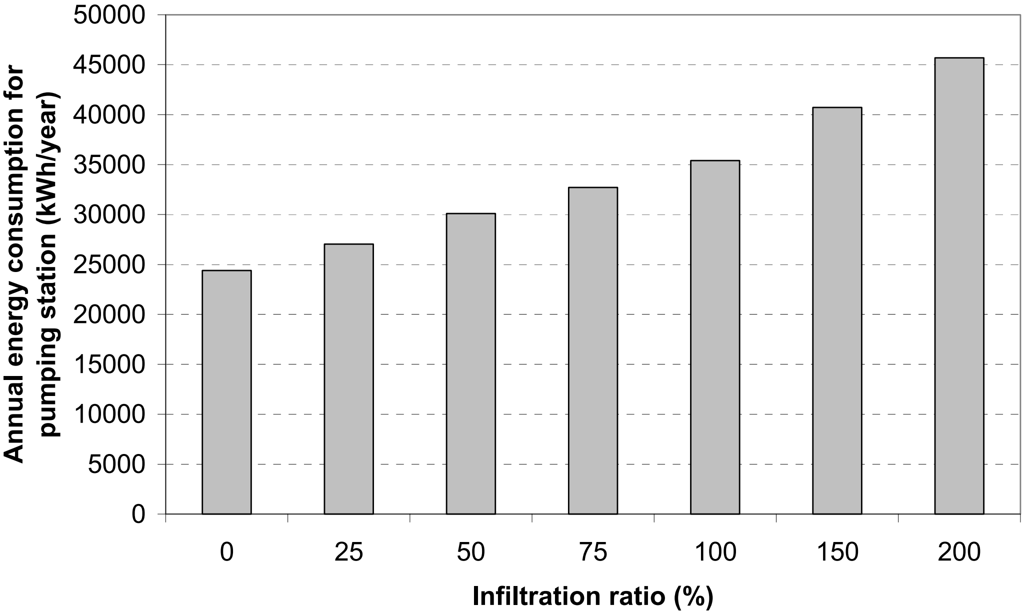

Figure 19 : Annual energy consumption of the pumping station vs. the infiltration ratio.

In the WWTP, it has been assumed that aeration is made by means of 2 fines bubbles diffuser devices, with a daily aeration time of 12 hours per day. Regarding the mixing of activated sludge, there is one mixer in the anoxic tank functioning 24 hours per day and one mixer in the aerobic tank functioning 12 hours per day. The return sludge and the nitrate recirculation pumps are assumed to be in function 12 hours per day.

With increasing values of IR, the sludge recirculation is also increasing (Figure 20 and Table 5) as it is correlated to the influent volume. Aeration and mixing durations do not increase as pollutant loads, volumes of the tanks and the F/M ratio are constant.

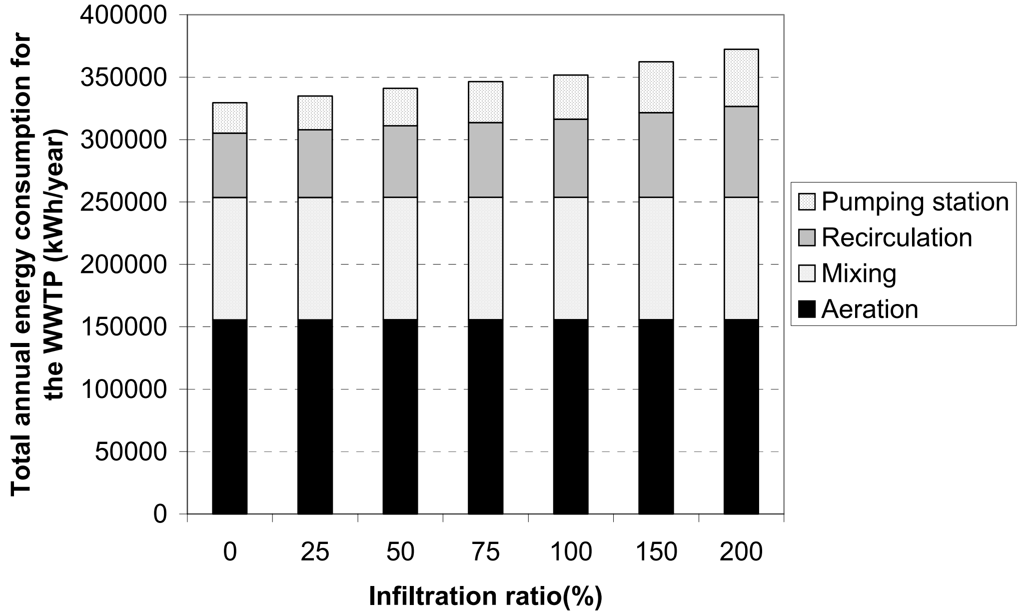

Figure 20 : Total annual energy consumption in the WWTP vs. the infiltration ratio.

IR	Energy for pumping station	Energy for aeration	Energy for mixing	Energy for recircu-lation	Total energy	Total costs (Euros/year)	Additional costs due to infiltration (Euros/year)
0%	1.00	1.00	1.00	1.00	1.00	19770	0
25%	1.11	1.00	1.00	1.05	1.02	20093	323
50%	1.23	1.00	1.00	1.11	1.04	20466	696
75%	1.34	1.00	1.00	1.16	1.05	20781	1011
100%	1.45	1.00	1.00	1.21	1.07	21100	1330
150%	1.67	1.00	1.00	1.32	1.10	21737	1967
200%	1.87	1.00	1.00	1.41	1.13	22336	2566

Table 5 : Total relative annual energy consumption (reference value = 1 for scenario A) and total annual costs in Euros vs. the infiltration ratio.

Energy costs have been calculated with a constant price of 0.06 Euros/kWh. The difference in the total annual energy consumption becomes significant from IR = 150 %. (ratio 1.10). The additional costs for the WWTP operation reach 13 % for IR = 200 %.

As the main conclusion for this case-study, for IR = 25 % and 50 %, impacts of infiltration is low on both the pollutant loads emitted by the WWTP and the energy consumption for all the years 1, 9 and 15. No rehabilitation or investment is really needed. From IR = 75 % for year 1 and 9 and from IR = 150 % for year 15, negative impacts on pollutant loads and on operation costs may lead the end-user to consider a rehabilitation of its drainage system. Of course, many rehabilitation or investment strategies may be defined and will be compared in the following chapter.

5 Definition of rehabilitation strategies

As described in paragraph 2.4, possible investment strategies should be defined by taking into account the specific context of the catchment. For example, no replacement will be carried out if excavation is not possible due to very critical traffic problems. In the considered semi-virtual case-study, no major problem limits the application of any strategy.

Three main investment strategies have been considered :
- investment in higher treatment plant capacities to account for infiltration, in order to have the same pollutant removal efficiency without infiltration,
- investment in larger CSO tank volume,
- investment in rehabilitation of the sewer system.

5.1 Investment in higher treatment plant capacities

5.1.1 Impacts on pollution

The objective is to reach an efficiency of 94 % regarding COD removal and 83 % regarding nitrogen removal. The new design made with the ASIM software is based on the dry weather flow and pollutant loads taking into account infiltration. Consequently, the capacity of the treatment plant increases and larger volumes and higher pollutant loads are accepted into the WWTP. This increase in WWTP capacity also reduces the volume of overflows emitted by the CSO tank (Figure 21).

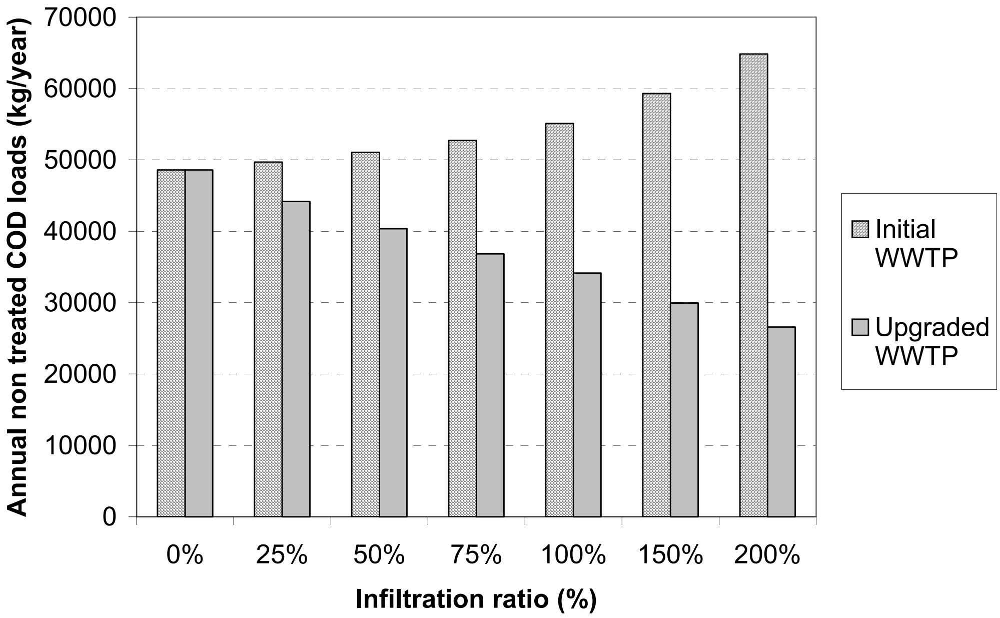

Figure 21 : Annual non treated COD load with initial and upgraded WWTP vs. the infiltration ratio (year 1).

Thanks to the upgraded WWTP, total COD loads emitted from both the WWTP and the CSO tank into receiving water are lower compared to scenario A without infiltration and with the initial WWTP (Figure 22). Similar trends are observed for the total nitrogen loads.

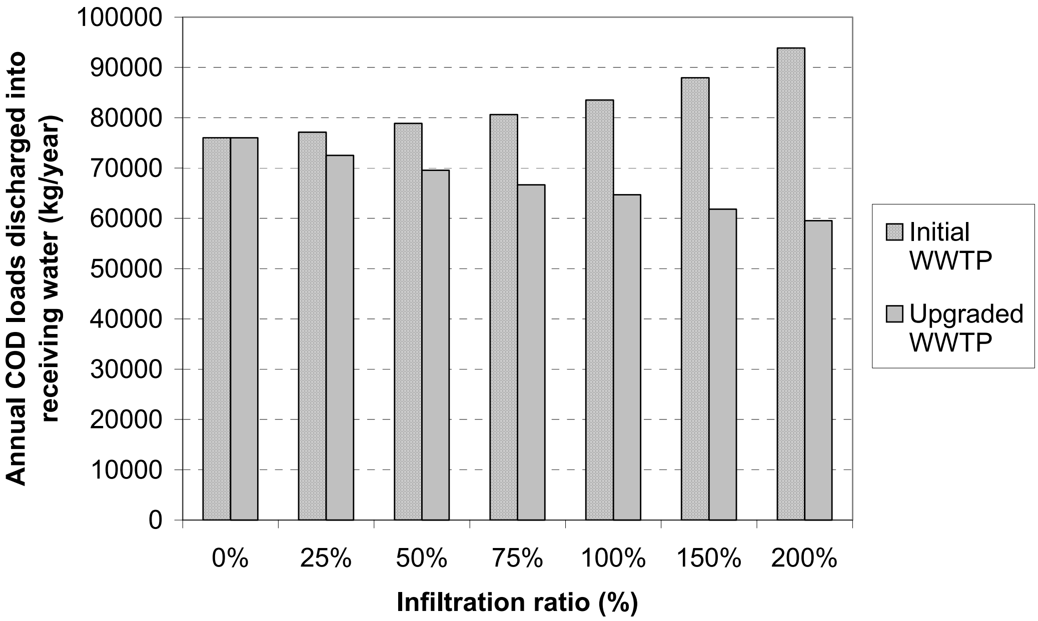

Figure 22 : Annual total COD loads emitted into the receiving water with initial and upgraded WWTP vs. the infiltration ratio (year 1).

5.1.2 Impacts on operation costs

The WWTP efficiency regarding nitrogen could be increased by at least two ways :
- increase of the residence time in the biological tank for a better nitrification by increasing the size of the tanks ;
- increase of the recirculation for a better denitrification by keeping the recirculation flow below 4 times the average dry weather flow.

The effects of infiltration can be compensated until IR = 100 % by increasing only the recirculation. This is why the size of the biological tank remains equal to 1 in Table 6. From IR = 150 %, the maximum limit for recirculation flow is reached, and the size of the biological tank size has to be increased in order to fully account for infiltration. The results of the simulations are given in Table 6. The secondary clarifier is designed based on hydraulic constraints and its size increases as a function of the increasing IR. For IR higher than 150 %, the impacts on the WWTP are very significant and obviously show that increasing the WWTP capacity is not the most pertinent solution.

IR (%)	0%	25%	50%	75%	100%	150%	200%
Biological tank volume	1.00	1.00	1.00	1.00	1.00	1.25	3.50
Secondary clarifier volume	1.00	1.14	1.41	1.69	1.98	2.08	2.36

Table 6 : Increase of the relative sizes of biological tank and clarifier to account for infiltration.

Regarding investment costs, only the biological tank and the secondary clarifier are taken into account because they are the main components affected by infiltration. For cost calculations, we have assumed that the mean cost of 1 m^3 of biological tank or secondary clarifier was approximately 600 Euros, including land, building and equipments. This cost is based on Swiss data and could be lower in other European countries.

Regarding operation costs, the results are presented in Figure 23. The energy consumption for pumping and recirculation is slightly increasing because the WWTP is designed to accept larger volumes. The energy consumption for aeration and mixing also increases because of the increase of the sludge age and of the size of the tanks.

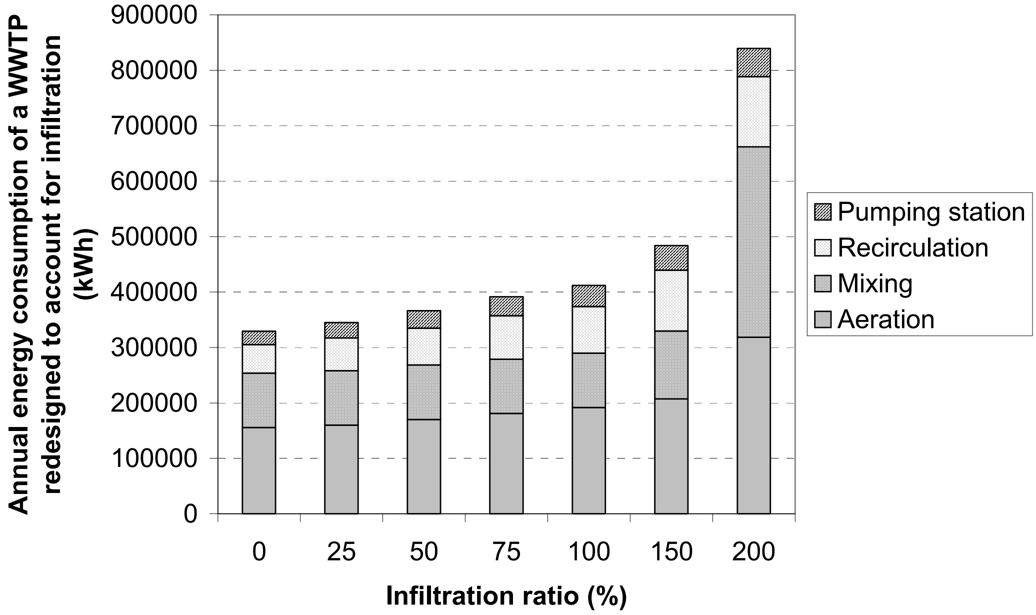

Figure 23 : Annual energy consumption of a WWTP redesigned to account for infiltration.

5.2 Investment in a larger CSO tank volume

Investing in a larger CSO tank volume reduces the annual number and volume of overflows. Various volumes of the CSO tank have been simulated for 20, 50 and 100 m^3 per active hectare (the active area is the fraction of the total catchment area which really contributes to the runoff entering into the sewer system). However, in terms of pollutant loads, the increase of the CSO tank size is effective only for low values of IR (Figure 24). These results are mainly explained by the maximum storage duration of 24 h in the CSO tank. When infiltration increases, the time necessary for emptying the CSO tank also increases. Consequently, on the one hand, investment in a higher CSO tank capacity allows the storage of larger volumes, i.e. limits the number of overflows. On the other hand, for high IR values, effluents can not be redirected to the WWTP because of high flows in the sewer system. An increasing fraction of the stored volume is thus discharged into the receiving water after 24 h of storage.

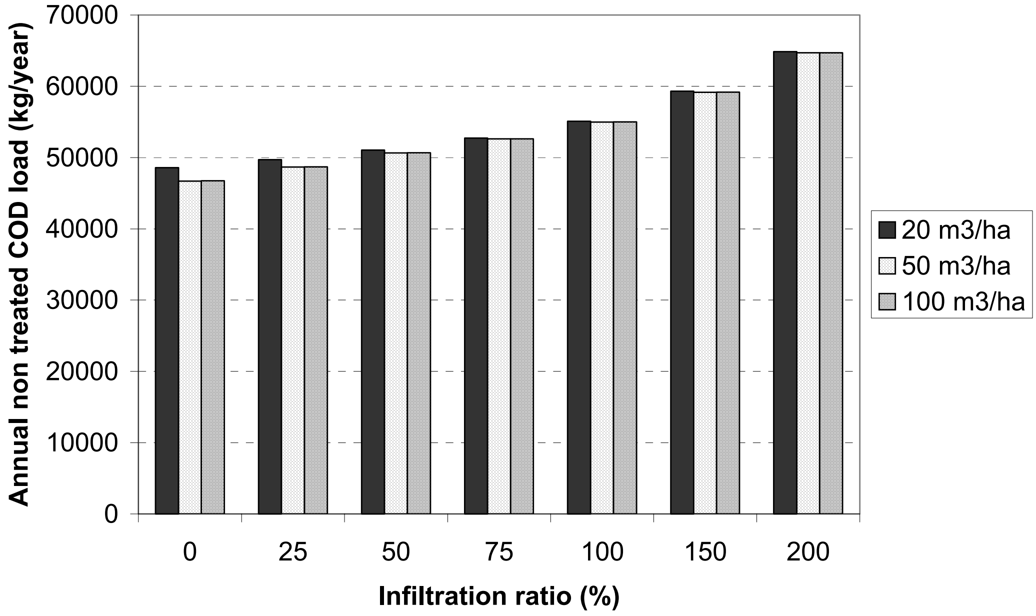

Figure 24 : Annual non treated COD loads vs. the infiltration ratio and the CSO tank volume (year 1).

The results for the years 9 and 15 follow the same pattern than the results for year 1 showed in Figure 24.

The total pollutant COD loads (from both CSO tank and WWTP) emitted into the receiving water do not significantly decrease with increasing CSO tank volumes (Figure 25).

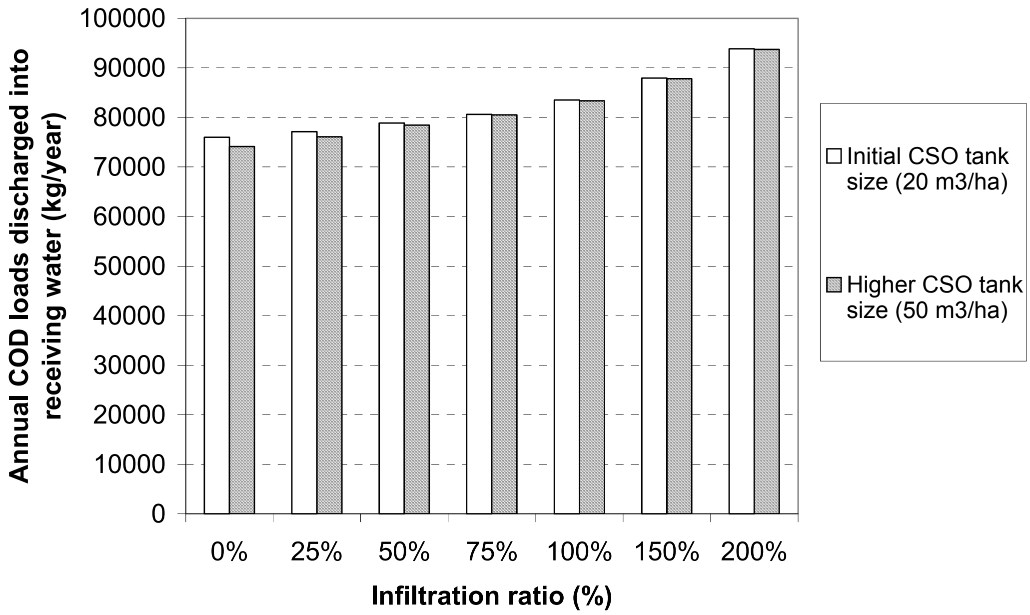

Figure 25 : Annual total COD load emitted into the receiving water vs. IR and CSO tank volume.

In this particular semi-virtual case study, it appears that investment in a larger CSO tank volume is not very promising. This investment strategy will not be further studied.

5.3 Rehabilitation of the sewer system

Four rehabilitation technologies (RT) have been selected for this case study :
- repair, which consists in local actions like injections, masonry, etc.
- sliplining : insertion of a liner, i.e. a new pipe into the existing damaged pipe (renovation).
- Cured-In-Place-Pipe CIPP : a flexible prefabricated tube impregnated with a thermosetting resin is inserted in the existing pipe and either winched or inverted into place with water pressure. Injected steam or hot water cures the resin and shapes the tube into the form of the existing pipe. Application of heat hardens the resin after a few hours, forming a jointless pipe-within-a-pipe (renovation).
- conventional trench (replacement).

Table 7 summarizes the advantages and the disadvantages of each selected RT.

Rehabilitation technology	Advantages	Disadvantages
Repair (injections, etc.)	- no excavation - not expensive	- no restoration of the site (correction of structural problems) - remaining of other defects
Sliplining (renovation)	- no excavation - restoration of the site	- capacity losses - construction inspection by CCTV
CIPP (renovation)	- no excavation - jointless pipe	- capacity losses - no entire restoration of the site - construction inspection by CCTV
Conventional trench (replacement)	- restoration of the site - construction inspection from the surface	- excavation

Table 7 : Advantages and disadvantages of some rehabilitation technologies.

In the present case-study, all selected RTs are supposed to be applicable. The structural state of sewer pipes in the Ecully catchment is not precisely known, due to non systematic CCTV inspections. Among 15 km of sewer pipes, only 1 km is man-entry. In non man-entry pipes, few CCTV examinations have been carried out, mainly for reactive actions. Only pipes which are assumed to be critical (according to the knowledge of sewermen) are inspected. Moreover, the exact location of infiltration is not really known. Facing this lack of data, the following hypotheses have been made for the case study :
- infiltration occurs only in the pipes located in the river aquifer,
- infiltration is diffuse and occurs along all pipes located in the aquifer.

With the above hypotheses, 3 km of pipes are concerned (2 km of non man-entry pipes and 1 km of man-entry pipes), which represents a rehabilitation of 20 % the complete sewer system. If these 3 km are assumed to be rehabilitated at once, this rate of 20 % is very high compared to the average rehabilitation rate in French cities which is around 2 % of the total sewer length per year.

Investment costs are difficult to estimate due to their great variations depending on the context of the catchment. However, the following average prices have been applied to the present case study :

- repair costs are given by French specialists based on experience. Costs are estimated to be 75 Euros per linear meter and 20 % of additional costs are added for planning, manpower and supervision.
- sliplining costs and conventional trench costs are estimated using data from Frehmann (2004), which depend on diameter and depth of the pipes.
- CIPP costs have been given by C. Montero from CARE-S project (extract from the CARES-S cost tool, Freni *et al.*, 2005). They depend on the diameter of the pipe and include re-opening of service connections and renovation of manhole bottom. 20 % of additional costs are added for planning and construction supervision.

For the semi-virtual case study, total investment costs for rehabilitation are summarized in Table 8.

	minimum	mean	maximum
Injections (repair)	-	1434551	-
Sliplining (renovation)	5161572	6474694	7787815
CIPP (renovation)	-	5443871	-
Conventional trench (replacement)	5257106	6806425	12413309

Table 8 : Investment costs (in Euros) for the selected rehabilitation technologies.

Regarding the two renovation technologies, CIPP appears less expensive than sliplining. Consequently only repair, CIPP and conventional trench have been selected for further analysis.

Another important hypothesis is that by rehabilitating 100 % of the sewer pipes subject to infiltration problems, 100 % of the infiltration problem will be solved. This optimistic hypothesis should be considered with extreme caution because :

- the elimination of infiltration could not be complete due to bad installation or incomplete repair,
- groundwater may find other ways to enter the sewer systems (HC, manholes, etc.).

However, we have used this simple hypothesis for convenience. Indeed, pollutant loads emitted into the receiving water and operation costs after sewer rehabilitation will be the same as in scenario A without infiltration.

As explained in paragraph 2.5, sewer system rehabilitation linked to infiltration may contribute to increase the global serviceability of the sewer system. If data and models are available, it can be expressed in decrease in flooding, decrease of silting, etc. In our case, this criterion has not been considered because many additional hypotheses would have been needed.

5.4 Mixed investment: renovation of 50 % of the pipes and investment in upgrading the WWTP

In the mixed investment strategy, only 50 % of the sewer pipes submitted to infiltration are renovated and the WWTP is re-designed in order to account for the remaining infiltration. Renovation technology has been chosen because it is an intermediate solution : it is cheaper than replacement and it improves the serviceability better than repair. One assumes that the renovation of 50 % of the pipes is eliminating 50 % of infiltration, and that investment costs for sewer pipes are 50 % less expensive than the rehabilitation of 100 % of the pipes (equal distribution of costs for all pipes). This investment appears as an intermediate solution regarding pollutant loads and investment costs.

All strategies have now been defined and will be compared in the next chapter.

6 Comparison of the strategies

6.1 Impacts of the investment strategies on each selected criterion

As observed in the above paragraphs, in the proposed case-study, infiltration significantly affects the drainage system from IR = 75 % for year 1 and 9 and from IR = 150 % for year 15. For IR = 25 % and 50 %, impacts are considered as negligible and no investment strategy has been defined.

Figure 26 to Figure 29 present the impacts of investment strategies on each selected criterion for the year 1. The conclusions described for the year 1 can also be applied to the results of years 9 and 15 given in Annex 4. Only the amplitude of the results is modified.

The operation, financial and total annual costs have been calculated for one year only because the annual volume and pollutant loads treated by the WWTP are stable from one year to another one as shown in the paragraph 3.2.3. Consequently, operation, financial and total annual costs are equivalent for all years.

From IR = 75 %, investment in higher WWTP capacities represents the best solution regarding pollutant loads emitted into the receiving water (Figure 26 and Figure 27). The COD loads discharged using WWTP investment strategy are 17 to 37 % lower than with sewer rehabilitation. Regarding total nitrogen loads, they are 13 to 33 % lower than with sewer rehabilitation. Taking the same uncertainty thresholds as in chapter 4.2.1 (5 % and 10 %), investment in WWTP appears better than sewer rehabilitation for COD and total nitrogen loads. As expected, “no investment” strategy appears as the worst solution.

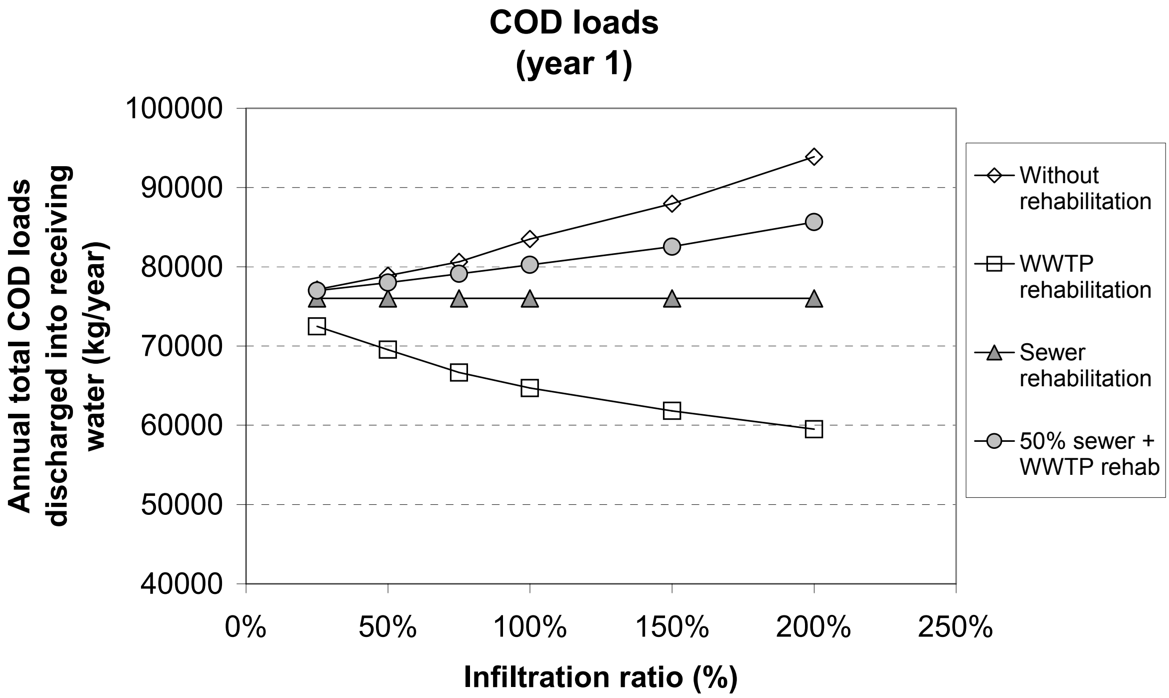

Figure 26 : Annual total COD load emitted into the receiving water vs. the infiltration ratio and the investment strategy.

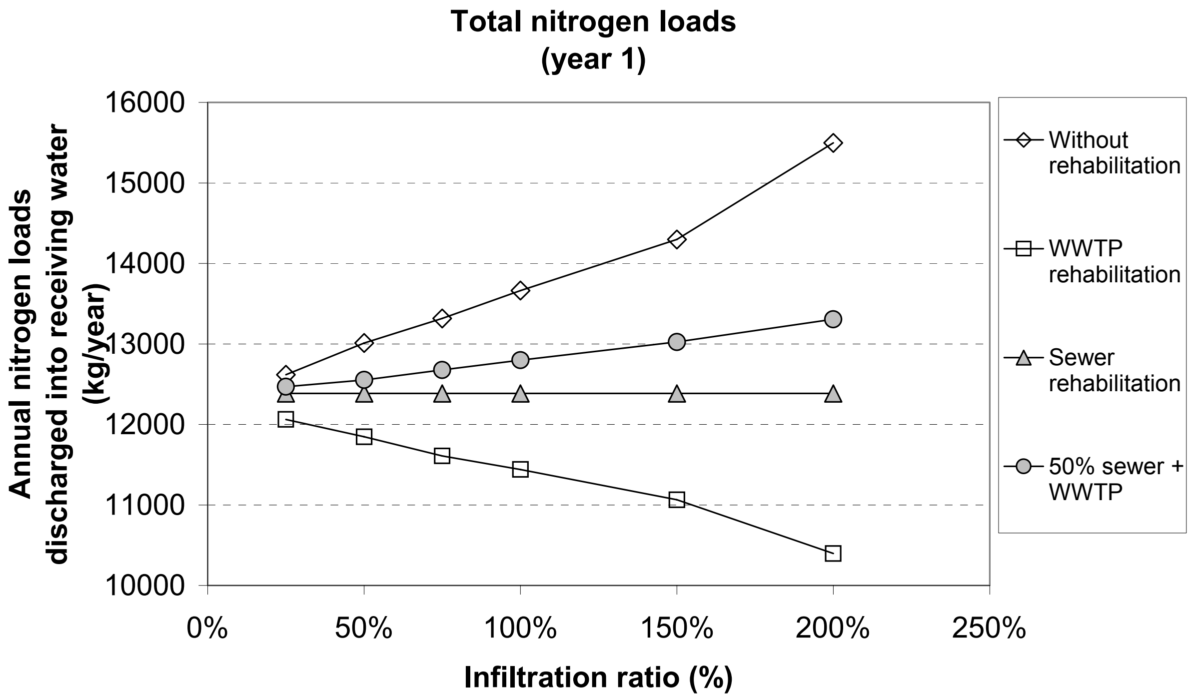

Figure 27 : Annual total nitrogen loads emitted into the receiving water vs. the infiltration ratio and the investment strategy.

Regarding operation costs, sewer rehabilitation is the cheapest solution whereas WWTP rehabilitation is the most expensive one, due to the increase of energy consumption in the pumping station and for the treatment (Figure 28).

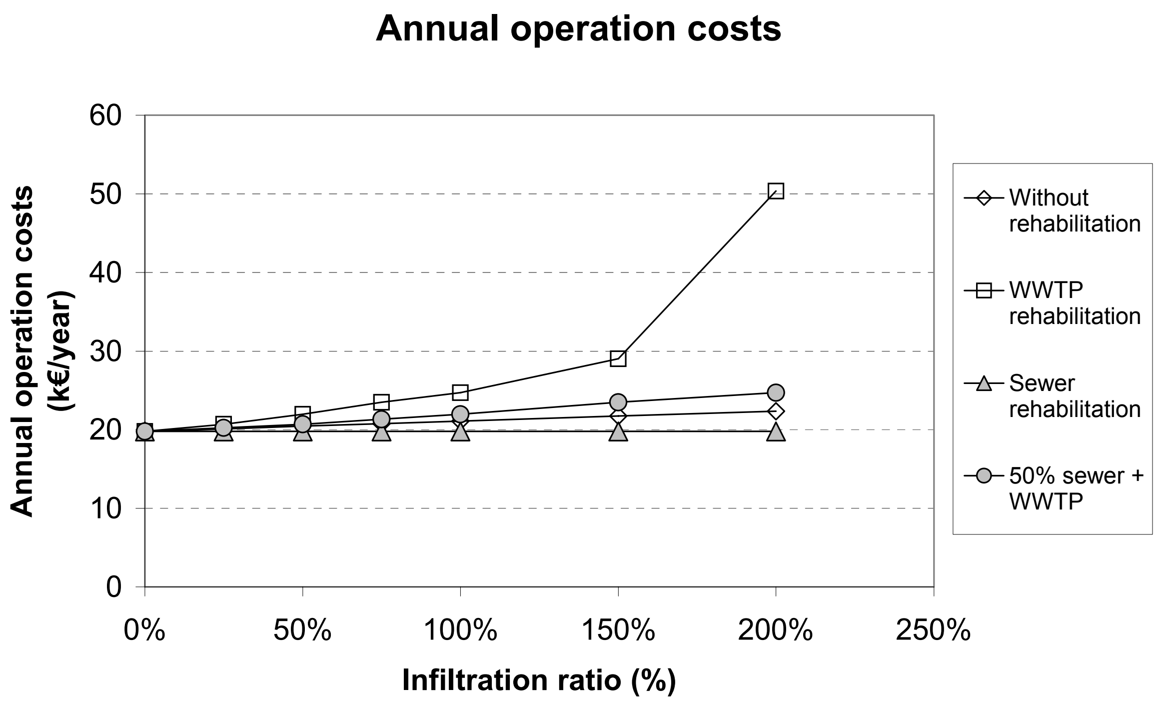

Figure 28 : Annual operation costs vs. the infiltration ratio and the investment strategy.

Regarding total investment costs, repair is the cheapest solution among the investment strategies (Figure 29). WWTP investment costs are cheaper than replacement, renovation or mixed investment until IR = 150 %, replacement being the worst solution. From IR = 200 %, investment in WWTP becomes more expensive than replacement, renovation or mixed investment solutions due to the increase of the tank volumes. Replacement remains the most expensive solution for all IR values.

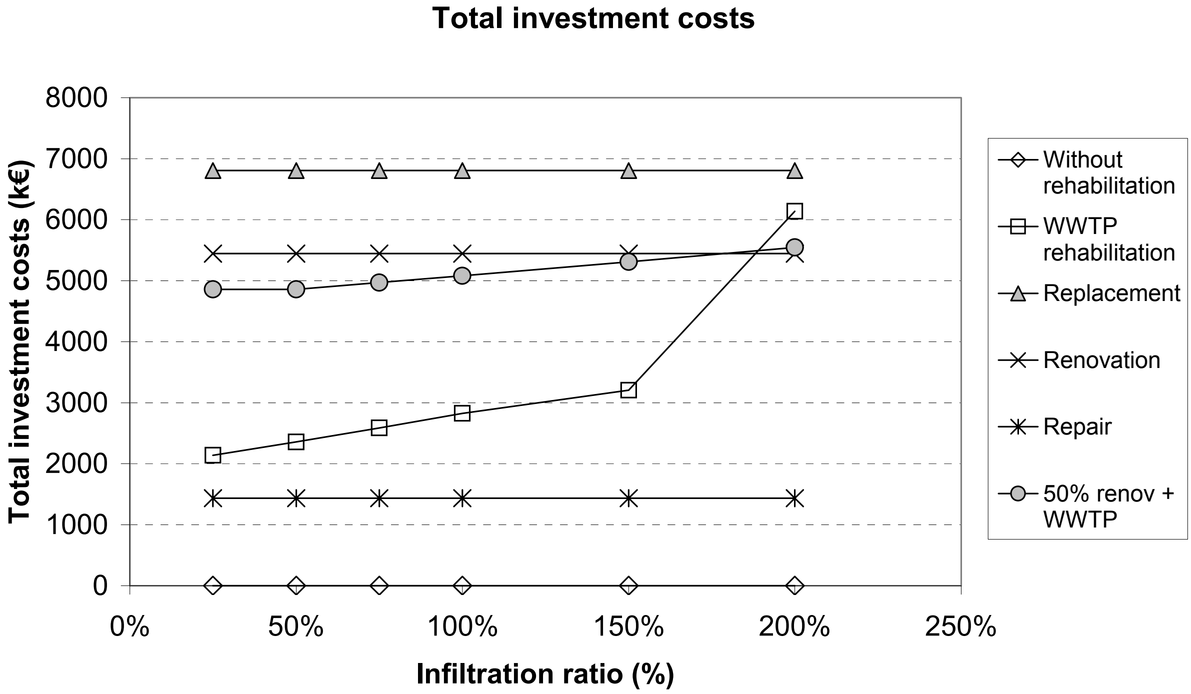

Figure 29 : Total investment costs vs. the infiltration ratio and the investment strategy.

If the total investment costs (or unit construction costs UCC) are transformed into annual unit construction costs (AUCC) based on the life cycle of the rehabilitation technology as proposed in paragraph 2.5, the annual total costs can be calculated as the sum of the AUCC and of the annual operation costs (Figure 30). The annual total costs represent the amount to be paid annually by the end-user. For the calculation of AUCC, the interest rate has been taken equal to 5 %. The expected lives of the selected rehabilitation technologies have been set to 80 years for the replacement, 50 years for the repair and renovation and 30 years for the WWTP based on the life cycle proposed in paragraph 2.5.

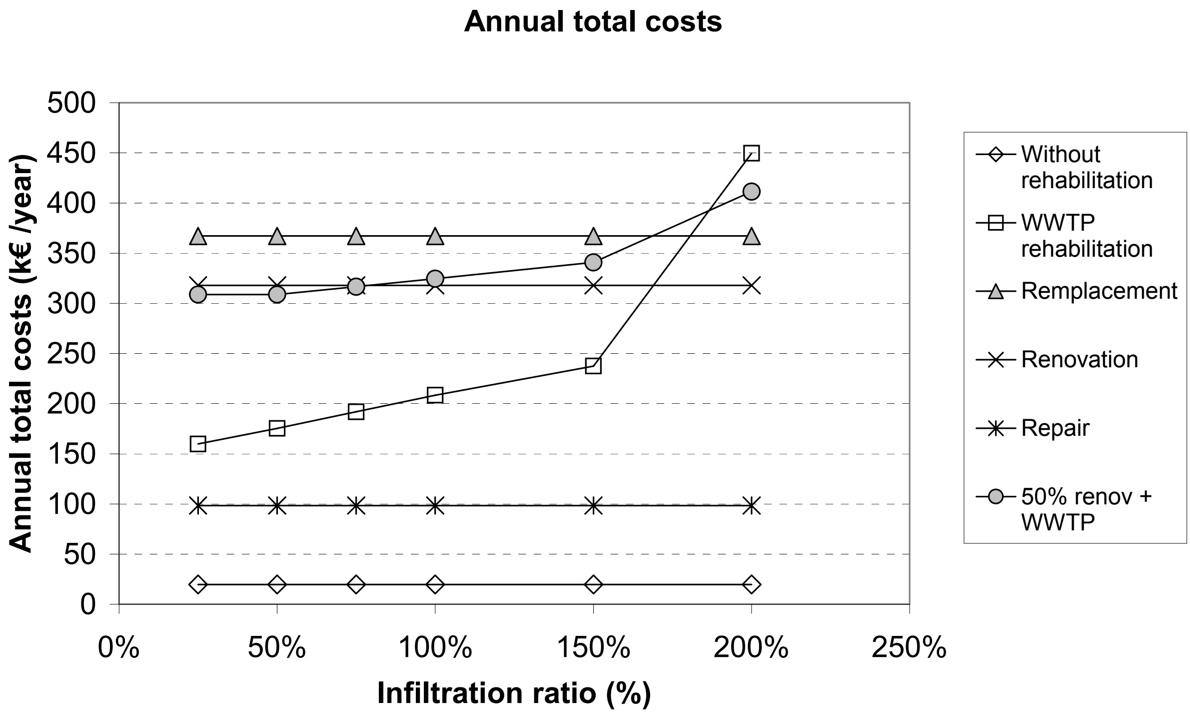

Figure 30 : Annual total costs vs. the infiltration ratio and the investment strategy.

Repair remains the less expensive solution. However, the mixed investment strategy becomes more expensive than renovation from IR = 100 % and than replacement from IR = 200 %. The WWTP investment strategy becomes the worst solution for IR = 200 %.

6.2 Multi-criteria analysis of each IR scenario

Each scenario for IR = 75, 100, 150 and 200 % has been analysed with the multi-criteria method Electre III.

The final Pareto graph for IR = 150 % is given in Figure 31. (Pareto graphs for the other scenarios are given in Annex 5).

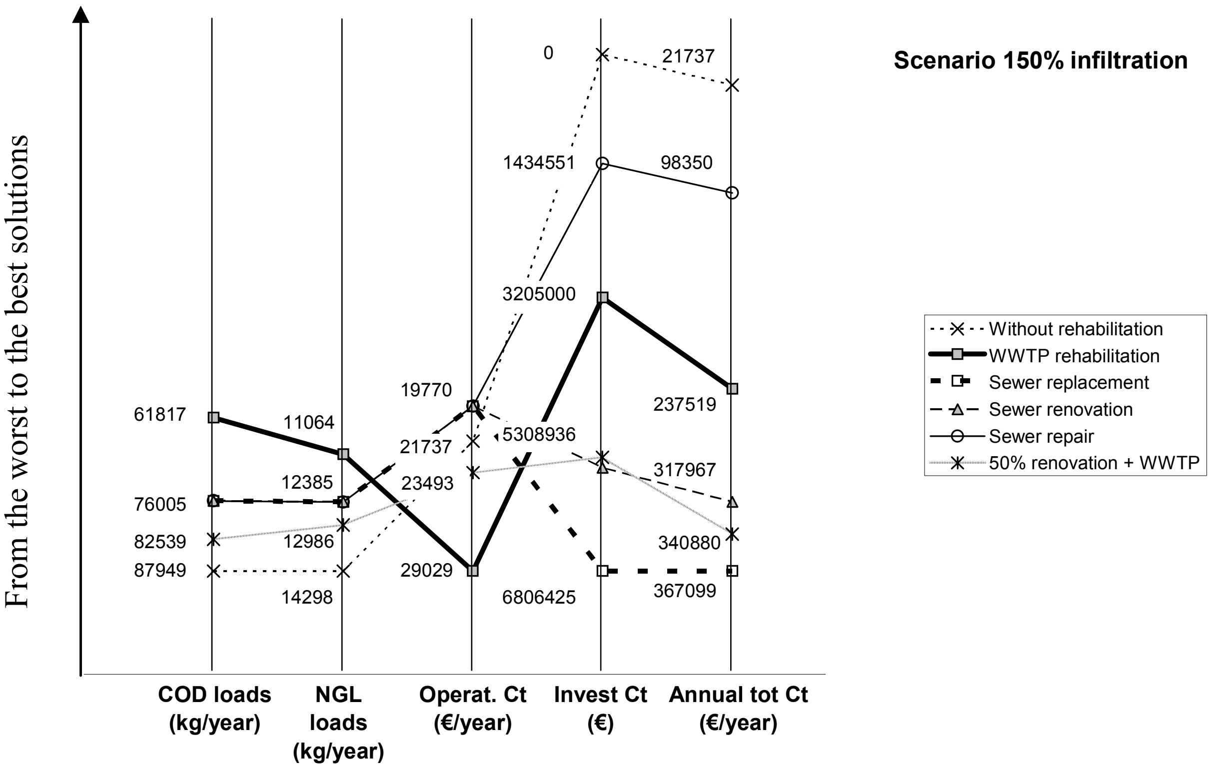

Figure 31 : Comparison of investment strategies according to the different criteria (Pareto graph). COD and NGL loads: total COD and nitrogen loads discharged into receiving water, Operat. Ct: operation costs, Invest Ct : investment costs, Annual tot Ct : annual total costs.

Figure 31 shows that “no investment” is the worst solution regarding COD loads and nitrogen loads criteria. However, it is better than WWTP rehabilitation and the mixed investment regarding operation costs and is the cheapest solution regarding investment costs and annual total costs.

WWTP rehabilitation is the best solution regarding COD and nitrogen loads but the worst one regarding operation costs. However, it is better than sewer replacement, sewer renovation and mixed investment strategies for investment and total annual costs.

Sewer replacement, renovation and repair are intermediate solutions regarding pollutant loads and the best solutions regarding operation costs. However, sewer replacement is the most expensive solution regarding investment costs and sewer renovation requires higher investment costs than WWTP rehabilitation. Taking into account the life times of the rehabilitations, sewer renovation outranks the mixed investment for annual total. Sewer repair appears as the less expensive solutions among the investment strategies.

Finally, the mixed investment is an intermediate strategy for all the criteria.

To summarize, none of the selected strategies appears as the best one for all criteria. The multi-criteria method Electre III is then used to compare and rank the different solutions.

Weights, indifference and strict preference thresholds should be defined for each criterion. Indifference threshold corresponds to the uncertainty. If, for a given criterion, the difference between two strategies is lower than the indifference threshold, the values are considered to be equivalent for this criterion. If the difference is higher than the preference threshold, one of the strategies is strongly preferred to the other one for this criterion. If the difference threshold is between indifference and preference thresholds, one strategy is slightly preferred to the other one for this given criterion.

The weights have been defined in order to equitably compare environmental and economic points of view. Of course, the choice depends on each end user and on each case.

As proposed in paragraph 4.2.1 for this semi-virtual case study, indifference and strict preference thresholds have been respectively set to 5 % and 10 % for pollutant loads.

Regarding costs, it is assumed here that they are established without uncertainty (which is not true in reality). Nevertheless, the end-user may consider, for example, that a difference of 2000 Euros per year is not significant to distinguish one strategy from another one for operation costs, and that a difference of 5000 Euros per year is very discriminating. Indifference threshold have then been set to 2000 Euros per year and preference threshold to 5000 Euros per year. Similarly, for investment costs, indifference and preference thresholds have been set respectively to 50 000 and 100 000 Euros, and to 10 000 and 20 000 Euros per year for total annual costs.

The veto threshold is an eliminatory threshold, which means that if the difference between two strategies is higher than this threshold, one of the strategies is eliminated. In our case, none of the values is considered as eliminatory. However, the veto threshold could be defined by the end-user, e.g. as a limit for a budget or as a threshold pollutant concentrations or loads which shall not be exceeded.

All above values are summarised in Table 9. An analysis of the robustness and of the sensitivity of the different thresholds and weights should be carried out in order to check if the calculated ranking is stable or not. In our case, the results appear quite stable (see detail in Annex 6).

Point of view	Environmental criteria		Economic criteria		
Criterion	Total COD load (kg/year)	Total nitrogen load (kg/year)	Operation costs (€/year)	Investment costs (€)	Annual total costs (€/year)
Weight	3	3	2	2	2
Indifference threshold	5 %	5 %	2000	50 000	10 000
Preference threshold	10 %	10 %	5 000	100 000	20 000
Veto threshold	-	-	-	-	-

Table 9 : Set of Electre III parameters without accounting for sewer serviceability.

The application of the Electre III method leads to the results given in Table 10.

	IR scenario	Ranking results
75%	Year 1	Without → WWTP → Repair → Renov → Replac
	Year 9	Without → WWTP → Repair → Renov → Replac
	Year 15	Without → WWTP → Renov → Replac Repair
100 %	Year 1	Repair → WWTP → Without → Renov → Replac → 50% R
	Year 9	Repair → Without → WWTP → Replac Renov
	Year 15	Without → WWTP → Renov → Replac Repair
150 %	Year 1	Repair → WWTP → Without → 50 % R→ Replac Renov
	Year 9	Repair → Without → Replac WWTP Renov
	Year 15	Without → WWTP → Renov → Replac Repair
200 %	Year 1	Repair → WWTP → Without → 50 % R Renov Replac
	Year 9	Repair → Without → Replac WWTP Renov
	Year 15	Repair → WWTP → Without → Replac Renov

Table 10 : Final ranking of the investment strategies after simulations using Electre III. "Without" means "no rehabilitation" and "50 % R" means the mixed investment strategy.

For IR = 75 %, the "no investment" strategy appears as the best solution for all years. It is outranked by the WWTP investment strategy but it is only slightly outranked by sewer rehabilitation regarding pollutant loads, and it is the cheapest solution regarding operation, investment and annual total costs.

The WWTP investment strategy is ranked in the second place because it strongly outranks the "no investment" strategy and it slightly outranks sewer rehabilitation regarding pollutant loads. It also strongly outranks the renovation and replacement strategies for investment and annual total costs and it is slightly outranked by sewer rehabilitation for operation costs.

For year 15, the repair strategy is also ranked in the second place because it is equivalent to the "no investment" strategy regarding pollutant loads and it is a cheap investment.

For IR = 100 %, the repair strategy becomes the best solution because it strongly outranks the “no investment” strategy regarding pollutant loads for year 1 and strongly outranks the WWTP investment strategy regarding operation, investment and total annual costs. The WWTP investment strategy remains on the second place because it strongly outranks the “no investment” strategy regarding pollutant loads.

For year 9, the repair strategy also becomes the best solution but the “no investment” and the renovation strategies are in the second place before the WWTP investment strategy. The repair strategy slightly outranks the “no investment” strategy for pollutant loads, it is equivalent to the “no investment” strategy for operation costs and it is strongly outranked by the “no investment” strategy regarding investment and annual total costs. The “no investment” strategy is equivalent to the WWTP investment strategy for pollutant loads and it strongly outranks the WWTP investment strategy regarding investment and total annual costs.

For year 15, the “no investment” strategy remains the best solution as it is equivalent to the sewer rehabilitation regarding pollutant loads.

For IR = 150 %, renovation is at the second place for year 1 as it strongly outranks the “no investment” strategy regarding pollutant loads. The WWTP investment strategy becomes equivalent to “no investment” and renovation strategies because it strongly outranks the “no investment” strategy regarding pollutant loads.

For IR = 200 %, repair becomes the best solution for year 15 as it strongly outranks the “no investment” strategy regarding COD loads and slightly outranks the “no investment” strategy for NGL loads.

The replacement strategy appears as the worst solution for all years due to its high investment and annual total costs.

As a conclusion, it appears that the ranking of investment strategies varies from one year to another one depending on the rainfall height, which may increase or reduce the impacts of infiltration, i.e. the annual pollutant loads discharged on the receiving water. During the decision making process, the end-user should remind this variability and make decisions based on several years simulations.

6.3 Discussion of the results – influence of hypotheses

Evaluation of pollutant loads

Runoff pollutants concentrations were taken constant for all storm events. In reality, these concentrations may vary significantly from one event to another one and even within one single event. Moreover, the treatment in the CSO tank (settling) and the WWTP secondary clarifier have not been explicitly modelled. If they would have been modelled, conclusions regarding pollutant loads may have been different (less pollutants discharged from the CSO tank, a little bit more from the WWTP). This is one of the limits of this simplified case-study.

The hypothesis that the rehabilitation of 100 % of the pipes submitted to infiltration leads to the elimination of 100 % of the infiltration could be further discussed. Groundwater will likely find other ways to enter the sewer system. For the same reason, the hypothesis that the rehabilitation of 50 % of the pipes will lead to a 50 % decrease of the total infiltration is even more questionable. There is not much literature on this topic. Usually, the water tightness is checked immediately after the sewer rehabilitation has been completed, but no long term observations are usually made. Heijs and Moroney (2003) showed that integrated programmes including public lines and private properties (such as house connections) lead to good results but that partial rehabilitation programmes (for example 80 % of public pipes) can lead to high investments without significant final improvements regarding infiltration. In that case, pollutant loads emitted into the receiving water remain at a high level after rehabilitation. For our semi-virtual case study, this means that the benefits expected from the rehabilitation have been very likely overestimated. Consequently, only integrated programmes (100 % of rehabilitation of the all infiltrating pipes) appear pertinent regarding infiltration problems. An investment in WWTP upgrading would probably be preferred to a partial sewer rehabilitation strategy.

Evaluation of the costs
The sewer rehabilitation rate applied in our case-study (20 % of the total length of the drainage system) is high compared to the mean usual rate of 2 % applied in French cities. By keeping the objective of 100 % of rehabilitation of infiltrating pipes, investment costs could likely be reduced by precise determination of the locations where infiltration mainly occurs. The first step consists to identify the pipes which are influenced by groundwater, i.e. pipes that are located within groundwater. Then the APUSS methods, which give results with lower uncertainties than traditional flow measurement methods, could be applied to decide which pipes have to be rehabilitated firstly. This could drastically reduce investment costs of sewer rehabilitation, especially for renovation and replacement techniques which can then appear less expensive than investment in WWTP upgrading. But this would not solve the above mentioned problem of partial rehabilitation programmes.

Up to now, the choice among possible sewer rehabilitation technologies has frequently been based on total investment costs and on the applicability of the technologies. However, their durability may have a high influence on this choice if the end-user is interested in long-term investments. Unfortunately, most of the techniques are too recent to allow a serious estimation of their expected life times (see paragraph 2.5) and rough hypotheses on their durability have to be made.

7 Conclusion

This report proposes a methodology allowing the end-user to account for different aspects (environmental, financial, etc.) of the impacts of infiltration (or exfiltration) on drainage systems, and to evaluate the impacts of possible selected investment strategies. The multi-criteria method Electre III is suggested to compare and to rank the various strategies, as this method has a high potential adaptability to diverse contexts and objectives. It is better than a simple traditional cost-benefit approach which over-promotes the economic aspects and introduces compensation of the heterogeneity between the different criteria.

An example of application of the proposed methodology to a semi-virtual case study for infiltration has been given. It reveals that the methodology can be used, but also that validated and site specific information (data, measurements, calibrated models, etc.) is necessary in order to carry out a valuable analysis. In many real practical cases, neither all impacts of infiltration and exfiltration nor the impacts of rehabilitation techniques can not be properly evaluated because of a lack of data, models and of knowledge on the involved phenomena. Consequently, many hypotheses will still be introduced in the analysis, with unknown bias in the application of the methodology. Results should always be reviewed carefully. The final decision making process should take into account the context of the study and not forget the aspects that have not been modelled.

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10 Annexes

Annex 1: Calibration of the model for rainfall-runoff process over the catchment and flow propagation in the sewer system

Annex 2 : Design of the WWTP in scenario A (i.e. without infiltration)

Annex 3: Calculation of the foul sewage flow

Annex 4: Annual total pollutant loads emitted into the receiving water for years 9 and 15

Annex 5: Comparison of different investment strategies according to the different criteria (Pareto graph) for different scenarios (IR = 75, 100 and 200 %)

Annex 6: Analysis of the robustness of the thresholds and weights used in the case-study

Annex 1 : Calibration of the model for rainfall-runoff process over the catchment and flow propagation in the sewer system

Presentation of the model

The aim of the model is to simulate the runoff flow at the outlet of the catchment using rainfall data series.

During rainfall events, the whole catchment is not contributing to the runoff process, especially the pervious areas where a part of the rainfall volume infiltrates into the soil. Desbordes (1974) considers that pervious areas could be neglected for an imperviousness coefficient greater than 20 %. With a catchment surface of 245 ha and an imperviousness coefficient of 46 % in Ecully, pervious areas are neglected and the active surface S_a which effectively contributes to the runoff is estimated to be equal to 113 hectares.

Initial losses correspond to the storage of water into depressions of the surface at the beginning of a rainfall event. Runoff occurs only if the rainfall height is higher than the height of initial losses P_i (mm). In addition, continuous losses due to i) infiltration and ii) the impervious areas not connected to the sewer system occur during the entire duration of the rainfall event.

In the case study, the model used to simulate the rainfall excess intensity is described by the following equations :

$$H_{cb}(t+\Delta t) = H_{cb}(t) + i(t) * \Delta t \qquad \text{Equation 1}$$

where $H_{cb}(t)$ is the cumulative gross rainfall height at time t (mm)
$i(t)$ is the gross rainfall intensity (mm/h)
and $H_{cb}(0) = 0$

If $H_{cb}(t+\Delta t) > Pi$ then $H_n(t+\Delta t) = H_{cb}(t+\Delta t) - Pi$
otherwise $H_n(t+\Delta t) = 0$ Equation 2

where $H_n(t)$ is the rainfall excess height at time t (mm)
P_i are the initial losses (mm)

$$i_n(t) = \frac{(H_n(t+\Delta t) - H_n(t))}{\Delta t} * (1 - Pc) \qquad \text{with } P_c \in [0,1] \qquad \text{Equation 3}$$

where $i_n(t)$ is the rainfall excess intensity (mm/h)
P_c is the continuous losses coefficient (-).

The flow propagation over the catchment and in the sewer network has been modelled using a single linear reservoir model. In this approach, the catchment and sewer network are considered globally (Figure 32).

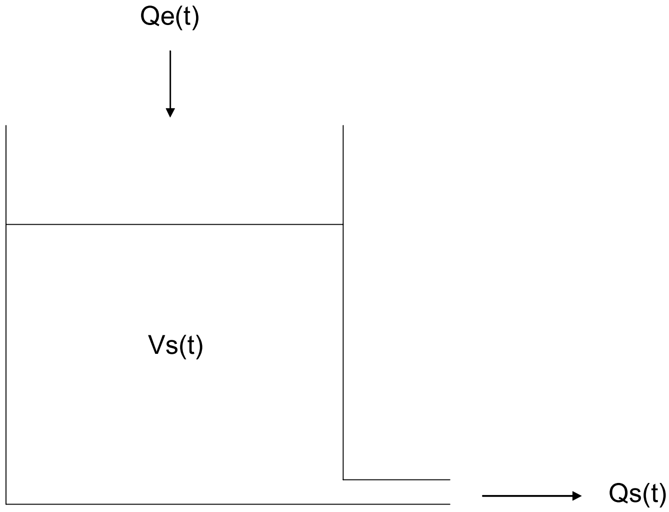


Figure 32 : Example of a reservoir model.

The linear reservoir model is characterised by the following equations:
- a continuity equation

$$\frac{dV_s(t)}{dt} = Q_e(t) - Q_s(t) \qquad \text{Equation 4}$$

where $V_s(t)$ is the volume stored in the reservoir at time t (m^3)
$Q_e(t)$ is the flow entering the reservoir at time t (m^3/s)
$Q_s(t)$ is the flow outgoing from the reservoir at time t (m^3/s)

- a storage equation

$$V_s(t) = KQ_s(t) \qquad \text{Equation 5}$$

where K is the lag-time coefficient (s).

Equation 5 is derived versus time and combined with Equation 4:

$$\frac{dV_s(t)}{dt} = K\frac{dQ_s(t)}{dt} = Q_e(t) - Q_s(t) \qquad \text{Equation 6}$$

By integration, Equation 7 is obtained:

$$Q_s(t+\Delta t) = C_1 Q_e(t) + C_2 Q_e(t+\Delta t) + C_3 Q_s(t) \qquad \text{Equation 7}$$

where $C_1+C_2+C_3 = 1$.

Depending on the numerical scheme used, different values of the coefficients C_1, C_2 and C_3 can be obtained. In the case study, the numerical integration was chosen. The values of the coefficients are then the following ones:

$C_1 = 0$ $\qquad$ $C_3 = \exp(-\frac{\Delta t}{K})$ $\qquad$ and $\qquad$ $C_2 = 1 - C_3$

Calibration of the model

The model was calibrated using 10 rainfall events recorded at the rain gauge of Champagne-au-Mont-d'Or with a time step of 6 minutes. The events were selected in order to obtain a large range of duration, rainfall height and rainfall intensity, representative of the diversity of rainfall events occurring in the Ecully catchment (Table 11).

Rain event (N°)	Dp (h)	Ht (mm)	Imax (mm/h)	Imoy (mm/h)
1	6.68	8.4	5.5	1.26
2	6.12	9.2	6.8	1.50
3	1.94	8.8	41	4.55
4	2.18	6.6	57.8	3.02
5	18.85	23.8	36	1.26
6	7.75	30.0	66	3.87
7	5.47	12.8	10.3	2.34
8	2.97	10.0	47.4	3.37
9	2.27	4.6	11.7	2.02
10	0.77	3.6	30.9	4.67

Table 11 : Rainfall events selected for the calibration of the linear reservoir model. D_p: duration of the event, H_t: rainfall height, I_{max}: maximum rainfall intensity, I_{moy}: average rainfall intensity.

For each rainfall event, different values were given to the model parameters K, P_i, P_c and T_1. The parameter T_1 expressed in minutes represents the time offset between the beginning of the rainfall event and the beginning of the increase of flow at the outlet of the catchment. The range of variation and the step of variation for each parameter are given in Table 12.

Model parameter	Range of variation	Step of variation
P_i (mm)	[0 ; 5]	$\Delta P_i = 1$mm
P_c (-)	[0 ; 1]	$\Delta P_c = 0.2$
K (min)	[10 ; 80]	$\Delta K = 5$ min
T_1 (min)	[5 ; 20]	$\Delta T_1 = 1$ min

Table 12 : Range and steps of variation of the model parameters P_i, P_c, K and T_1 for the calibration of the linear reservoir model for the Ecully catchment.

Simulated hydrographs $Q_s(t)$ are compared to the hydrographs recorded at the outlet of the Ecully catchment with a 2 minutes time step. The mean squared error e is calculated for each set of parameters according to Equation 8:

$$e = \sum_t (Q_s mes(t) - Q_s sim(t))^2 \qquad \text{Equation 8}$$

where $Q_s mes\ (t)$ is the flow measured at the outlet of the catchment at time t (m^3/s)
$Q_s sim\ (t)$ is the flow simulated using the linear reservoir model at time t (m^3/s).

The best set of parameters is the one which minimizes the mean squared error function. It is given for each rainfall event in Table 13.

Rain event (N°)	Pi (mm)	Pc (-)	K (min)	T1 (min)	Vmeas (m3)	Vsim (m3)	(Vsim-Vmeas)/ Vmeas (%)
1	1	0.6	30	13	2623	3131	19
2	2	0.6	35	15	2785	3049	9
3	2	0.6	20	11	3142	2879	-8
4	2	0.6	15	15	1927	1945	1
5	0	0.6	60	10	8950	10077	13
6	0	0.6	50	12	13897	12685	-9
7	1	0.6	40	13	4393	4990	14
8	2	0.6	15	9	3540	3389	-4
9	1	0.6	20	17	1652	1523	-8
10	1	0.8	35	13	318	300	-6
All	**1**	**0.6**	**40**	**12**	36965	38807	5
Av	1	0.6	33	13			

Table 13 : Sets of parameters selected for each rainfall event after calibration of the linear reservoir model in Ecully catchment with associated measured and simulated volumes (respectively V_{meas} and V_{sim}). 10 rainfall events have been used separately (n° 1, 2,..., 10) and globally (All). The averages of the results are given in the line named Av.

The results are constant for P_c but more variable for P_i, K and T_1. The variation of T_1 is not really important in Ecully because it does not change the runoff volumes and the runoff flows. However, P_i influences the runoff volumes and K influences the amplitude of the flows. As a first step, the values of the model parameters were set to the values obtained by the calibration using the 10 events globally:

- P_i = 1 mm
- P_c = 0.6
- K = 40 min
- T_1 = 12 min.

The choice of this set of parameters was then verified with 5 other rainfall events.

Verification of the model

The 5 rainfall events selected for the validation were also recorded at the rain gauge of Champagne with a time step of 6 minutes. As for the calibration, they were selected in order to obtain a large range of duration, rainfall height and rainfall intensity (Table 14).

Rain event (N°)	Dp (h)	Ht (mm)	Imax (mm/h)
1	4.80	4.19	5.63
2	11.30	7.8	13.96
3	5.60	7.4	7.58
4	28.40	12.19	3.85
5	11.37	15.99	9.79

Table 14 : Rainfall events selected for the validation of the linear reservoir model.
D_p: duration of the event, H_t: rainfall height, I_{max}: maximum rainfall intensity.

For each event, the linear reservoir model was applied using the set of parameters P_i, P_c, K and T_1 selected from the calibration phase. The results are given in Table 15.

Rain event (N°)	Pi (mm)	Pc (-)	K (min)	T1 (min)	Vmeas (m3)	Vsim (m3)	(Vsim-Vmeas)/ Vmeas (m3)
1	1	0.6	40	12	1519	1352	-11
2	1	0.6	40	12	2363	2879	22
3	1	0.6	40	12	2649	2706	2
4	1	0.6	40	12	2973	4734	59
5	1	0.6	40	12	4742	6340	34
All	**1**	**0.6**	**40**	**12**	**14245**	**18001**	**26**

Table 15 : Comparison of the volume simulated V_{sim} using the linear reservoir model with the selected parameters P_i, P_c, K and T_1 and the measured volume V_{meas} at the outlet of the Ecully catchment. 5 events have been used separately (1, …, 5) and globally (all).

The model overestimates the volume at the outlet of the catchment for the events n° 2, 3, 4 and 5 and underestimates it for the event n° 1. However, taking into account the uncertainties in the measurement of the flow, the results obtained for the events n° 1, 2, 3 and 5 appear correct and are validated. The overestimation of the volumes for the event n° 4 appears quite important. This can be explained by the low rainfall intensity of this rainfall event which probably leads to a low runoff. Consequently, the selected values of P_i and P_c have been validated for the case-study.

Regarding *K*, the selected value was not validated because the maximum peak flow was often over- or underestimated. As *K* seemed to vary significantly from one event to another one, correlations with the rainfall event characteristics were searched (Table 16).

	Dp (h)	*Ht (mm)*	*Imax (mm/h)*	*Imoy (mm/h)*	*K (min)*
Dp (h)	1				
Ht (mm)	0.67	1			
Imax (mm/h)	-0.17	0.51	1		
Imoy (mm/h)	-0.34	0.39	0.81	1	
K (min)	0.75	0.54	-0.19	-0.18	1

Table 16 : Correlation matrix between the lag-time *K* and the characteristics of the rainfall event, i.e. the duration of the event D_p, the total rainfall height H_t, the maximum rainfall intensity I_{max} and the average rainfall intensity I_{moy}

K appears strongly correlated to the duration of the rainfall event and the total rainfall height. To express this correlation, two models have been tested:

- a linear function: $K = a + b \times D_p + c \times H_t$ Equation 9

- a power function: $K = a \times D_p^{\,b} \times H_t^{\,c}$ Equation 10

where *a*, *b*, *c* are constants to be set. Both functions are calibrated using the values of *K* obtained in the calibration of the linear reservoir model and best sets of constants are selected using the mean squared error optimisation. Both functions give satisfactory results. As the linear function is simpler, it was selected with the following set of parameters:
- $a = 17.82$
- $b = 3.19$
- $c = 0.11$.

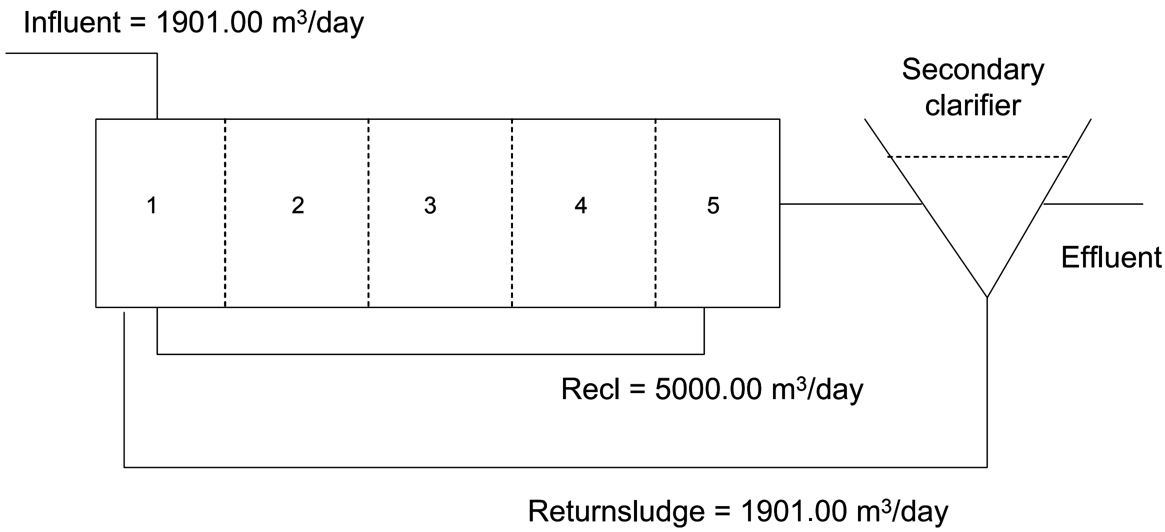
After the calibration and the validation of the linear reservoir model, it appears that initial and continuous losses are quite important : respectively 1 mm and 60 % of the total cumulative rainfall height, which means that many impervious surfaces are not connected to the sewer system. These values are specific for the Ecully catchment but are quite high for the purposes of a demonstration case-study. The continuous losses have been finally set to 30 % of the total cumulative rainfall height in order to work with a more usual case-study. The following selected set of parameters was applied for each simulation in the case-study :

- P_i = 1 mm
- P_c = 0.3
- $T1$ = 12 min
- $K = 17.82 + 3.19 \times D_p + 0.11 \times H_t$

Annex 2 : Design of the WWTP in scenario A (i.e. without infiltration)

The design of the WWTP has been calculated i) for a maximum inflow *Qmax* equal to 3 times the mean dry weather flow DWF, i.e. 3×1901 m3/day for case-study, and ii) for a maximum nitrogen concentration in the effluent equal to 10 mg/L as set in the European Directive 91/271 and its amendment Directive 98/15 for the sensitive areas of more than 100 000 people-equivalent.

Figure 33 shows the biological tanks and the different circulating flows (influent, nitrate recirculation and return sludge flows) as represented in the ASIM software. The secondary clarifier is schematized for the clarity of the figure but its functioning is not explicitly simulated in the ASIM software.



Reactor n°	1	2	3	4	5
Volume (m^3)	400.00	400.00	400.00	400.00	400.00
O2 Conc. (mg/L)	-	-	2.00	2.00	2.00

Figure 33 : Flow scheme of the WWTP designed for the scenario A, i.e. without infiltration (adapted from the representation of flow scheme in the ASIM software).

The total biological tank including reactors 1, 2, 3, 4 and 5 has a volume of 2000 m^3 which represents a retention time of approximately 24 h for the daily dry weather influent. The separation of the different tanks into reactors in ASIM is made to simulate the hydraulic conditions in the WWTP. Reactors 1 and 2 constitute the anoxic zone of 800 m^3 with the oxygen concentration equal to zero. Reactors 3, 4 and 5 constitute the aerobic zone of 1200 m^3 with oxygen concentration of 2 mg/L. The return sludge flow is equal to the influent flow and the recirculation flow is calibrated in order to maintain a sufficient nitrate concentration in the anoxic zone for the denitrification. In our case, with a design temperature of 13°C, ammonium and the nitrate concentrations in the effluent during dry weather are equal to 0.3 mg N/L and 9.3 mg N/L respectively, which correspond to a 83 % removal rate for total nitrogen. COD concentration in the effluent reaches 31 mg O_2/L which corresponds to a 94 % removal rate during dry weather.

It has been assumed that aeration is made by means of 2 fine bubbles diffuser devices with an efficiency of 2.7 kg O_2 per kWh under standard oxygenation test conditions and a global transfer coefficient of 0.55. Daily aeration time is 12 hours per day which leads to an energy consumption of 426 kWh/day.

Regarding the mixing of activated sludge, there is one mixer in the anoxic tank with a specific power of 8 kW/m^3 functioning 24 hours per day and one mixer in the aerobic tank with a specific power of 8 kW/m^3 in function 12 hours per day. Energy consumption for total mixing of anoxic and aerobic tanks is equal to 269 kWh/day.

The recirculation pump from the aerobic to the anoxic tank and the return sludge pump are assumed to be in function 12 hours per day and to have a power of 6 kW. The energy consumption for the recirculation is equal to 141 kW/day.

Annex 3 : Calculation of the foul sewage flow

In Ecully, the flow is measured continuously at the outlet of the catchment with a time step of 2 minutes. 4 or 5 dry weather days per month have been selected and monthly averaged for the year 2003 (Figure 34).

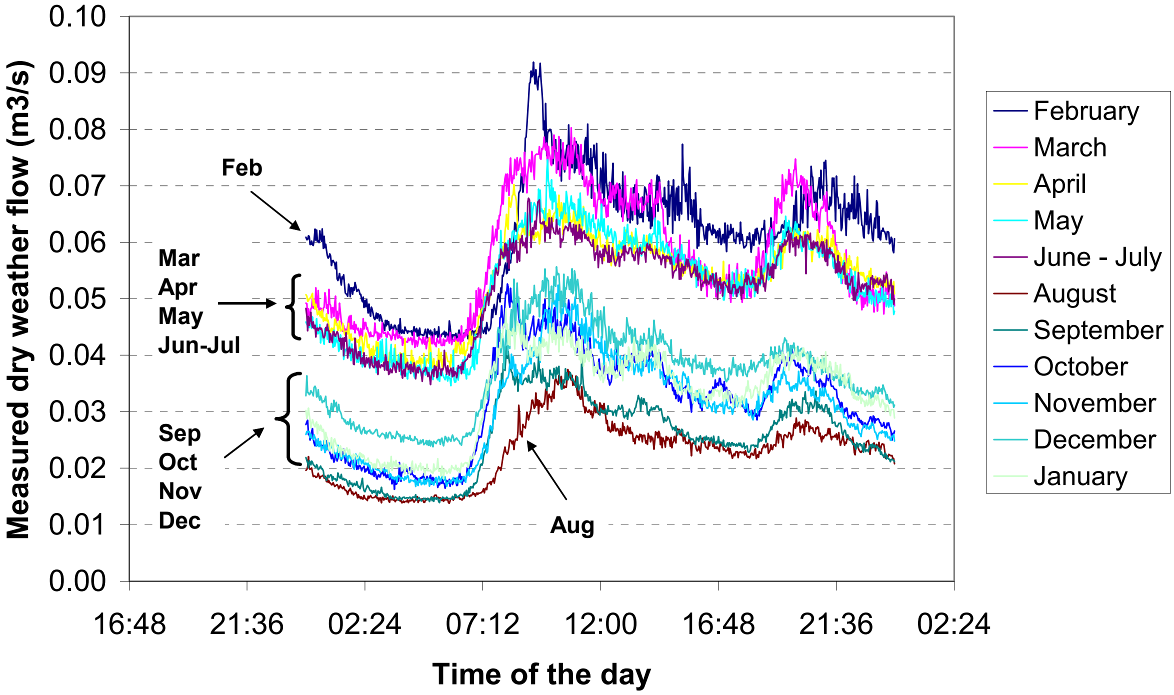


Figure 34 : Mean dry weather flow measured in Ecully.

Figure 34 shows i) a high flow period from February to July with average daily dry weather flow of 4760 m^3/day and ii) a low flow period from August to January with average daily dry weather flow of 2600 m^3/day. Infiltration measurement campaigns carried out in December 2003 (De Bénédittis, 2004) showed that infiltration occurs even during the low groundwater level period corresponding to a daily foul sewage flow equal to 1901 m^3/day . This foul sewage flow value has been used for all the simulations in the case-study. Moreover, in order to take into account the seasonal variability of the infiltration, it was assumed in our simulations that infiltration occurs 7 months per year, from January to July.

Annex 4 : Annual total pollutant loads emitted into the receiving water for years 9 and 15

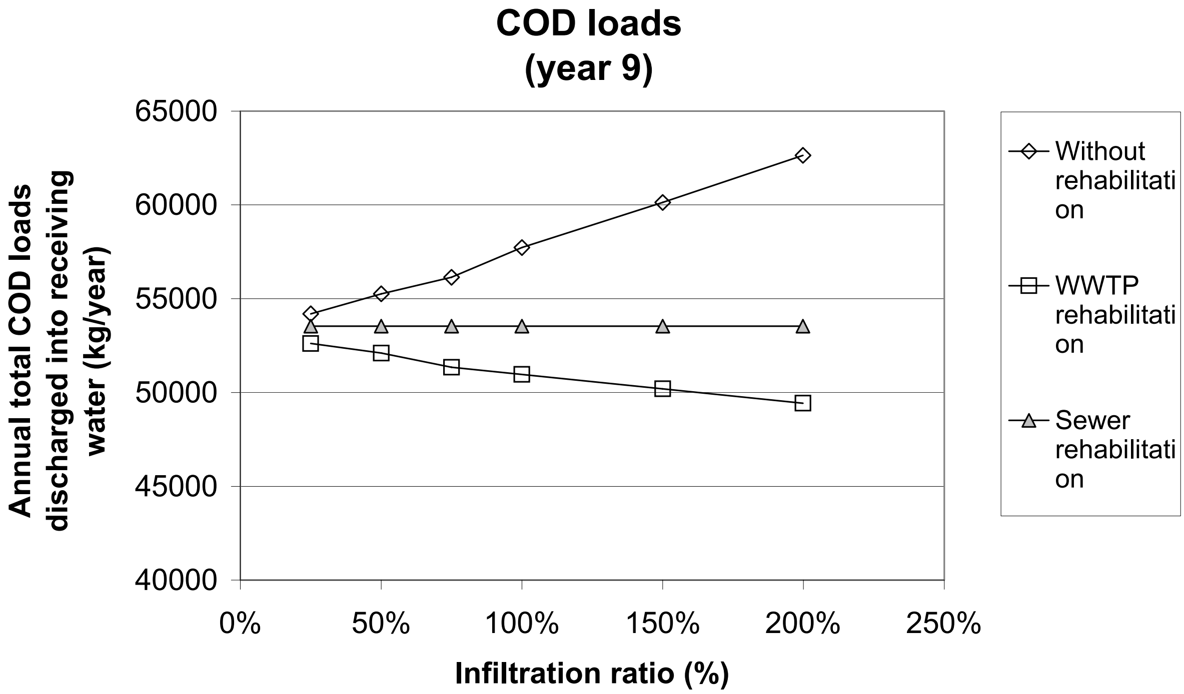


Figure 35 : Annual total COD load emitted into the receiving water vs. the infiltration ratio and the investment strategy for year 9.

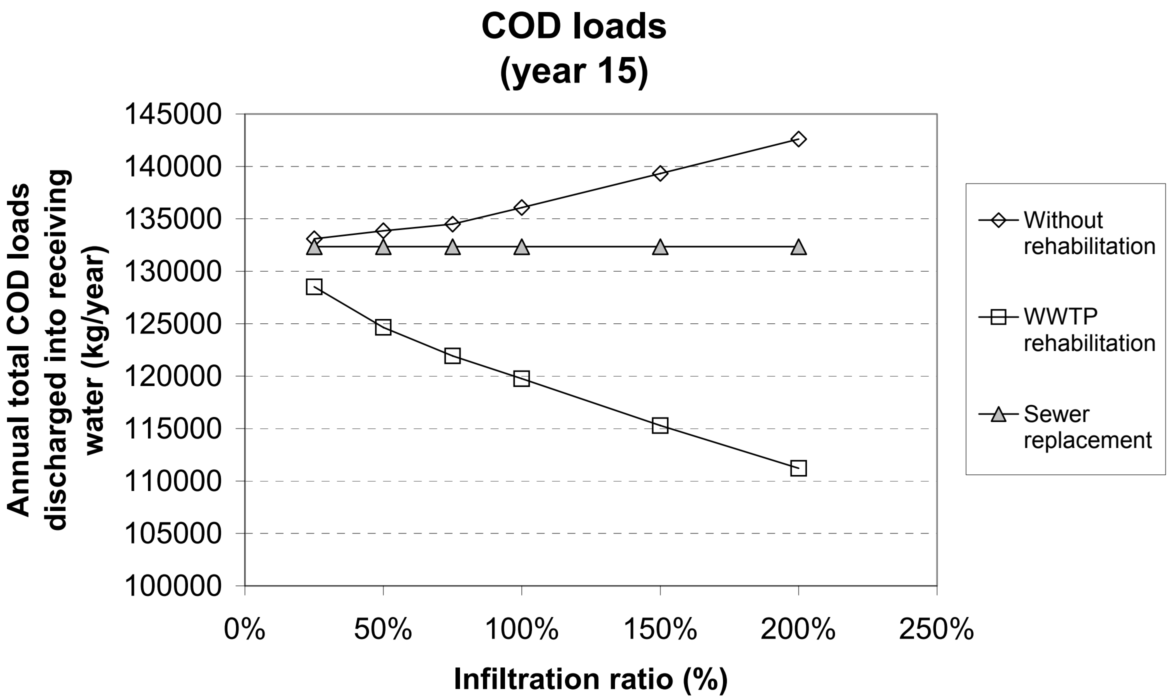


Figure 36 : Annual total COD load emitted into the receiving water vs. the infiltration ratio and the investment strategy for year 15.

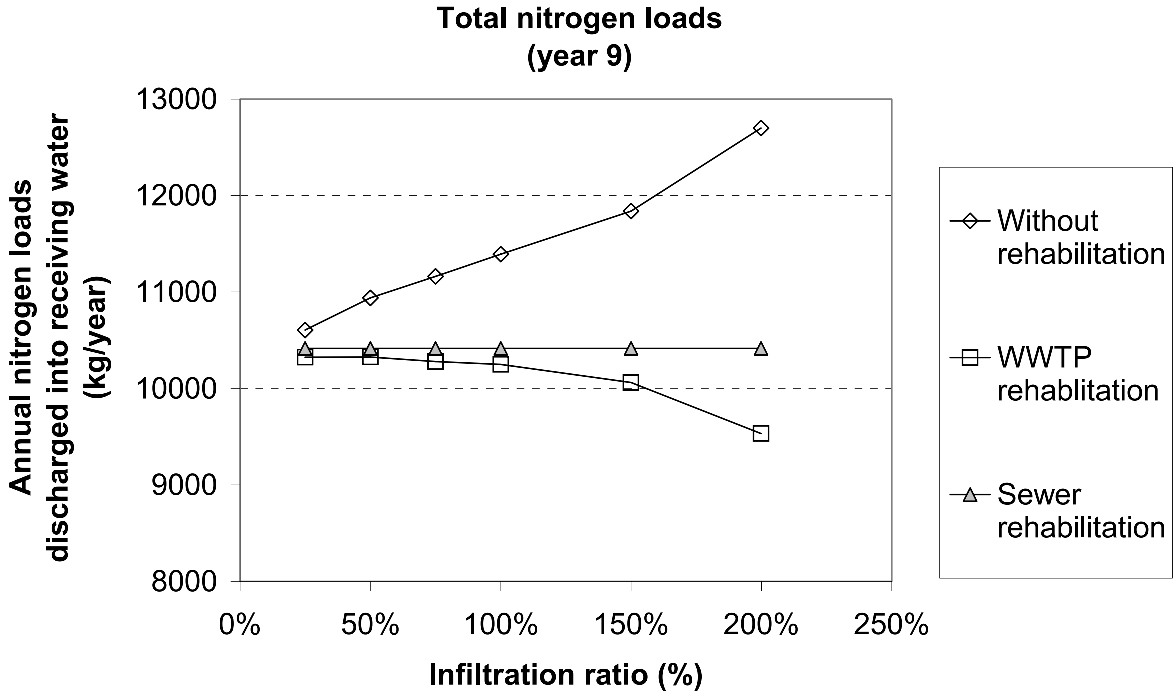


Figure 37 : Annual total nitrogen load emitted into the receiving water vs. the infiltration ratio and the investment strategy for year 9.

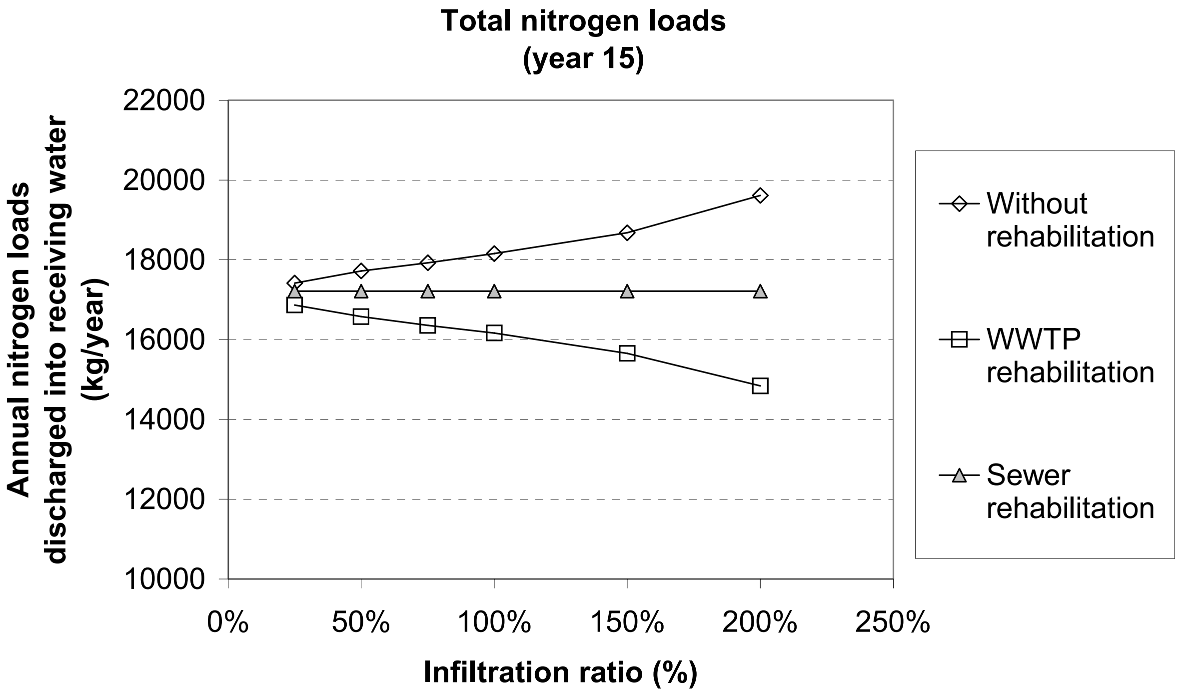
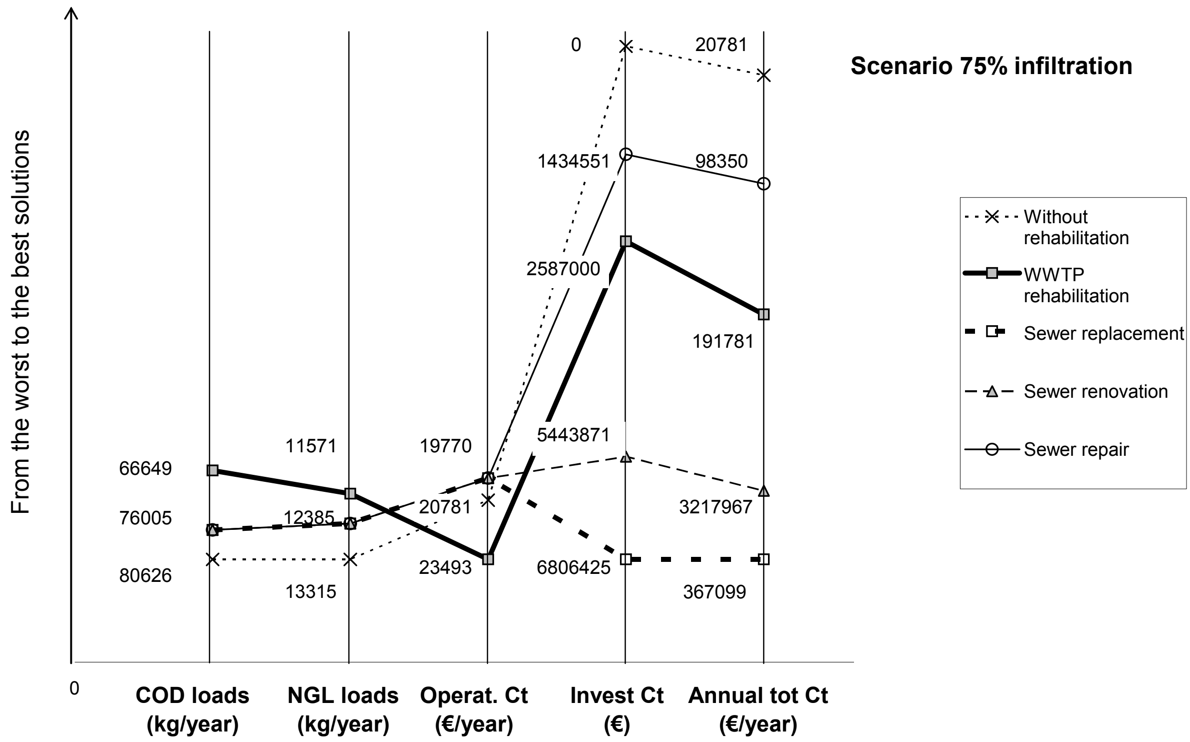
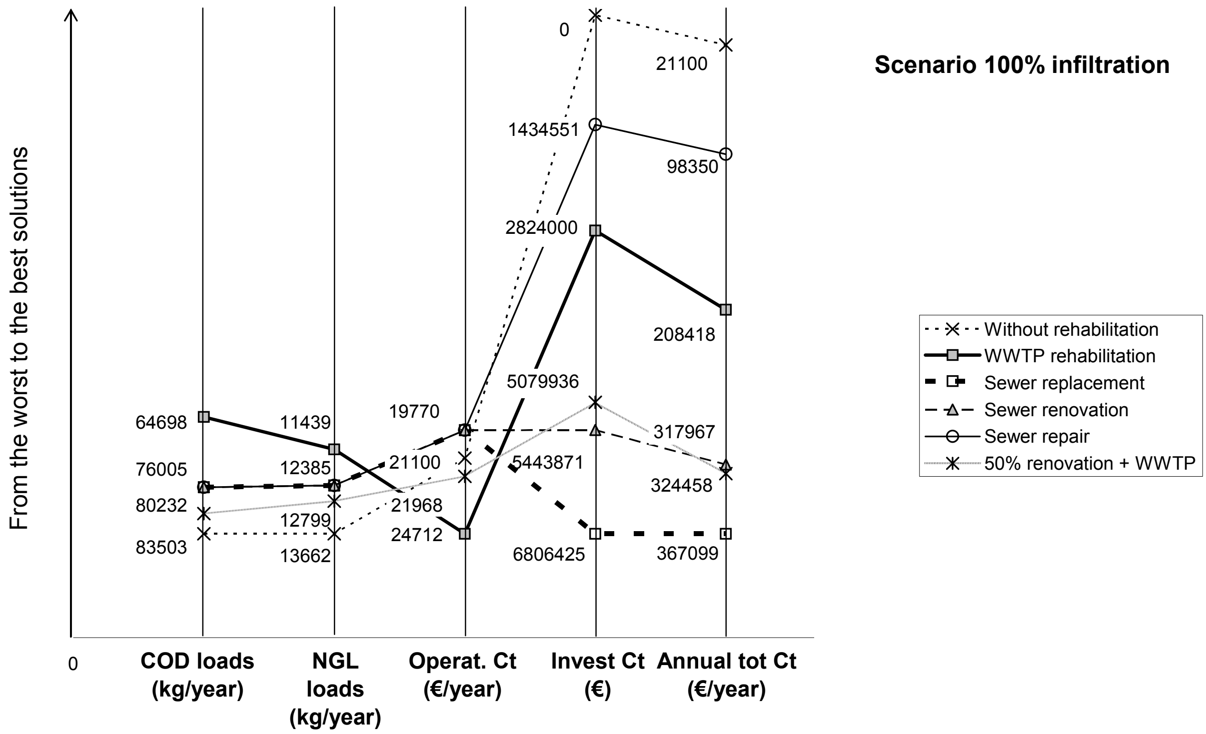


Figure 38 : Annual total nitrogen load emitted into the receiving water vs. the infiltration ratio and the investment strategy for year 9.

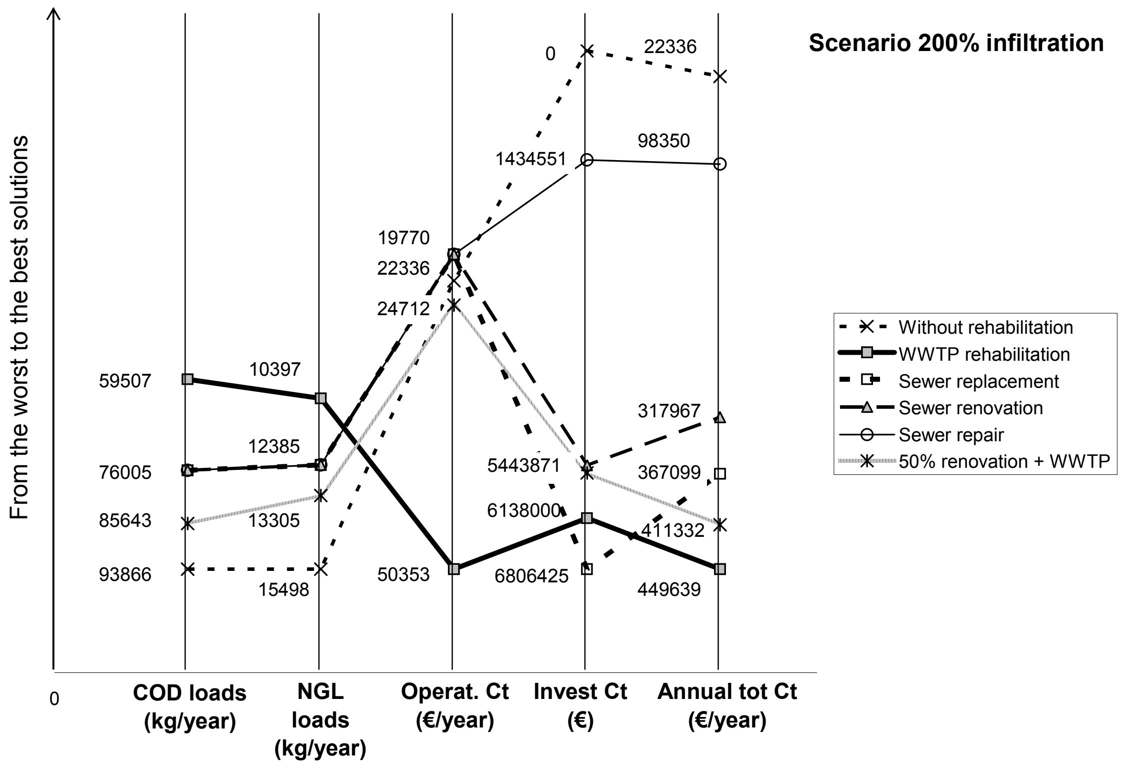
Annex 5 : Comparison of different investment strategies according to the different criteria (Pareto graph) for different scenarios (75 – 100 – 200 % of infiltration).

Year 1





Scenario 100% infiltration
From the worst to the best solutions
0
COD loads (kg/year)
NGL loads (kg/year)
Operat. Ct (€/year)
Invest Ct (€)
Annual tot Ct (€/year)
Without rehabilitation
WWTP rehabilitation
Sewer replacement
Sewer renovation
Sewer repair
50% renovation + WWTP
64698
76005
80232
83503
11439
12385
12799
13662
19770
21100
21968
24712
0
1434551
2824000
5079936
5443871
6806425
21100
98350
208418
317967
324458
367099



Scenario 200% infiltration
From the worst to the best solutions
0
COD loads (kg/year)
NGL loads (kg/year)
Operat. Ct (€/year)
Invest Ct (€)
Annual tot Ct (€/year)
Without rehabilitation
WWTP rehabilitation
Sewer replacement
Sewer renovation
Sewer repair
50% renovation + WWTP
59507
76005
85643
93866
10397
12385
13305
15498
19770
22336
24712
50353
0
1434551
5443871
6138000
6806425
22336
98350
317967
367099
411332
449639

Annex 6 : Analysis of the robustness of the thresholds and weights used in the case-study

The set of parameters chosen for the application of Electre III for the case-study presented in Table 17 and the associated results given in Table 18 are considered as the initial set of parameters and the initial solutions.

Point of view	Environmental criteria		Economic criteria		
Criterion	Total COD load (kg/year)	Total nitrogen load (kg/year)	Operation costs (€/year)	Investment costs (€)	Annual total costs (€/year)
Weight	3	3	2	2	2
Indifference threshold	5 %	5 %	2000	50 000	10 000
Preference threshold	10 %	10 %	5 000	100 000	20 000
Veto threshold	-	-	-	-	-

Table 17 : Set of Electre III parameters without accounting for sewer serviceability.

	IR scenario	Ranking results
75%	Year 1	Without → WWTP → Repair → Renov → Replac
	Year 9	Without → WWTP → Repair → Renov → Replac
	Year 15	Without → WWTP → Renov → Replac Repair
100 %	Year 1	Repair → WWTP → Without → Renov → Replac → 50% R
	Year 9	Repair → Without → WWTP → Replac Renov
	Year 15	Without → WWTP → Renov → Replac Repair
150 %	Year 1	Repair → WWTP → Without → 50 % R→ Replac Renov
	Year 9	Repair → Without → Replac WWTP Renov
	Year 15	Without → WWTP → Renov → Replac Repair
200 %	Year 1	Repair → WWTP → Without → 50 % R Renov Replac
	Year 9	Repair → Without → Replac WWTP Renov
	Year 15	Repair → WWTP → Without → Replac Renov

Table 18 : Final ranking of the investment strategies after simulations using Electre III.
“Without” means “no rehabilitation” and “50 % R” means the mixed investment strategy.

The analysis of the robustness of the initial parameter set consists in finding the thresholds and weights values which will change the order of the strategies compared to the initial solutions. In practice, all parameters are kept constant and equal to their initial values unless one of them. The evaluated parameter is given successive values in increasing or decreasing order until the initial ranking of the investment strategies changes. Table 19 summarizes the results of the robustness analysis for the pollutant loads for the case-study for year 1.

Scenario (% of infiltration)	Electre III parameter			Value changing the initial solution	New final ranking
75%	Weight	Environmental criteria		≤ 2	Without → **Repair** → **WWTP** → Renovation → Replacement
		Economic criteria		≥ 3	Without → **Repair** → **WWTP** → Renovation → Replacement
	Thresholds	COD loads	Indif. thres.	≥ 0.12	Without → **Repair** → **WWTP** → Renovation → Replacement
			Pref. thres.	≥ 0.14	
		NGL loads	Indif. thres.	≤ 0.04	**WWTP** → Without → Replacement **Repair** Renovation
			Pref. thres.	≤ 0.09	**WWTP** → Without → Replacement **Repair** Renovation
100%	Weight	Environmental criteria		≤ 2	Repair → **Without** → WWTP → 50% R → Renovation → Replacement
		Environmental criteria		≥ 4	**WWTP** → Without → Renovation → 50% R → Replacement Repair
	Thresholds	COD loads	Indif. thres.	≥ 0.06	Repair → **Without** → **50% R** →Renovation → Replacement WWTP
			Pref. thres.	-	
		NGL loads	Indif. thres.	≥ 0.08	Repair → **Without** → Renovation → 50% R →Replacement WWTP
			Pref. thres.	-	

<table>
<tr><th>Scenario (IR)</th><th colspan="3">Electre III parameter</th><th>Value changing the initial solution</th><th>New final ranking</th></tr>
<tr><td rowspan="7">150%</td><td rowspan="2">Weight</td><td colspan="2">Environmental criteria</td><td>≤ 1</td><td>Repair → WWTP → 50% R → Replacement
Without Renovation</td></tr>
<tr><td colspan="2">Environmental criteria</td><td>≥ 4</td><td>WWTP → Without → Replacement → 50% R
Repair Renovation</td></tr>
<tr><td rowspan="5">Thresholds</td><td rowspan="2">COD loads</td><td>Indif. thres.</td><td>≥ 0.08</td><td>Repair → Renovation→ Without → Replacement
WWTP
50% R</td></tr>
<tr><td>Pref. thres.</td><td>-</td><td></td></tr>
<tr><td rowspan="2">NGL loads</td><td>Indif. thres.</td><td>≥ 0.1</td><td rowspan="2">Repair → Without → WWTP → Replacement → 50% R
Renovation</td></tr>
<tr><td>Pref. thres.</td><td>≥ 0.12</td></tr>
<tr><td colspan="4" style="display:none"></td></tr>
<tr><td rowspan="6">200%</td><td rowspan="2">Weight</td><td colspan="2">Environmental criteria</td><td>≤ 2</td><td>Repair → Renovation → Replacement → 50% R
Without
WWTP</td></tr>
<tr><td colspan="2">Environmental criteria</td><td>≥ 5</td><td>Repair → Renovation → Replacement → 50% R
WWTP Without</td></tr>
<tr><td rowspan="4">Thresholds</td><td rowspan="2">COD loads</td><td>Indif. thres.</td><td>≥ 0.2</td><td rowspan="2">Repair → Renovation → Replacement → 50% R
WWTP
Without</td></tr>
<tr><td>Pref. thres.</td><td>≥ 0.24</td></tr>
<tr><td rowspan="2">NGL loads</td><td>Indif. thres.</td><td>-</td><td rowspan="2"></td></tr>
<tr><td>Pref. thres</td><td>-</td></tr>
</table>

Table 19 : Robustness analysis for the pollutant loads for the case-study.