



CARE – S

**Computer Aided REhabilitation of Sewer networks.
Decision Support Tools for Sustainable Sewer Network Management**

WP3 – Description and Validation of Hydraulic performance

Task 3.2– Modeling Hydraulic Performance Temporal Decline

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1 INTRODUCTION

The CARE-S project is funded by the European Community and aims to develop methods and software that will enable engineers of the water undertakings to establish and maintain an effective management of their sewer rehabilitating the right sewer at the right time. The results shall be disseminated as a manual on Best Management Practice (BMP) for sewer Rehabilitation.

This project is organised in the following Working Packages (WP):

- WP1: Construction of a control panel of Performance Indicators (PI) for rehabilitation;
- WP2: Description and validation of structural condition;
- WP3: Description and validation of Hydraulic performance;
- WP4: Rehabilitation technology information system;
- WP5: Socio – economic consequences;
- WP6: Multi – criteria decision support;
- WP7: Elaboration of CARE-S prototype;
- WP8: Testing and validation of CARE-S prototype;
- WP9: Dissemination;
- WP10: Project management.

WP3 is divided into three sub – tasks, each one with its specific objective, schedule, deliverables and methodology.

Task 3.1:

- **Evaluation of current best practice:** *The task is based on the evaluation of available software (commercial or freeware) capable of modelling water flow, water quality and sediment transport in sewer systems, receiving water bodies, WWTP and groundwater. Modelling is needed at different level of complexity: in integrated urban drainage modelling, adoption of detailed models leads to unacceptably long calculation times and implies large memory needs. Therefore, simplified models are appropriate to simulate very quickly long rainfall time series, thus allowing to evaluate environmental impact on receiving water. However, detailed models are needed in order to evaluate discharge, depth, and velocity in each pipe of the sewer system and its ancillary structure (e.g. Combined Sewer Overflow). The review activity focused on the evaluation of available software (commercial or freeware) capable of modelling water flow, water quality and sediment transport phenomena drove into the selection of three 1D models which will be included in the CARE-S Prototype: MOUSE, InfoWorks and SWMM.*

STATUS: Task completed in 2003 and described in delivery D7, April 2003.

Task 3.2:

- **Modelling hydraulic performance temporal decline:** *The task will be based on a general "dynamic wave" transport model. The effect of failures on pipes on flow capacity will be studied in a hydraulic model concept and tested under real conditions. Increasing*

roughness of the material in concrete or brick sewer networks will also be included in the model, thanks to the advances made in WP2. Sub task 3.2 is scheduled for the first year of the project.

Task 3.3:

- **Environmental impacts of rehabilitation:** *A literature survey on the environmental impacts influenced by rehabilitation has been carried out focussing on impacts on wastewater treatment plant and receiving water due to infiltration and inflow into the sewer system. The study has shown that the influence of infiltration can cause increased pollution in dry weather periods and in wet weather periods in the receiving water. A methodology for the system analysis by using Integrated simulation has been developed. Therefore SIMBA and SIMBA-Sewer are used. A synchronous simulation of the sewer network, the wastewater treatment plant and the receiving water and their interactions is possible with this software package. A model of the wastewater treatment plant based on the benchmark treatment plant of the COST group has been implemented in SIMBA and coupled with an artificial sewer network and receiving water. So scenario analysis investigating environmental effects of rehabilitation can be carried out in the project.*

A concept for the WP3.3-tool carrying out the assessment of impacts to the treatment plant and the receiving water has been developed.

A concept has been developed to modify runoff coefficients within the runs of the hydraulic model

STATUS: Task to be completed in 2004. This task results in delivery D9 with delivery +18 months.

Task 3.4:

- **Combining hydraulic and reliability model:** *A model for sewer and storm water reliability will be established, based on work package 2 and task 3.1, 3.2 and 3.3 of this work package.*

This report refer to sub – task 3.2, the analysis of hydraulic performance of sewer systems in temporal decline.

The present report is the second milestone in the CARE-S Work package 3 "Hydraulic performance evaluation" which is organised in four major sub-packages. It describes a generic modelling procedure to assess the key hydraulic performance indicators, its extension to handle the hydraulic performance temporal decline aspects/modelling and in order to meet the requirements of other CARE-S WPs.

The first part of the report covers the main aspects of procedures, data requirements and result formats related to hydraulic performance evaluation of sewer systems by means of the three UDM tolls that are to be used in the CARE-S project. Because of its importance for the comparison of flooding problems in different climatic and operational environments, the three case studies provided by the three end-users have been included.

The second part of the report describes the development of a method for hydraulic temporal decline modelling. The method combines a precise characterisation and quantification of failures in the recently adopted European coding system EN 13508-2 and 3D CFD tool Fluent for hydraulic modelling of each failure in order to provide formulas for the calculations of failures in 1D UDM tools. A case study has been used to establish a hydraulic temporal decline model based on a very rough description of the pipelines conditions (the Norwegian CCTV inspection standard) and illustrate the effect on temporal decline on extend of surface flooding and sewer system environmental performance (CSO).

The third part of the report introduces the basic concepts of the combining hydraulic and reliability models, later still under development within WP2.

The last chapter of the report sets (up) a basis for the interaction between WP3.2 and its internal links within the WP3 and external links with other WPs in the CARE-S project.

2 MODELLING HYDRAULIC PERFORMANCE AND TEMPORAL DECLINE

The first aim of the CARE-S WP 3.2 is to provide the end-users of CARE-S Rehabilitation Manager with a possibility to assess hydraulic performance of the piped sewer system deploying the existing methods which are included in three widely used 1D UDM software packages: MOUSE, InfoWorks and SWMM. The most important hydraulic performance indicators obtained from a model simulation are flow and velocity during dry weather and *frequency of surcharging/high surcharging/surface flooding for any location in the network (pipe/manhole)*. In addition to modelling the hydraulic performance of the piped sewer system, the commercial UDM software provides a great deal of information on the performance of specific ancillary structures such as combined sewer overflows and pumping stations. Some of information obtained from a model simulation such as *the total volume and frequency of CSO discharges together with the total infiltration volume entering the pipe sewer system* can be categorised as environmental and related to the derived standards which are surrogates for fundamental standards related to both the pollution discharge and the quality of the receiving water (WFD, E/E).

The second aim of the CARE-S WP 3.2 is to extend the standard hydraulic modelling approach used in 1D UDM to cover a temporal decline of the system's hydraulic performance. This is done by adopting the hydraulic characterisation of failures defined by the European defect coding system EN13508-2 (k, lambda, M), and by hydraulic computations performed with 3D CFD modelling software (FLUENT). Because of the level of detail to which the failures attributes are calculated in a 3D model, the new 1D model will be superior to the existing methods of sewer flooding prediction.

Good understanding of procedures and methods being used in the commercial 1D UDM software packages MOUSE, InfoWorks and SWMM is of crucial importance in the development of the WP 3 concept, especially for the model for hydraulic performance temporal decline and reliability modelling. Due to different procedures and methods being used in different commercial 1D UDM software packages, three different CARE-S end-user case studies have been chosen to support the WP 3.2 concept development and reinforce its links with other WPs and CARE-S Rehabilitation Manager.

The three case studies on hydraulic performance modelling which are based on the MOUSE, INFOWORKS and SWMM software package were established and calibrated by CARE-S end-users respectively in Oslo (Norway), Correggio and Palermo (Italy). These case studies consider:

- Hydrologic approach related to hydraulic performance assessment
- Model building, time series data collection and processing
- Process data and model calibration
- Performance indicators as function of the model outputs.

The major details from the case studies are given in Appendices 6.1, 6.2 and 6.3.

2.1 The analysis of hydrologic and hydraulic approaches in MOUSE, INFOWORKS and SWMM: the main features for Hydraulic performance modeling.

2.1.1 MOUSE –(Appendix 1 - The Nordberg case study in Oslo, Norway)

The MOUSE Procedure of modelling and verifying sewer networks for analysis in sewerage rehabilitation is supported by a range of modules which form a complex modelling environment. In this chapter the main MOUSE modules and collection of the key process data which are used to build and calibrate a model to replicate sewer system performance is reviewed.

The aim of the Nordberg case study in Oslo, Norway, is to provide detailed information and guidance on the most effective means of achieving interaction or coupling between the MOUSE simulator and other software components in the CARE-S project.

2.1.1.1 Hydrologic and hydraulic approaches related to hydraulic performance assessment

a) Surface Runoff Module and Rainfall Dependent Inflow& Infiltration module (RDII)

For the computation of the catchment surface runoff, different models are available in MOUSE:

Model A: Time/Area method

Model B: Non-linear reservoir (kinematic wave) method

Model C: Linear Reservoir method, proposed in two sub-variants:

Model C1: Dutch runoff model

Model C2: French runoff model

UHM: Unit Hydrograph Model

For all these models some general data about catchment area is needed. This data includes the catchment size, the connection point from the network, coordinates indicating the geographical position, the number of inhabitants, and specification of additional inflow. Then, for each model, some specific catchment data and some model parameters are also required.

RDII module

MOUSE RDII is one of the several modules available in MOUSE Package. This model has been developed in order to take into account the effects of previous hydrological events on the description of the discharge generated in sewer systems. Indeed, it has been noticed, that actually, the flows are often increasing in the system long after the rainfall and the surface runoff have stopped. This is a consequence of the phenomenon, usually called Rainfall Induced Infiltration, which does not depend only on the actual precipitation, but which is also heavily affected by the actual hydrological situation, in other words, by the earlier hydrological events. For a certain rainfall event, the increase in flow will therefore differ, depending on the hydrological events during the previous period. The Rain Induced Infiltration is also distinguished by a slow flow response, which takes place during several days after the rainfall event.

MOUSE RDII model describes to components of the flow hydrographs:

FRC - Fast Response Component, which is not influenced by the previous hydrological situation (i.e. high or low soil moisture contents). This component consists of the rain induced inflow and fast infiltration component.

SRC - Slow Response Component, which comprises slow infiltration component, is highly dependent on the previous hydrological conditions, and usually responses slowly to a rainfall. The SRC component consists of the rest of the precipitation-induced infiltration and dry weather infiltration/inflow.

MOUSE RDII is a lump type of model, which enables the description of the FRC component based on MOUSE Surface Runoff model (A or B), and the description of the SRC component based on a hydrological model (NAM). It permits simulation of single event as well as simulation of very long periods and calculation of the total discharge within the catchment area as a sum of surface runoff and the inflow/infiltration. MOUSE RDII enables the computation of snow storage and snow melting. A degree-day algorithm accounts for the melting of snow.

For the computation, some information about initial conditions and boundary conditions are required, as well as specific parameters for the simulation. The initial conditions data should be as realistic as possible, and it is recommended to disregard the simulation results for the period of about half a year to one year from the start simulation, in order to reduce errors. The boundary conditions include data relevant to precipitation, temperature and potential evaporation.

The RDII model structure, based on a set of linked empirical and physical equations is shown below.

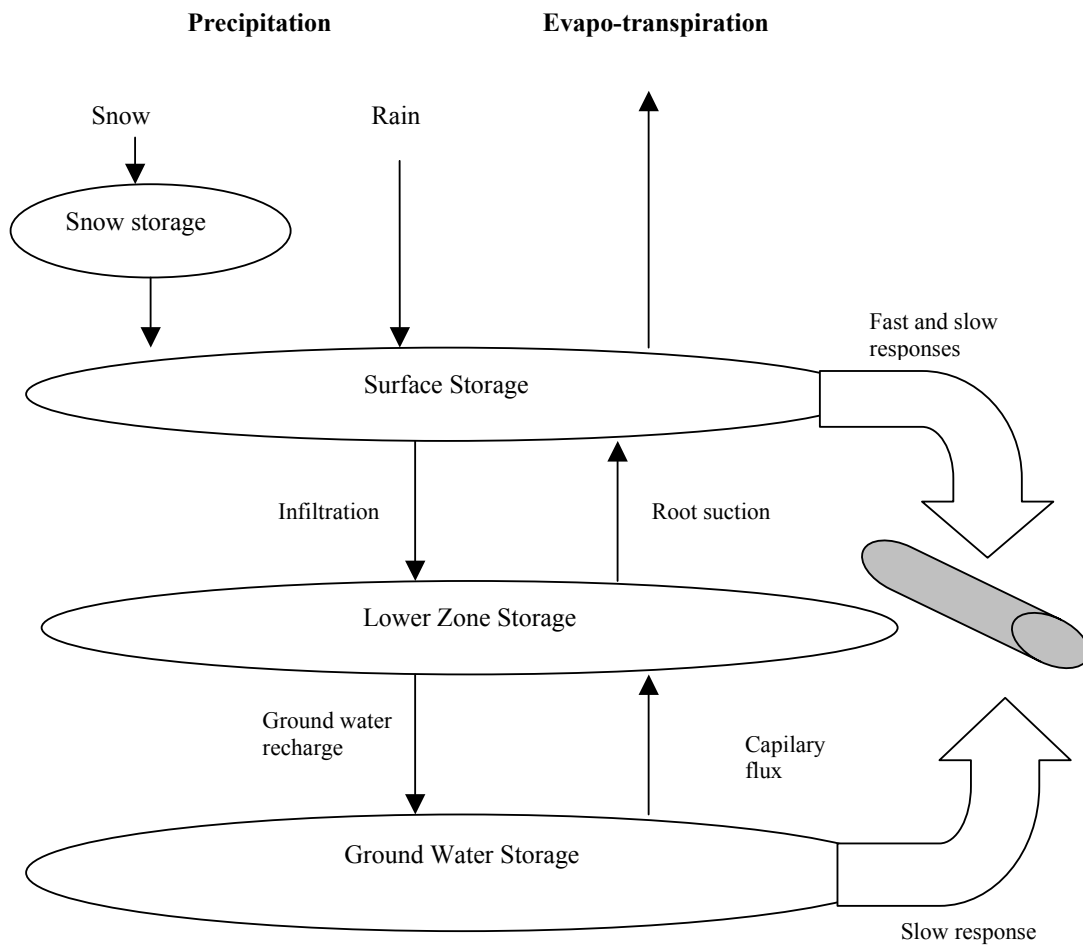


Figure 1 Principles of RDII module. RDII models the fast or direct flow component from an impervious surface, and the indirect flow component from a pervious surface. The precipitation is routed through four different types of storage: snow, surface, root zone (Lower Zone) and groundwater. Infiltration of water into soil is related to a coefficient and the ratio of water stored in the soil to the maximum storage. Sewer infiltration is related to the modelled height of the water table.

MOUSE RDII requires input data, such as catchment and hydrological data, but also time series of meteorological data (precipitation, evaporation and temperature). This data is stored in the different MOUSE input files.

RDII simulation is carried out continuously for the whole period specified and the results are saved in *.CRF file, which can then be used as an input data for pipe flow computation in MOUSE. In this result file, the following results are available:

- Discharge calculated with the surface runoff model (FRC component)
- Discharge calculated with the RDII model (SRC component)
- Total discharge
- Variation of water content in the surface storage
- Variation of water content in the snow storage

To validate the model, the calculated discharge can be plotted on the same graph with the measured discharge and compared. This is possible by using MIKE View, which is the presentation tool for MOUSE.

b) Pipe Flow Module (HD)

MOUSE HD is related to the pipe flow computation. It allows the simulations of unsteady flows in pipe networks with alternating free surface and pressurized flow conditions. The model used for this computation is based on an implicit, finite difference numerical solution of free surface flow equations, the Saint Venant's equations. It can be applied to branched as well as to looped pipe network. MOUSE HD also allows the modelling of backwater effects, flow reversal, surcharging in manhole and storage basins.

The modelling of the physical system requires the definition of several different elements and functions in the network:

- Links, where the dependent flow variables such as discharge or water level can be uniquely described as functions of time and space. Links are defined between two nodes and can stand for closed conduits or open channel. The main characteristics of a link are the associated nodes, the material, the longitudinal parameters and the cross section.
- Nodes, which can stand for manholes, basins, storage nodes and outlets.
 - Manholes are characterized by their shape and the properties of inlet and outlet pipes.
 - Structures (basins) based on arbitrarily shaped structures of significant volume. These structures are used to define non-circular manholes, tanks, reservoirs, basins and natural ponds.
 - Storage nodes which enable a controlled simulation of the surface flooding, that means a controlled return of the water into the sewer system.
 - Outlets, where the modelled system interacts with receiving waters. It is assumed that the external water volume is so large that the outlet water level is not affected by the outflow from the sewer system. An outlet can also behave as an inlet and generate reverse flow.

Functions used for the calculation of the flow between two nodes or in specified links, such as:

- Overflow weirs introduced to reduce the hydraulic load in the pipe system during extreme flow conditions by allowing a part of the flow to be spilled to a recipient. This function allows flow “backwards”.
- Q/H relation, which links the water level in the structure or in the manhole with the released discharge.
- Overflow formula used for the computation of free overflow. Two methods of flow computation are available; one based on the energy loss coefficient and weir

orientation and the other based on a standard rectangular overflow weir formula with the specification of the discharge coefficient.

- Orifice function characterizing an opening of any shape, allowing water passage between separated parts of the network. Usually, an orifice represents a flow restriction, and four different types of flow regimes can be considered: free overflow, submerged overflow, free underflow and submerged underflow.

- Pump function defined by start and stop levels, but also by a $\Delta H/Q$ relation (pump performance is defined as a function of the water level difference) or an H/Q relation (correlation between the pump discharge and the water level in the pump-sump basin).

- Flow regulation: contrary to other functions, this one enables the control simulation within the pipe.
- Non-return valve included into the model.

Assumptions

The description and computation of unsteady flow in links are based on Saint Venant's equations. These equations expressed the conservation of continuity and momentum. The assumptions, on which this theory has been established, are the following:

- Water is incompressible and homogeneous
- Bottom slope is small
- Wavelengths are large compared to the water depth
- Flow is sub-critical

These equations are non-linear, hyperbolic partial differential equations, and initial and boundary conditions are required.

Pressurized flow

The full flow capacity of a closed pipe can be defined as a discharge at which the flow depth is equal to the conduit height. Then, the flow changes from the free surface flow to the pressurized flow. Other equations should be applied to describe this situation. However, it is possible to generalize the equations for free surface flow, so that the pressurized flow in closed conduits is covered. This can be done by introducing a fictitious slot in the top of the closed conduit.

Approximations and simplification

In order to simplify the computation and to reduce the computation time, some approximations and simplifications can be made in the general Saint Venant's equations. However, this should be used very carefully, otherwise false results, which do not reflect the reality of the case study, can be generated.

Three main approximations are available in the MOUSE software:

- Kinematic wave approximation. The flow is calculated on the assumption of a balance between friction and gravity forces. Thus, the inertia and pressure terms in the momentum equation are neglected, which is not very realistic in most real flow situations. With this approximation, backwater effects can not be simulated.
- Diffusive wave approximation. In this method, the pressure term is maintained in the computation, and only inertia terms are neglected, so it is possible to consider backwater effects.
- Dynamic wave approximation. This method uses the full momentum equation, which enables the description and the taking into account of all forces affecting the flow conditions.

The last method is the most precise and reliable method, but also the longest one, in terms of computation time.

Digitisation

In MOUSE, the flow equations are solved by an implicit finite difference method. The transformation of these equations to a set of implicit finite difference equations is performed on a computational grid consisting of alternating Q and h points, in other words the points where the discharge Q and water level h, respectively, are computed at each time step. It is possible to express, after some considerations, the water level or discharge variable within the branch as a function of the water levels in the upstream and downstream nodes, and therefore to build a “branch” matrix. Stability criteria are introduced for a stable solution of the finite difference scheme. The simulation time step is also optimized in the computation, because the computational efficiency of a discrete-time numerical simulation algorithm is highly dependent on the time step applied in simulations. This time step can be constant or time-dependent.

Initial conditions

For the computation of the pipe flow, some initial conditions are also required. In general, the hydrodynamic computation is started from the flow conditions in the system specified for $t=0$. Then default conditions are applied. For example, the default initial water depth is taken equal to 0,5 % of the diameter for circular pipes, or to a maximum of 0,005m. However, realistic initial conditions can be specified by taking the water levels and discharges from previously calculated result file.

Boundary conditions

The definition of a set of boundary conditions is also necessary. Flow equations are solved for each conduit, between two nodes, and the boundary conditions are required at both end of the conduit, at each time step throughout the computation. Two groups of boundary conditions can be distinguished:

- The external boundary conditions, describing the interaction of the modelled system with its surroundings (for example, constant outlet water level)
- The internal boundary conditions, describing relations between certain parts of the model (for example, when a weir discharges to another manhole or structure)

Other parameters used in the computation

At last, some criteria like the flow resistance, friction resistance, Manning's coefficient, the nature of the flow (turbulent or laminar), head losses in manholes and structures due to a change in the flow direction, a change in elevation, or contraction at an outlet, are also taken into account in the computation and can be adjusted by the user.

Simulation of a surface flooding and dried pipes

If the water level in a manhole or a basin reaches the ground level, an artificial “inundation” basin is inserted above the node. The surface area of this basin is gradually increased from the area in the manhole or the basin to a 1000 times larger area, thus simulating the surface inundation. The maximum level of inundation is 10 meters above the specified ground level. When the outflow from node surmounts the inflow, the water stored in the inundation basin re-enters the system.

It is however possible to prevent water from exceeding the ground level and consequently to prevent surface flooding, by specifying sealed nodes. These nodes enable the pressure to rise without any water on the ground surface.

On the contrary, spilling nodes allow the water to spill irreversibly out of the system, when the ground level is reached.

If parts of the sewer system dry-out during the simulation, then the model artificially maintains a minimum water depth in those conduits, corresponding, per default, to 2% of the characteristic dimension of the conduit (diameter for circular pipes) or to maximum 0,02 m.

c) MOUSE LTS

MOUSE LTS – Long Term Statistics module is an add-on module to MOUSE HD Pipe flow model. This module is used to convert a usual MOUSE simulation covering a long historical period into a discontinuous series of relevant events feasible for the simulation, and to generate event-based, annual and system statistics related to the simulated period. The fundamental principle behind the MOUSE LTS is to introduce a system which automatically combines dynamic pipe flow simulations during wet weather and **simple hydrological simulation** during dry weather periods, which results in accurate computation of sewer system components, while reducing the simulation time.

Creation of a job list

For the purpose of isolating the events relevant for dynamic simulation, MOUSE LTS generates a job list on the basis of system information and knowledge of time series for dry weather flow (wastewater), rainfall and other loads. The job list criteria START and STOP are used to determine the start/ending dates and times for the dynamic simulations.

Some additional criteria (so-called 'runtime criteria') may optionally be specified, which would extend the simulations beyond the time intervals defined in the job list.

The job list (*.MJL) is written into an ASCII file, so that the actual start times and dates and the estimated stop time for the individual jobs may be modified. The specified job list criteria are saved into a MOUSE LTS file (*.MTF).

Initial conditions

MOUSE LTS provides a system to define appropriate initial conditions for each simulated event. The initial condition can either be specified as an empty pipe for each job in the job list (MOUSE default), or a set of hotstart files can be specified to be used for individual events.

Runtime criteria

Runtime criteria can optionally be specified in order to extend the simulations beyond the time intervals defined in the job list. The purpose of these criteria is to determine the earliest time at which the simulation can be stopped without losing important information. These criteria are evaluated through an Evaluation matrix. In fact, at every time step during computation (but only after the end timer in the job list), each criterion is subject to logical evaluation as an element of the Evaluation matrix. The Evaluation matrix consists of an arbitrary number of lines, each containing up to five individual criteria. Each line of the evaluation matrix represents a combination of criteria which have to be fulfilled simultaneously if the combination is to be evaluated as TRUE and the simulation stopped.

Statistical results

MOUSE LTS provides the possibility to generate event-based and annual statistics. The computation of statistics can be controlled by specifying the minimum time intervals between the two occurrences (ΔT) in order to consider these as independent.

Extreme statistics are based on a list with ranked results for the specific parameter. The 'return period' for a certain event is calculated by dividing the complete period covered by the simulation with the rank of the event. If for example the observation period is 10 years, the event with rank 1 gets assigned a return period of 10 years, number two gets a return period of 5 years, number 3 gets a return period of 3.33 years, and so on.

For nodes three types of statistical results can be selected: peak water levels, total event and annual discharge for weirs, pumps and outlets. For links (pipes) six types of statistical results can be selected: peak water level, peak flow, peak flow velocity, total event discharge, event duration for discharge and total yearly discharge.

Annual statistics for discharge at weirs, pumps and outlets and spatial extreme statistics for entire system are stored in a statistical result file (*.ERF). The results can be assessed from the MIKE View. Extreme values Hmax and Qmax as function of return period can be presented on the Horizontal plot and on the Longitudinal profile plot. MIKE View generates a series of tabular

reports on the basis of the computed statistics. Any report can be generated in ASCII or in HTML format.

2.1.1.2 Catchment and sewer system data and model building

a) In-data for catchment and sewer system

The main MOUSE files are shown in **Table 1**. Some of them are input files whereas other are results files automatically generated by MOUSE after a computation.

Table 1 MOUSE main files

Extension of the file	Name	Contents
*.MPR		The main work file of the project (list of the different files related to this project and the different parameters needed for the simulation)
*.HGF	Hydrological and Catchment Data	Catchments and parameters such as run-off coefficient
*.UND	Urban Network Data	Network configuration data: nodes, links, pumps, weirs Operational data: real time control of weirs, gates and pumps Hydraulic data: flow regulation, Manning numbers for pipes Boundary data: rain intensities, water levels, temperatures, evaporation
*.RPF	Repetitive Profile Data	Definition of a temporal dry weather variation
*.DWF	Dry Weather Flow Data	Quantification of dry weather loads
*.MTF	MOUSE LTS Data	Definition of a long term discontinuous simulation
*.ADP	Additional Parameter File	Optional sections
*.PRF	Result file	Standard results from a Mouse network computation
*.CRF	Computed runoff file	MOUSE runoff results: computed runoff time series
*.NOF	Result file	MOUSE detailed RDI result file used as hotstart in run off computation

The HGF in-data file for MOUSE (*HydroloGical F*ile) contains data for catchments as well as the parameters sets for Runoff model A, B, A and RDII. The format of the file is the DHI pfs-file format. This type of syntax is also used for the other in-data files for MOUSE, i.e. UND, DWF, MPR, RPF, RSF, ERS. The pfs-file format is a text format.

The UND in-data file for MOUSE (*U*rban *N*etwork *D*ata) includes data from former SWF (Catchment and Pipe System Data File), BSF (Time-Series Boundary File), PWF (Hydraulic Data File for Pipe Flow Model), RWF (Selected Rain Data File), PID (Control of weirs, gates and pumps) and ADP (ADditional Parameter file) files. The *.ADP file is an ASCII file which

temporarily includes new MOUSE functions prior to their full implementation in the MOUSE GUI, as well as some rarely used functions, which are of interest of only a few MOUSE users. Generally, the best way to work with the in-data is using the MOUSE editor, which subsequently saves the in-data in the HGF/UND file. Advance user may wish to edit the file directly. This can be done by using either a standard text editor like Notepad or by using a spreadsheet program like Excel. If using Excel the file must be opened as an comma delimited file and after editing the file saved as such again. Excel will often add trailing commas after some of the lines. This entails problems only at lines with target and section headers, i.e. all lines ending with a right bracket ']'. Commas in such lines will have to be removed before using the file in MOUSE.

A third option of how to edit in-data files outside the MOUSE editor is to establish an ODBC link between your database and MOUSE.

A detail description of the in-data files for MOUSE hydrological and pipe model (HGF, UND, DWF, RPF in-data files) are provided in Appendix 1.5.1.

b) Model building – GIS interface to the network and catchment data in MOUSE GM

MOUSE GIS

MOUSE GIS is an ArcView based application for MOUSE users developed to support the process of model building.

MOUSE GIS consists in two main parts: the network editor, and the results presentation.

- The network editor enables to display a huge amount of data and elements, to simplify the network according to specific criteria (maximum change in the diameter of the pipe), to condense the network and finally to save the data as a model for further use in MOUSE. In other words, the network editor allows the preparation of the model which is going to be exported in MOUSE for simulations.
- The results presentation allows displaying results from MOUSE simulation in combination with GIS data. Therefore, maximum height of water level, flooding problems can easily be shown on a map and localized. Moreover, this information can also be related to other information contained in the databases in ArcView.

MOUSE GM

MOUSE GM is also an application which links GIS ArcView, and MOUSE software. Most of the possibilities in MOUSE GIS are also available in MOUSE GM, but the implemented simplification concept is different. Moreover, there is an additional function, compared to MOUSE GIS: management.

The main functions of MOUSE GM are:

- Management: enables importation and exportation of model data
- Network: enables editing of network data, and gap-filling

- Catchment: enables the definitions of catchments and model parameters
- Simplification: enables the simplification of the model according to specific criteria
- Result: enables importation and presentation of results from MOUSE for example

For each project, the Folder Structure is the following.

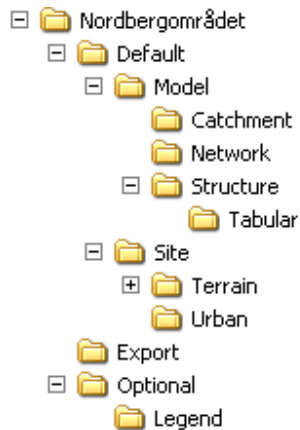


Figure 2 Folder structure in MOUSE GM.

2.1.1.3 Process data and model calibration

The use of MOUSE simulator requires precipitation data for a simple hydrodynamic computation. In addition dry weather flow data is needed. Use of RDII module requires additional data about temperature and evaporation.

Although the collection of flow data associated with CSO and pumping stations is one of the most common source of information for the model calibration and verification, the collection of flow data from a sewer collection system using velocity and depth monitors is the primary aim of any sewer flooding analysis.

GANDORF is a MOUSE module for processing, control and presentation of time series data. The module supports various data formats of different monitoring devices, such as rain gauges, water level monitors, flow meters etc. Both raw and processed data can be stored in different formats. All time series can be browsed, edited and presented in text and/or graphical form.

2.1.1.4 Hydraulic performance indicators as function of the model outputs

MOUSE produces several types of result files, which have one thing in common – they all contain time series data for various variables in the modelled system.

The different result files types are distinguish by their file name extension. Each result file type contains specific types of time series relevant to the processes simulated by the model (e.g.

runoff, sewer network flow, statistics, etc). In addition, a summary *.HTM file containing information on the current simulation and result summary after simulation is generated. A summary *.HTM file is presented in Appendix 1.5.1.

In MIKE View, which is a GUI for the presentation of results from one-dimensional DHL software, the MOUSE result files are hierarchically divided in three groups:

- *.PRF file type – MOUSE HD result file (binary)
- *.CFR file type – MOUSE runoff result file (binary)
- *.ERF file type – MOUSE LTS result file (ASCII file).

The results variables are divided into three groups:

- Variables attached to network nodes (manholes): **water level in nodes and discharge weir**
- Variables attached to links (pipes): **water level, discharge, velocity**
- System data: **Volume in sewer system, accumulated in-flow, accumulated out-flow, continuity balance.**

MIKE View provides the additional types of derived output including:

TS in links:

- Depth – Computed as a difference of the computed water level and the bottom levels
- Flood – Computed as a difference of the computed water level and the ground levels (+/-)
- Pressure – Computed as a difference of the computed water level and the conduit ceiling
- Pipe filling – Computed as the ration between the water depth and the conduit height
- Q/Qmanning. Qmanning – Computing the Q as given by the Manning formula for the link (pipe)

TS in nodes:

- Depths
- Flood
- Critical vs Water Level

All these results can be displayed in combination with the time series taken from other sources external to the result file such as ASCII files, MOUSE time series databases, etc. The results can be presented as:

- Colour animations or static plots on a horizontal plan (results viewed as lower (Min) or upper (Max) envelope for the selected variable being displayed)
- Animated longitudinal profiles (the selected variable will be presented dynamically as a replay of the model simulation through time)
- Time series and duration curve graphs
- Q-H relations

An example of the interface of MIKE View and of some results in links (pipes) that can be displayed with this program is shown below.

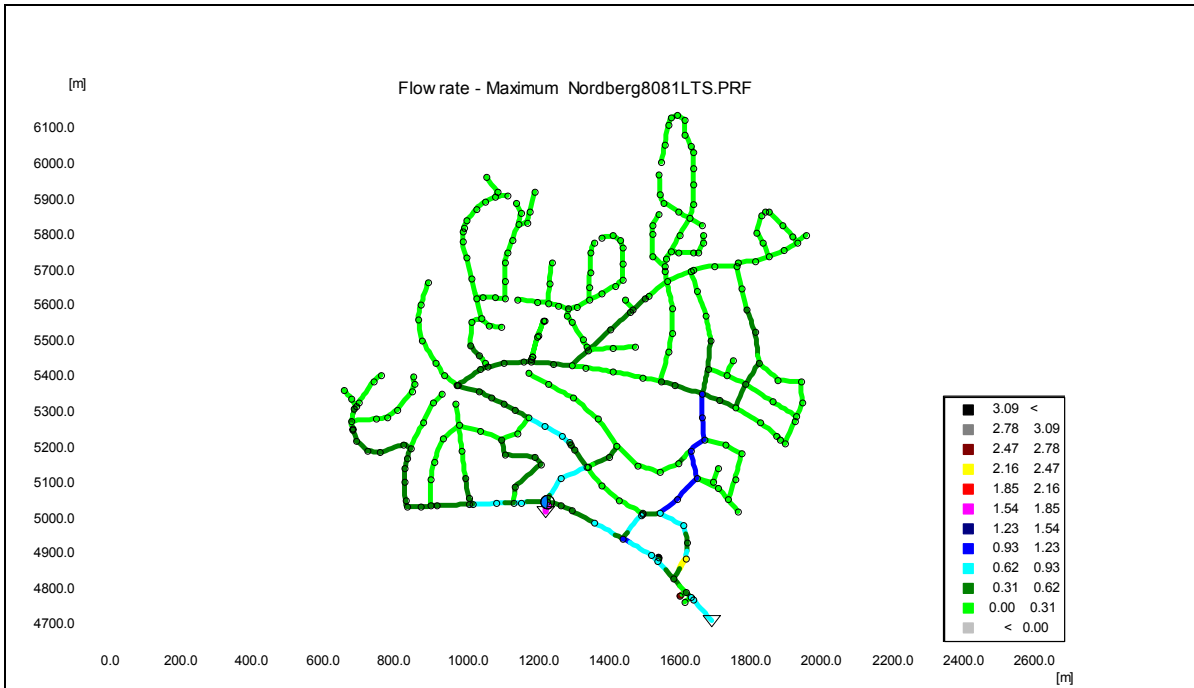
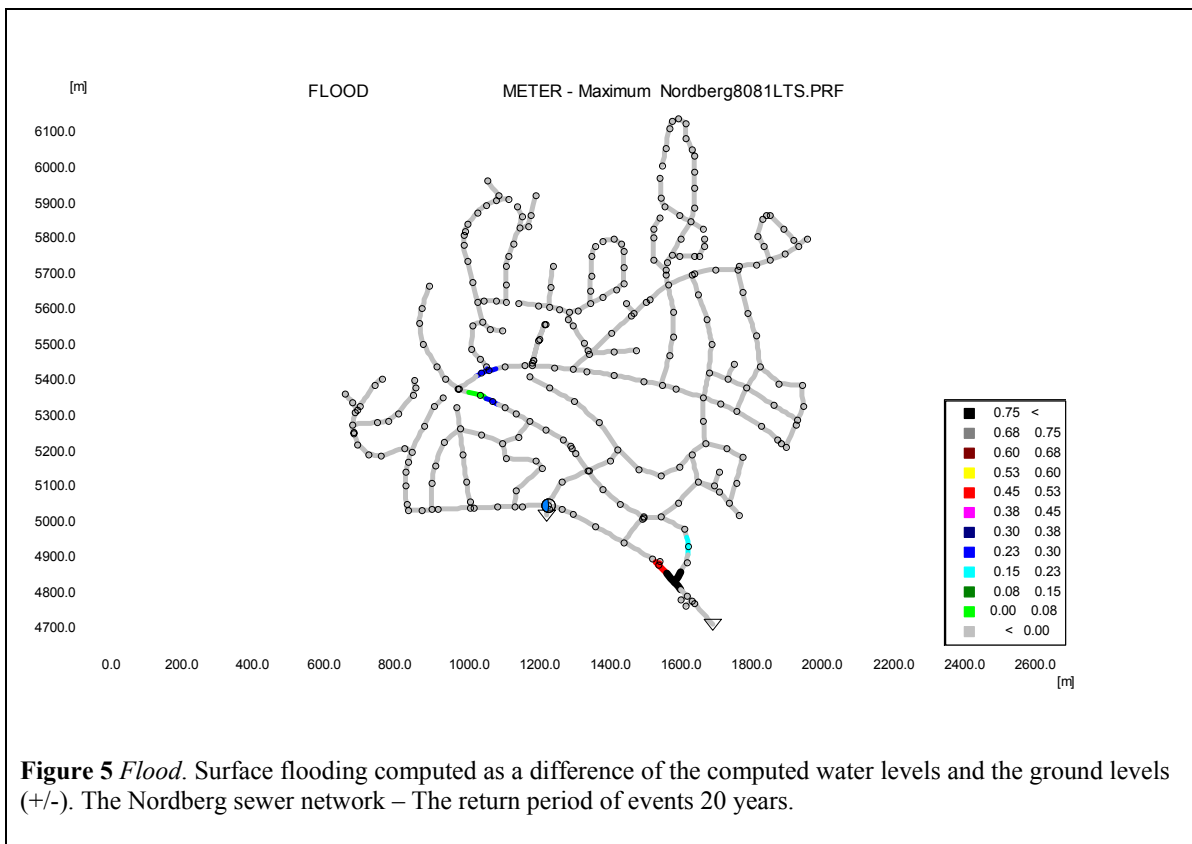
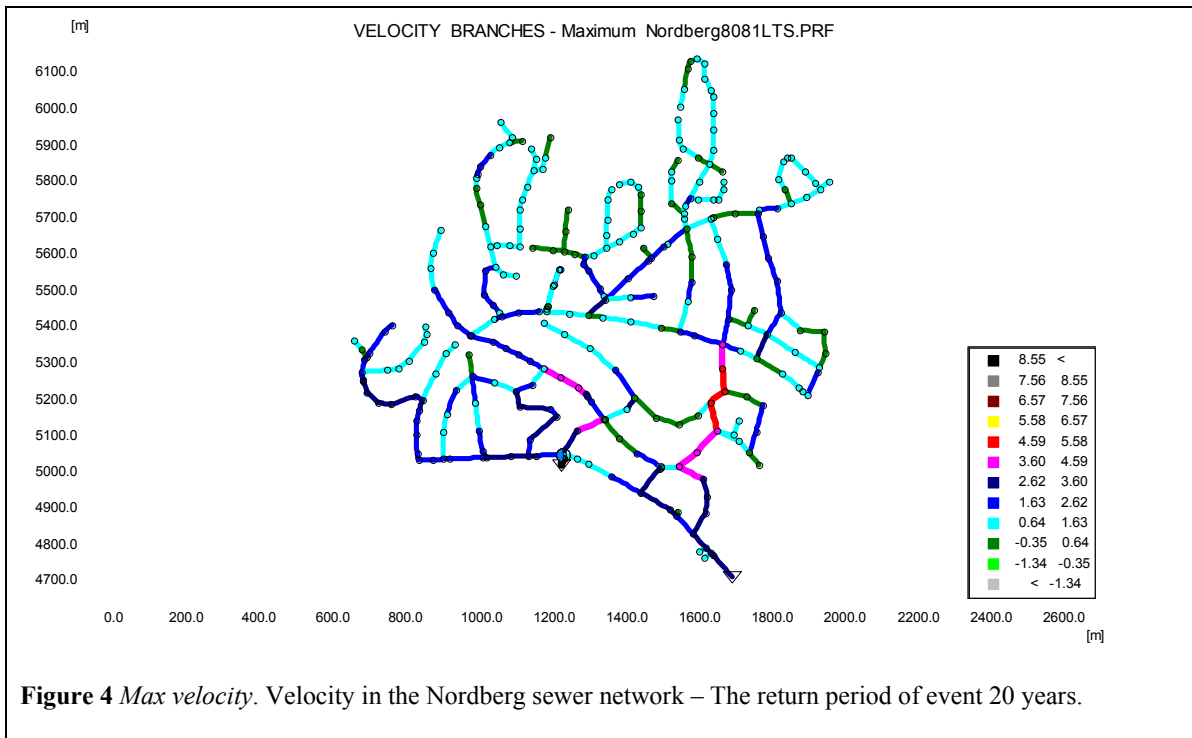


Figure 3 Max flow rate. Flow rate in the Nordberg sewer network – The return period of event 20 years.



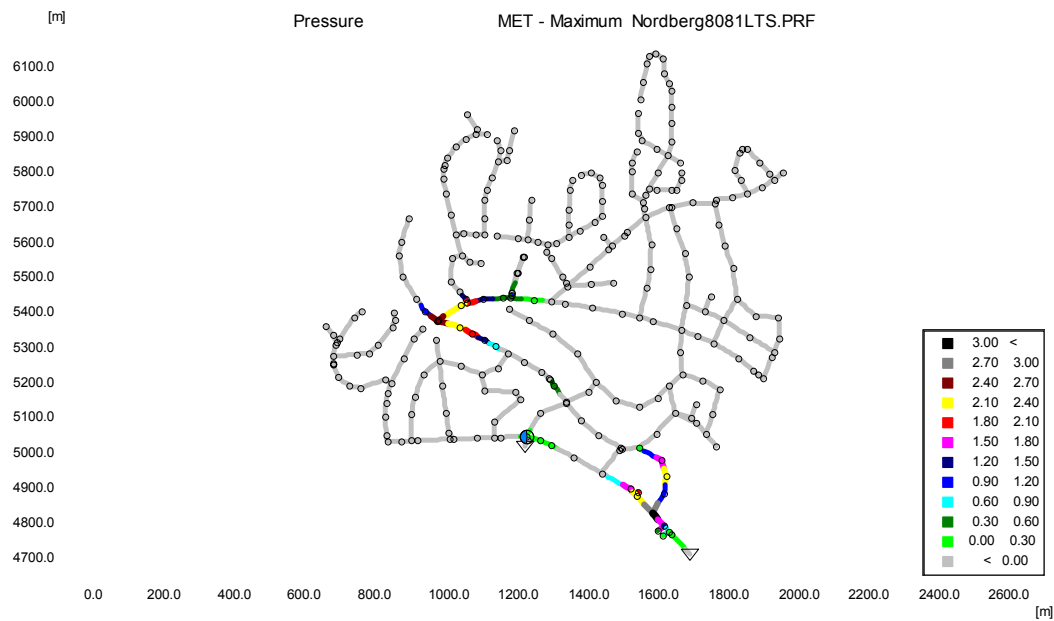


Figure 6 Pressure. Surge/Surface flooding computed as a difference of the computed water levels and the conduit ceiling. The Nordberg sewer network – The return period of events 20 years.

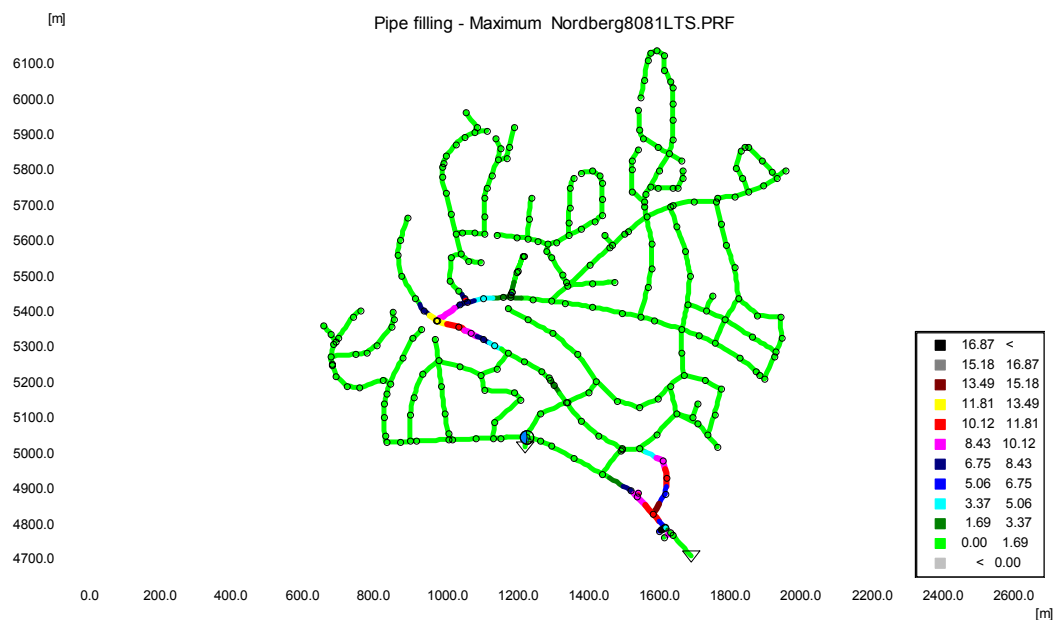


Figure 7 Pipe filling. Computed as the ratio between the water depth and the conduit height. The Nordberg sewer network – The return period of events 20 years.

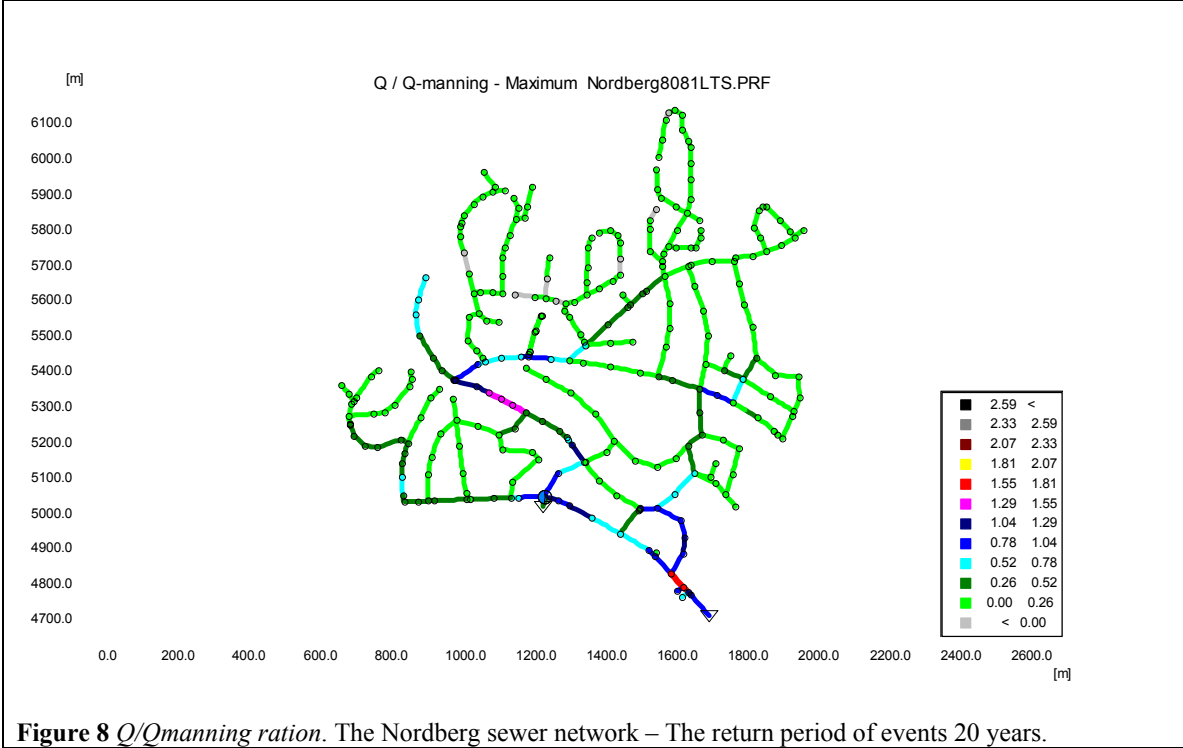


Figure 8 *Q/Q_{manning} ration*. The Nordberg sewer network – The return period of events 20 years.

An example of the interface of MIKE View and of some results in nodes (manholes) that can be displayed with this program is shown below.

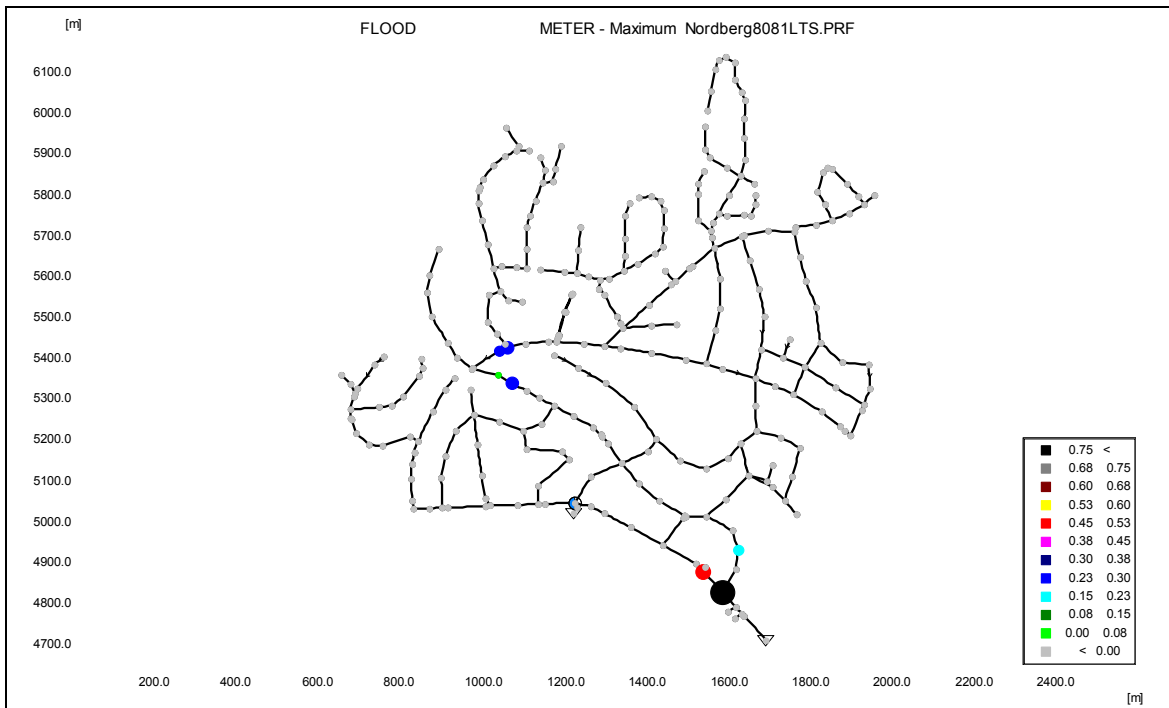


Figure 9 Flood. Surface flooding computed as a difference of the computed water level and the ground levels (+/-). The Nordberg sewer network – The return period of events 20 years.

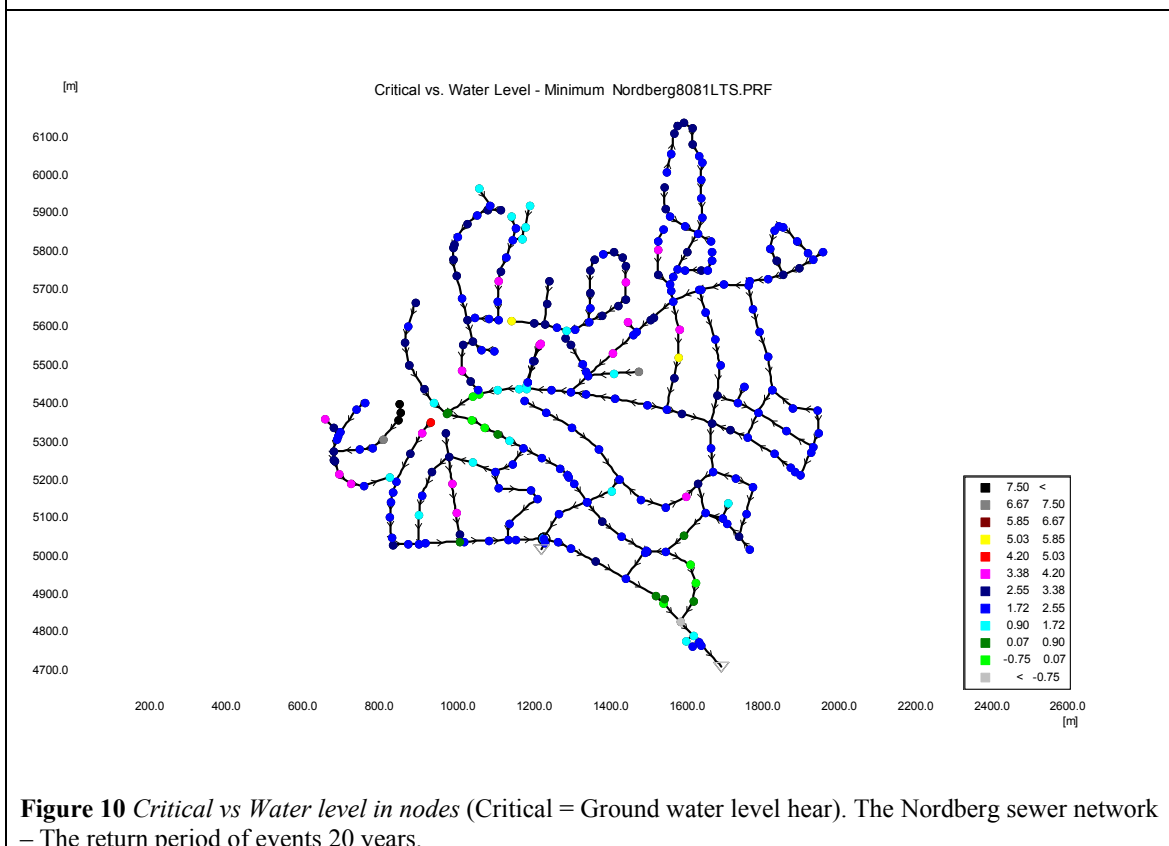


Figure 10 Critical vs Water level in nodes (Critical = Ground water level hear). The Nordberg sewer network – The return period of events 20 years.

Selecting and exporting MOUSE results/hydraulic performance indicators to the CARE-S Rehabilitation manager and other WPs

Surface flooding and other hydraulic criteria, such as surcharging for any location in the modelled network, are some of the most important physical and operational indicators (wPh/wOp) defined in the framework of CARE-S project. Some of information obtained from the model simulation such as *the total volume and frequency of CSO discharges together with the total infiltration volume entering the pipe sewer system* can be categorised as environmental and related to the environmental indicators (wEn) which can be treated as the surrogates for fundamental standards related to both the pollution discharge and the quality of the receiving water (WFD, E/E).

Use of the MOUSE hydraulic simulator with long time series of recorded rainfall (plus temperature and evaporation in cold climate urban areas) or synthetic storm events of different frequency and duration can provide information to deduce the performance of the existing network against the desired level/standard of services. As an example, the desired level of *surcharge* is that water levels within the sewer system must not exceeds the level of the lowest sewage inlet inside the house (basement) or rise to the ground surface level (cover level) and cause surface flooding for a 1 in 20 (EN 752 standard, CEN, 1996).

MOUSE gives as an output a range of indicators so that the PIs can be easily assessed/calculated. MOUSE simulation results are associated with links (pipes) and nodes (manholes/overflows/outlets) or related to the whole system (system items). All of them can be copied via the Windows Clipboard from MIKE View to spreadsheets and other applications so that the most important performance indicators can be calculated and exported to CARE-S WPs and Rehabilitation manager. **Figure 11** shows an example of the 'List' button in the 'Data Type Selection Window' in MIKE View environment wherefrom the time series list can be copied via the Windows Clipboard to spreadsheets and further to GIS dbf format. The time series list contains the time series identifiers, as well as maximum and minimum values of the item along with the associated times that the maximum and minimum occurred in the time series.

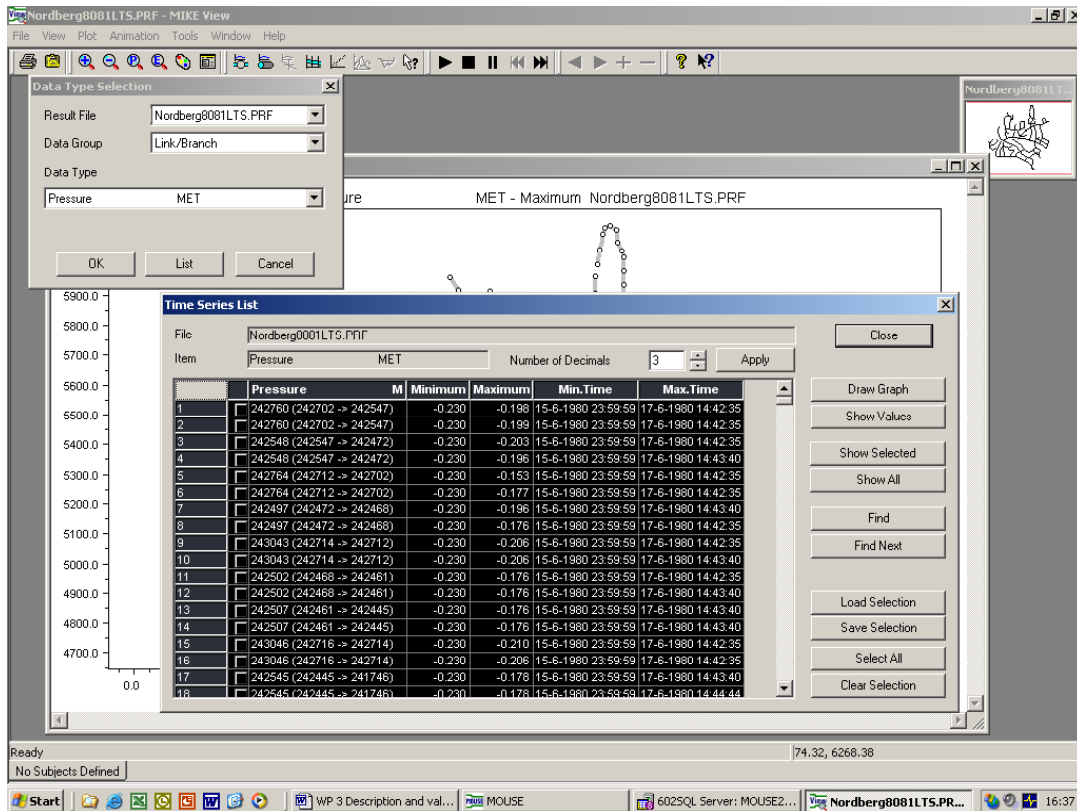


Figure 11 MOUSE simulation results can be copied via the Windows Clipboard from MIKE View to spreadsheets and other applications so that the most important performance indicators can be calculated.

The results in MOUSE model which can be used to deduce the PIs and conclusions from comparison between performance of the existing system and users- or standard-based performance level are:

Table 2 Time Series in links/Pressure - Computed as a difference of the computed water level and the conduit ceiling, (+/- m). (PI - Surcharging in sewer in dry and wet weather (frequency))

Pressure	MET	Minimum	Maximum	Min.Time	Max.Time
242760 (242702 -> 242547)	0.00	-0.230	-0.198	15.06.1980 23:59	17.06.1980 14:42
242760 (242702 -> 242547)	19.66	-0.230	-0.199	15.06.1980 23:59	17.06.1980 14:42
242548 (242547 -> 242472)	0.00	-0.230	-0.203	15.06.1980 23:59	17.06.1980 14:42
242548 (242547 -> 242472)	25.75	-0.230	-0.196	15.06.1980 23:59	17.06.1980 14:43
242764 (242712 -> 242702)	0.00	-0.230	-0.153	15.06.1980 23:59	17.06.1980 14:42
242764 (242712 -> 242702)	22.64	-0.230	-0.177	15.06.1980 23:59	17.06.1980 14:42
242497 (242472 -> 242468)	0.00	-0.230	-0.196	15.06.1980 23:59	17.06.1980 14:43
242497 (242472 -> 242468)	41.62	-0.230	-0.176	15.06.1980 23:59	17.06.1980 14:42
243043 (242714 -> 242712)	0.00	-0.230	-0.206	15.06.1980 23:59	17.06.1980 14:42
243043 (242714 -> 242712)	54.86	-0.230	-0.206	15.06.1980 23:59	17.06.1980 14:43
242502 (242468 -> 242461)	0.00	-0.230	-0.176	15.06.1980 23:59	17.06.1980 14:42
242502 (242468 -> 242461)	34.90	-0.230	-0.176	15.06.1980 23:59	17.06.1980 14:43

Table 3 Time Series in nodes/Critical level vs water level, (+/- m).
(PI - High sewer surcharging and frequency)

Critical vs. Water Level	Minimum	Maximum	Min.Time	Max.Time
242716	2.430	2.450	17.06.1980 14:42	15.06.1980 23:59
242714	2.486	2.510	17.06.1980 14:42	15.06.1980 23:59
242712	3.063	3.140	17.06.1980 14:42	15.06.1980 23:59
242702	2.688	2.720	17.06.1980 14:42	15.06.1980 23:59
242547	2.803	2.830	17.06.1980 14:42	15.06.1980 23:59
242472	3.126	3.160	17.06.1980 14:43	15.06.1980 23:59
242468	3.286	3.340	17.06.1980 14:42	15.06.1980 23:59
242461	2.406	2.460	17.06.1980 14:43	15.06.1980 23:59
242445	2.328	2.380	17.06.1980 14:43	15.06.1980 23:59
241746	2.411	2.460	17.06.1980 14:44	15.06.1980 23:59
241744	2.366	2.420	06.08.1980 01:46	15.06.1980 23:59
241742	2.360	2.410	06.08.1980 01:47	15.06.1980 23:59

Table 4 Time Series in links/Flood pipe – Computed as a difference of the computed water level and the ground levels, (+/- m).
(PI - Surface flooding and frequency)

FLOOD	METER	Minimum	Maximum	Min.Time	Max.Time
242760 (242702 -> 242547)	0.00	-2.722	-2.690	15.06.1980 23:59	17.06.1980 14:42
242760 (242702 -> 242547)	19.66	-2.828	-2.797	15.06.1980 23:59	17.06.1980 14:42
242548 (242547 -> 242472)	0.00	-2.828	-2.801	15.06.1980 23:59	17.06.1980 14:42
242548 (242547 -> 242472)	25.75	-3.160	-3.126	15.06.1980 23:59	17.06.1980 14:43
242764 (242712 -> 242702)	0.00	-3.141	-3.064	15.06.1980 23:59	17.06.1980 14:42
242764 (242712 -> 242702)	22.64	-2.722	-2.669	15.06.1980 23:59	17.06.1980 14:42
242497 (242472 -> 242468)	0.00	-3.160	-3.126	15.06.1980 23:59	17.06.1980 14:43
242497 (242472 -> 242468)	41.62	-3.340	-3.286	15.06.1980 23:59	17.06.1980 14:42
243043 (242714 -> 242712)	0.00	-2.510	-2.486	15.06.1980 23:59	17.06.1980 14:42
243043 (242714 -> 242712)	54.86	-2.661	-2.637	15.06.1980 23:59	17.06.1980 14:43
242502 (242468 -> 242461)	0.00	-3.340	-3.286	15.06.1980 23:59	17.06.1980 14:42
242502 (242468 -> 242461)	34.90	-2.460	-2.406	15.06.1980 23:59	17.06.1980 14:43

Table 5 Time Series in Nodes/Discharge weir.
(PI - Overflow discharge volume intermittent, annual)

DISCHARGE WEIR	Minimum	Maximum	Min.Time	Max.Time	Acumm.value
243421w1 (243421 -> 999039)	-0.000	0.182	27.07.1981 02:50	06.08.1980 01:47	434.497

Table 6 MOUSE Summary file (*.HTM). Continuity balance – Infiltration.

Continuity Balance

Start volume in Pipes, Manholes and Structures			0,951 m3
End volume in Pipes, Manholes and Structures			35,605 m3
Total inflow volume			
3.1 Runoff :	6746,502 m3		
3.2 Boundary :	0,000 m3		
3.3 DWF :	1834,402 m3		
3.4 Outlets (inflow) :	0,000 m3		
3.5 Infiltration :	320,640 m3		
	8901,544 m3	-->	8901,544 m3
Total diverted volume			
4.1 Weirs :	0,000 m3		
4.2 Pumps :	0,000 m3		
4.3 Spilling nodes :	0,000 m3		
4.4 Outlets :	8867,876 m3		
	8867,876 m3	-->	8867,876 m3
Water generated in empty parts of the system :			3,816 m3
Continuity balance = (2-1) - (3-4+5) :			-2,829 m3

From MOUSE LTS

MOUSE LTS provides the possibility to generate event-based and annual statistics. The 'return period' for a certain event is calculated by dividing the complete period covered by the simulation with the rank of the event.

Annual statistics for discharge at weirs, pumps and outlets and spatial extreme statistics for entire system are stored in a statistical result file (*.ERF). The results can be assessed from the MIKE View. MIKE View generates a series of tabular reports on the basis of the computed statistics. Any report can be generated in ASCII or in HTML format.

The MOUSE LTS model cannot yet cope with the computational complexity associated with the used of RDII model in combination with HD model. But it can be a useful tool in the case of use a simple runoff model in combination with HD model, Figure 12-15.

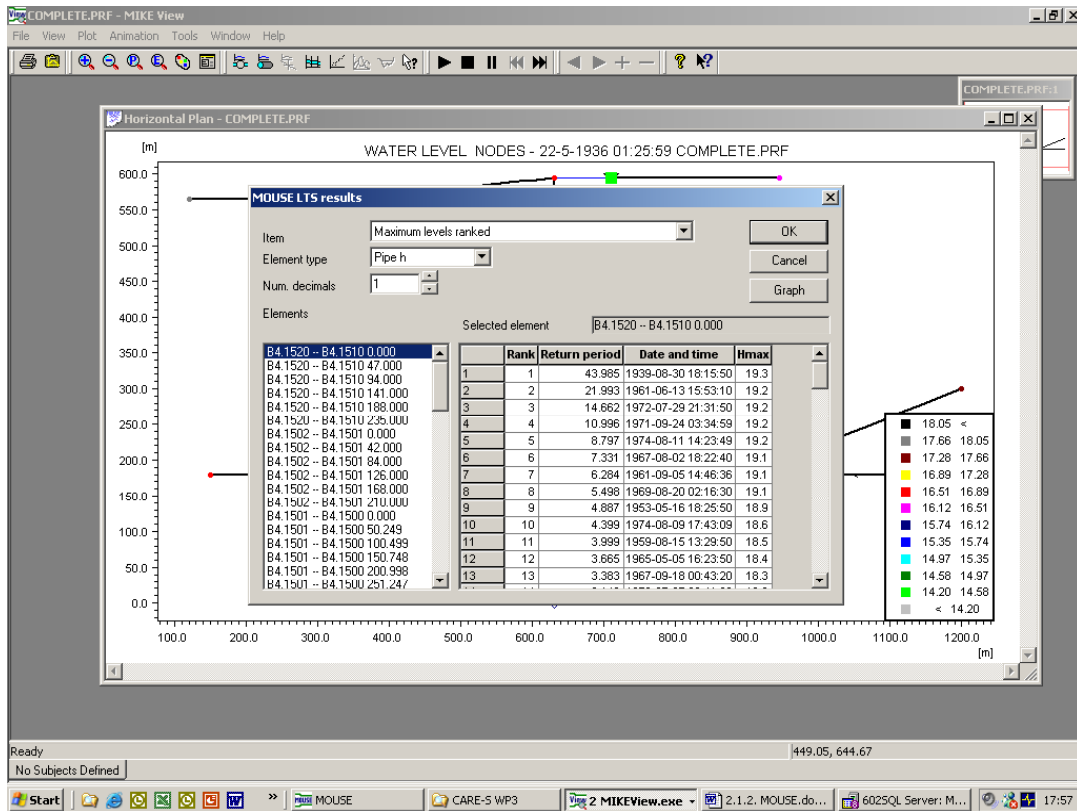


Figure 12 MOUSE LTS result file (*.ERF). Pipe h – maximum levels ranked.

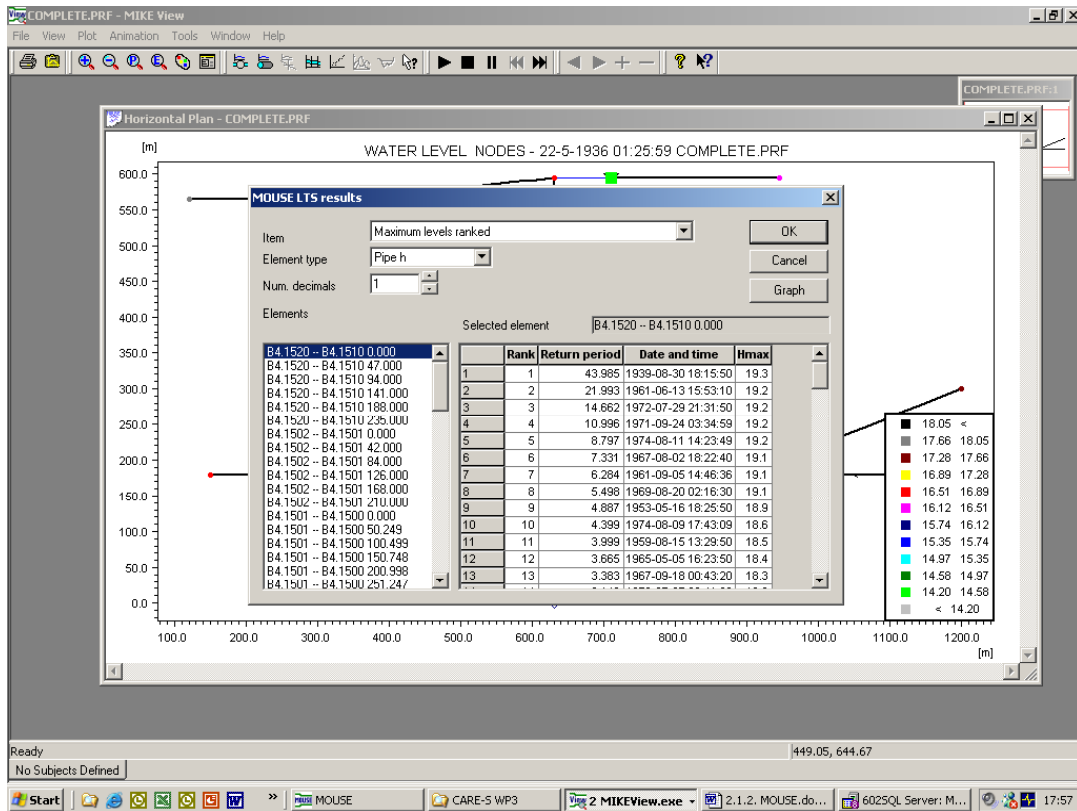


Figure 13 MOUSE LTS result file (*.ERF). Weir – Event discharge ranked.

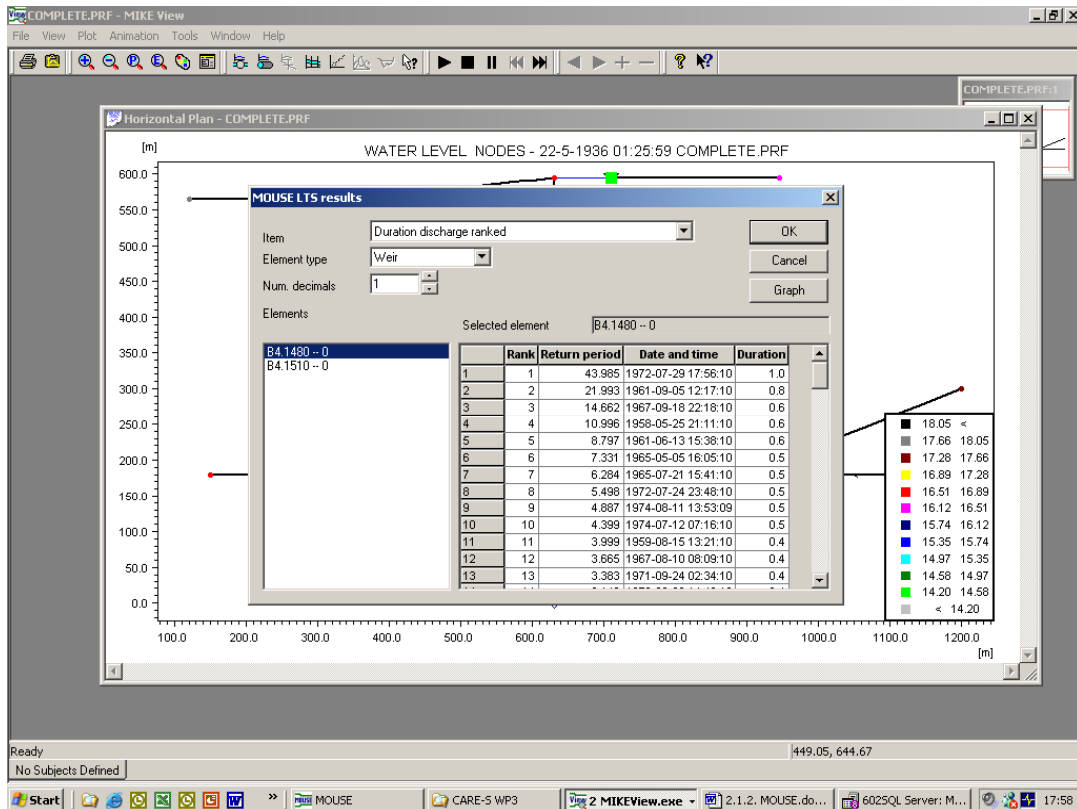


Figure 14 MOUSE LTS result file (*.ERF). Weir – Duration discharge ranked.

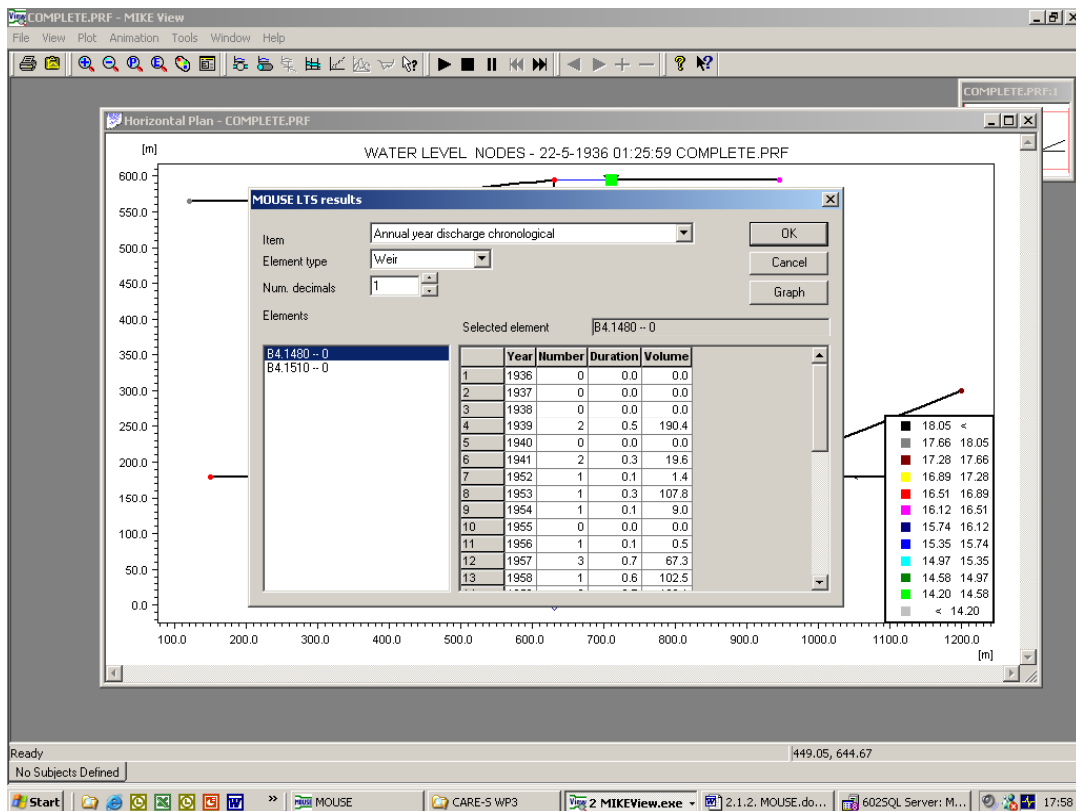


Figure 15 MOUSE LTS result file (*.ERF). Weir – Annual year discharge chronological.

2.1.2 INFOWORKS–(Appendix 2 - The Correggio case study in Reggio Emilia, Italy)

In InfoWorks CS (for Collection System), Wallingford Software has combined a Relational Database with Geographical Analysis to provide a single environment that integrates asset planning with detailed and accurate modeling. This product, when utilised with the new 'Time Series' simulation engine, allows planners and engineers to predict hydraulic behaviour and environmental impact following a rainfall event by providing fast, accurate and stable modeling of the key elements of wastewater and/or combined sewer systems. The software incorporates full solution modeling of backwater effects and reverse flow, open channels, trunk sewers, complex pipe connections and complex ancillary structures.

InfoWorks CS incorporates full interactive views of data using geographical plan views, long sections, spreadsheet and time varying graphical data. A new 3-Dimensional junction view has been introduced for improved visual presentation of manholes. Access to the underlying data is available from any graphical or geographical view.

Animated presentation of the results in Geographical Plan, Long Section and 3D junction views is standard, together with results reporting and flood frequency analysis using tables and graphs. The powerful 'Time Series' simulation engine provides automatic time-stepping and implicit numerical solution to optimise run time and ensures mathematical stability. The software contains comprehensive diagnostic error checking and warning, and rapid access to full on-line documentation that is integrated with the help system.

When the model has been created, InfoWorks allows you to simulate the behaviour of the network under a range of conditions. A variety of reporting tools are provided for analysis the results.

In particular, InfoWorks CS allows you to perform the following tasks:

- Manage and maintain network models over a long time period.
- Share model data among a workgroup of users.
- Import model data from other systems.
- View a geographical representation of the network on screen, with the network displayed over the top of a detailed local map.
- Enter rainfall and other time-series or event data (either recorded or synthetic).
- Perform hydraulic simulations to model the effects of a particular series of events.
- Produce reports and graphs, demonstrating the results of the simulations.

It is important to have an understanding of the following key concepts:

- a) Types of network that can be modelled by InfoWorks CS
- b) The modelling process
- c) Data management
- d) Reporting facilities
- e) GIS Components

These concepts are described below.

a) Types of network

InfoWorks CS can model networks of stormwater or wastewater drainage systems (or a combined stormwater and wastewater system). A network contains all the information needed to describe the drainage system. Each network is modelled as a collection of subcatchment areas that drain to nodes (manholes or grade breaks) which are joined by links (conduits, pumps etc.)

For a network containing two separate drainage systems (both stormwater and wastewater), two sets of overlapping subcatchment areas can be modelled.

b) Modeling processes

InfoWorks CS incorporates the sophisticated HydroWorks modelling engine for simulating the behaviour of the network under many different conditions. When a network has been set up, you can supply event data (real or synthetic) representing the volume of water entering different points on the network over a period of time. The modelling engine then runs a simulation to

demonstrate the effect of the water on the network, so that weak points in the network can be identified.

Many different models can be created for the same network, based on different event data and various periods of time, from a few hours to a number of years. Models can also be run using just a part of the network.

c) Managing data

All your model data and results are stored in a master database. The master database consists of a set of model definitions and will include the results that are created when you run simulations. Most organisations will have only one active database, containing all their networks and models. However, you may decide to create further databases. If necessary, data can be exported from one database to another.

The system includes comprehensive database management facilities, which let you organise your work logically and efficiently. Using these facilities you can see the overall structure of the database, break it down into its objects and view any part of the data itself.

A master database is created using the separate InfoWorks Database Administrator program, which is run directly from Windows. You can also use the Database Administrator to create and update transportable databases and archives. Transportable databases allow you to copy of some or all the data from one InfoWorks master database to another; archives allow you to store data for future use and for backup purposes.

d) Reporting facilities

The modelling engine runs simulations by applying specific rainfall events to your network. The results that are generated will predict the effect on the network of these events (such as a specific pattern of rainfall over a given number of hours). InfoWorks allows you to review the results in several different ways:

- Calculating summary statistics.
- Replaying results on the views of the network.
- Creating many different types of graph.
- Exporting results to files and to other programs (such as a spreadsheet or word processor).

e) GIS components

Networks can be displayed against the background of a user-supplied map. In the UK, this will usually be a detailed Ordnance Survey map. Other user-defined or external data can be displayed as additional layers on the GeoPlan View. InfoWorks provides facilities for importing and exporting data from a standalone professional GIS package.

The GIS systems currently supported by InfoWorks are:

MapInfo Professional and MapX	MapX provides functionality within InfoWorks and allows you to display additional data and maps in the MapInfo table format. You can import from and export to MapInfo Professional, which is a stand-alone package.
ESRI ArcView and Map Objects	MapObjects allows you to display additional data and maps in the ESRI layer format within InfoWorks. You can import from and export to ArcView.

You should choose the MapX or MapObjects version of InfoWorks depending on the format you prefer to use for imported data. You can export to both MapInfo Professional and ArcView regardless of which internal GIS component is used by InfoWorks.

External GIS

InfoWorks can communicate directly with two external GIS packages, MapInfo Professional from MapInfo Corporation and ArcView from ESRI. InfoWorks data can be exported to either of these packages for viewing and additional analysis. Data can be created or imported into the external GIS package, edited if necessary, and then loaded into InfoWorks for use with InfoWorks models.

Inference of Missing Data

Missing network values can be inferred from existing data, allowing you to fill in gaps in your data with reasonable values. For example, if the width of a conduit is missing but you have the width of the conduit immediately upstream, it is reasonable to assume that the missing width is the same.

InfoWorks CS includes a set of inference rules that you can apply to all or part of the network. The use of a common set of rules within the software means that all missing data can be inferred to the same standard, rather than depending on who does the inference or when it is done. InfoWorks CS has built-in inference rules for conduit sizes, conduit invert levels, node coordinates and node ground levels. In addition, every manhole and junction can also be scanned and the appropriate Headloss type and coefficient applied to the end of each connecting pipe, based on the angle of entry/exit from the manhole.

You can infer values for either the currently selected objects or the whole network. You can also choose which missing values are to be inferred and select the user-defined data flag that is to be applied to all inferred values. The use of a data flag is important for auditing purposes, as it allows you to distinguish between inferred data and data created in other ways.

Engineering Validation

The InfoWorks Engineering Validation option performs additional checks on network data to ensure that it is consistent with expected engineering values.

Engineering Validation allows you to define your own set of validation rules, thus allowing you to modify the way in which Engineering Validation is performed, depending on the data concerned. The items are contained in engineering validation groups for easy reference. By setting up separate groups and objects you can define the exact criteria you wish to apply. An Information message is displayed in the Output window for each item of data that is outside normal values. You can use this window to investigate the fields for which messages have been

included. When you open an engineering validation object for the first time you are supplied with the default validation rules in the Engineering Validation dialog.

Storage Compensation (additional manhole storage - MADD)

In any system, there will always be a certain amount of storage available which is not explicitly defined in the model. It is possible that this additional storage volume, representing for example, small pipes from individual properties entering the system, may not be adequately represented in your InfoWorks model.

In the early days of model building, it was very difficult to quantify this storage, so an algorithm was developed at HR Wallingford in the mid 1980's. This set out some rules upon which to base the amount of 'lost' storage that needed to be added into the model to correctly account for the extra volume of storage available below ground in reality.

Today, GIS systems are much more powerful than 20 years ago, and the amount of information stored on such systems is of much greater accuracy and detail. It is therefore questionable if the MADD approach is still needed when building a model directly from today's GIS systems.

However, if a model is predicting surcharging and flooding contrary to observations, then a possible cause is the absence in the model of the 'extra storage' that is available below the ground in the network of pipes and manholes. To compensate for this 'lost storage', one approach might be to apply the MADD algorithm to the network model. Wallingford® has added extra fields for additional chamber storage and additional shaft storage, into which the user can specify some extra storage. Eventually, this tool will be expanded to account for lost storage on merging and pruning and for Preissmann slot compensation.

Model build and simplification functionality

The selection tools within InfoWorks CS allow the geographical selection of model data, which then can be globally modified, deleted or copied and used to create a new model version.

Geographical model building tools

The InfoWorks Geographical View allows the node/link network to be shown in conjunction with raster or vector data in MapInfo or ArcView format. InfoWorks CS supports the on-screen creation of additional nodes, links and sub-catchment areas via the Geographical view in conjunction with a map background, making the addition of new data intuitive, simple and quick to do. The edit tools that allow the modification of node location and sub-catchment boundaries are also intuitive, simple and quick to use.

Sub-catchments and Area-take-off

InfoWorks CS offers the direct import or graphical creation of sub-catchment boundaries, over vector map backgrounds. This provides a geographical representation of the contributing area for each manhole, and allows automatic calculation of the total area. In addition, the different surface types and areas can be calculated using accurate area take-off from a vector map containing road and roof areas. This feature alone provides a significant timesaving when compared with the manual process, and it is repeatable and auditable. InfoWorks CS also provides quick and accurate calculation of population from 'Address-point' or 'building seed' map data.

Model merging

The merging of existing models into a larger macro model is performed quickly and accurately, utilising the copy and paste networks facility within InfoWorks CS. To merge, for example, 5 dendritically numbered models (of approx. 400 nodes each) into a single model within HydroWorks, could take a couple of days and might potentially include a number of

'unintentional' errors. Within InfoWorks CS this task can be performed in minutes, and accurately represent the source models.

Real Time Control Module

Real time control is the remote manipulation of control structures within a drainage system, based on conditions at any point in the system, in order to optimise storage and operation. You can apply Real Time Control (RTC) to individual, isolated, ancillary structures to provide local control of flows. It can also make global management of flows possible throughout an entire network. For example, a level meter at the top of the sewer system may operate a sluice gate near the treatment works.

You can combine RTC modeling parameters to build up complex rules. This gives you enormous scope to explore the potential storage capacity and optimal operating patterns within the system being modeled by routing and storing flows. The components of RTC for the management of sewer networks are the use of sensors in the drainage network to monitor flows continuously. By using telemetry, you can implement a control system using local operating rules, allowing you to change the state of the ancillary structures such as pumps, sluice gates and weirs during a storm.

UPM (SIMPOL) reporting tools

In a move towards a fully Integrated Catchment Management approach, InfoWorks CS now incorporates 'UPM' style post-processing tools and 'SIMPOL' style input and output as a standard part of the InfoWorks CS reporting functionality. This function is available without having to purchase the detailed Water Quality Module.

Water Quality Module

The InfoWorks CS Water Quality Module is designed to help engineers develop cost effective AMP3 and UPM2 solutions for pollution and sedimentation problems. The Water Quality Module can model physical processes such as the first foul flush, sediment built up behind closed gates and penstocks. Using the Water Quality Module, engineers can control pollution by targeting the SSOs and CSOs problems, and predict quality components such as the volume of spillage and flooding. This leads to recommendations for corrective action through storage and real time control.

The InfoWorks Quality Module contains facilities to model the main water quality parameters such as TSS (total suspended solids), BOD (biochemical oxygen demand, attached & dissolved), COD (chemical oxygen demand, attached & dissolved), NH₄ (ammonia, dissolved), TKN (total Kjeldahl nitrogen, attached & dissolved), and Tph (total phosphorus, attached & dissolved). It also allows for user-defined pollutants, bed-load sediment fractions, and the modeling of bed-load movement separately from the suspended sediment movement.

Physical process models within the Water Quality Module include a Surface pollutant build-up Model, Surface pollutant washoff Model, Gully pot Model, Wastewater profile Generator, Sediment transport Model and an in-pipe water quality Model.

Clearer graphical presentation and results analysis using Themes and Contours

InfoWorks CS supports 'Storm' and 'Combined/Foul' systems within the same model, and represents them using different colours. In addition, it provides support for separate contributing areas and floodable areas as separate data items. 'Break Nodes' have been introduced to prevent the need to create 'dummy' manholes in the model.

Themes can be used to highlight parameters graphically on the GeoPlan view. The chosen parameter could be a static network parameter with a fixed display, or you can use a theme to highlight a results parameter during a replay of a simulation. In this case the display will be continually updated during the course of the replay. InfoWorks allows you to set up exactly how your selections are displayed. You can set up the display entirely manually, or use the built in options to automate the process.

You can also display a contour plot of many different types of data on the GeoPlan View. Like themes, the chosen parameter could be a static network parameter (such as ground level), or a results parameter (such as water level) during a replay of a simulation. In this case the contours will be continually updated during the course of the replay.

A very useful option allows you to draw an arrow from the centre of each sub-catchment to the node (or nodes) that receive direct runoff from that catchment. Lines with a direction arrow are drawn from the centre of the sub-catchment to the receiving node(s).

InfoWorks CS provides simple and fast creation of summary tables for conduit surcharging, node flood volumes and Return Period Analysis, including the creation of 'x:x' diagrams within InfoWorks itself. By exporting this data to the GIS (MapInfo or ArcView) it is also possible to create simple and accurate 'x:x' drawings for other people within minutes. This is a task that could traditionally take days to do manually in AutoCAD.

User Permissions

You have the option to apply a set of simple User Access Permissions at both the database level and the catchment group level. With access permissions activated there are three types of InfoWorks user:

Database Owner - a database owner has full administrative powers over the database.

Catchment Group Owner - appointed by a Database Owner, the owner of a Catchment Group has full edit and delete powers over that Catchment Group.

Database User - a Database User has read-only access to the database. Catchment Group Owners are also Database Users and have read-only access to Catchment Groups that they don't own.

Hardware configurations

The minimum hardware requirements for InfoWorks CS are a Pentium II 266MHz PC with 128Mb RAM, and a 1024x768 high-resolution screen. InfoWorks CS requires the Windows 95/98/Me, Windows NT 4.0 or Windows 2000/XP operating system (NT4.0/2000/XP Preferred). The software is distributed on CD-ROM, and is a full 32-bit application, supporting long file names, and IT configurations involving Local Area Networks (LAN's).

Input and Output Data

The next tables present the list of all the data required by the model. Input data are classified depending on the input file format.

The last tables include the general output produced by the model. A much more detailed description of them is shown in the next tables.

INPUT

*.dsd file

Parameter	Data type	Units	Definition
Land use index	ASCII		You must specify a default land use index in the node defaults record.
Soil class	ASCII		
Infiltration flow	ASCII	$\frac{\text{m}^3}{\text{s} \cdot \text{km}^2}$	
Rainfall profile index	ASCII		Where you wish to use records from several rain gauges to represent spatial variation, this gives the number of the gauge associated with a contributing area.
Flood type	ASCII		The flood type defines what happens to water that floods out of the drainage system at the node
Flood area 1	ASCII		This defines the first part of a double conical flood storage volume
Flood depth 1	ASCII		This is the depth of flood water in m at which the flood water spreads over the flood area 1
Flood area 2	ASCII		This defines the second part of a double conical flood storage volume.
Flood depth 2	ASCII		This is the depth at which the flood water spreads over the flood area 2
Ground level	ASCII		All levels must be above a common datum, and ideally choose this so that all levels in the system are above datum
Contributing area	ASCII		This is the area of the sub-catchment in hectares draining to this node and liable to flooding. You must define this if the area is liable to flooding
Land use index for DWF	ASCII		It refers to a definition in the land use definition file
Population	ASCII		This is the number of people in the sub-catchment
Dry weather flow index	ASCII		This is an integer between zero and 99. The software uses this index to determine the DWF profile for this node
Infiltration flow index	ASCII		
Rainfall profile index	ASCII		This gives the number of the gauge used for this contributing area where you wish to use records from several raingauges to represent spatial variation
Contributing surfaces	ASCII		It is possible to divide the contributing area of the sub catchment among three surface types. Conventionally, these are paved, roof and pervious areas respectively
Runoff index	ASCII		Give the runoff index for each surface. This indicates to the software which of the surface types in the runoff data to use. Any value between 1 and 99 is valid
Pollution index	ASCII		Give the pollution index for each surface. This indicates to the software which of the pollution models to use. Values between 1 and 99 are valid
Connectivity (%)	ASCII	%	The connectivity defines the percentage of the sub-catchment connected
Ground (catchment) slope	ASCII		Use this field to define the average ground slope for this sub-catchment
Catchment length	ASCII	m	The default value is the radius of the (assumed circular) catchment area for this node.
Manhole level 1 (roof)	ASCII	m	This defines the level of shaft storage in the manhole
Manhole area 1 (shaft)	ASCII	m^2	This defines the plan area of the storage in the manhole between manhole level 1 and ground level
Manhole level 2 (floor)	ASCII	m	This defines the level at which the manhole could provide more storage than that in the sewer or channel itself.
Manhole area 2 (chamber)	ASCII	m^2	This defines the plan area of the storage in the manhole between manhole levels 2 and 1

***.wwg file**

<i>Parameter</i>	<i>Data type</i>	<i>Units</i>	<i>Definition</i>
Average flow	ASCII	litres/person/day	This is the average dry weather flow
Average concentration of sediment fractions	ASCII	mg/l	This is the average concentration in the dry weather flow
Average concentration of dissolved pollutants	ASCII	mg/l	This is the average concentration of pollutants in the dry weather flow
Potency factor	ASCII	mg/mg	This is a multiplier factor used to express a pollutant's potency in terms of each particulate fraction
Calibration profiles timestep	ASCII	hours	This is the time increment at which the pollutant and flow multipliers will be given, to create time varying profiles.
Weekday flow multipliers	ASCII		These are the dimensionless multipliers which make up the 24 hour profile for the weekday flow.
Weekday pollutant multipliers	ASCII		These are the dimensionless multipliers which make up the 24 hour profiles for the weekday dissolved pollutants and weekday sediment fraction 1
Weekend flow multipliers	ASCII		These are the dimensionless multipliers which make up the profile for the weekend flow
Weekend pollutant multipliers	ASCII		These are the dimensionless multipliers which make up the profiles for the weekend dissolved pollutants and weekend sediment fraction 1
Monthly flow multipliers	ASCII		These are flow multipliers for each month of the year.
Monthly pollutant multipliers	ASCII		These are multipliers for the dissolved pollutants and for sediment fraction 1 for each month of the year
Daily flow multipliers	ASCII		These are the dimensionless multipliers which make up the design diurnal flow profile
Daily pollutant multipliers	ASCII		These are the dimensionless multipliers which make up the design diurnal profile for dissolved pollutants and sediment fraction 1
Water Quality Component	ASCII		This is the name of the pollutant to which you wish to apply the multipliers
Pollutant multipliers	ASCII		These are the dimensionless multipliers which make up the local profile for the pollutant

***.red file**

<i>Parameter</i>	<i>Data type</i>	<i>Units</i>	<i>Definition</i>
Start date	ASCII	Dd/mm/yy	Give the start date of the event
Start time	ASCII	hh:mm	Give the start time
Timestep	ASCII	sec	Give the timestep of the data
Number of profiles	ASCII		The number of rain gauges for which you wish to enter data in the file
Urban catchment wetness index	ASCII		This defines the antecedent wetness of the catchment for the Wallingford runoff model.
Antecedent rainfall depth	ASCII		This is the rainfall in the hour immediately before the storm that has caused wetting of the catchment surface
Evaporation	ASCII	mm/day	This is the rate of evaporation
Wetness index	ASCII		This defines the catchment wetness for use with the SCS runoff model
Local antecedent rainfall depth	ASCII		This is the rainfall in the hour immediately before the storm that has caused wetting of the catchment surface
Areal reduction factor	ASCII		This field has two uses depending on whether the storm is synthetic or actual.
Local evaporation	ASCII	mm/day	Evaporation at each rain gauge
Local SCS wetness index	ASCII		

***.wpf file**

<i>Parameter</i>	<i>Data type</i>	<i>Units</i>	<i>Definition</i>
Buildup decay factor	ASCII		This is a coefficient used in the surface buildup equation.
Rainfall erosion calibration coefficients	ASCII		There are three rainfall erosion calibration coefficients, c1, c2 and c3
Pollutant name	ASCII		You must define the name of each of the pollutants you wish to washoff from the catchment surface in the pollutants record

OUTPUT

File name	Description	Value
<model_name>.HYD	Depth hydrographs for gauged pipes	
	May be converted into .lev file using either:	
	i. The Flow Survey Converter	
	ii. The [Model] Results Files option.	
		ASCII
<model_name>.HYQ	Discharge hydrographs for gauged pipes	
	May be converted into .qin file using either:	
	i. The Flow Survey Converter	
	ii. The [Model] Results Files option.	
<model_name>.HYV	Velocity hydrographs for gauged pipes	
<model_name>.QSR	Water quality simulation results	
	This is a binary file that cannot be viewed in a text editor.	
<model_name>.CSI	Water quality mass concentration and mass flow	
<model_name>.CB1	Details of biochemical oxygen demand attached to sediment fraction 1 mass concentration hydrographs	
<model_name>.CC1	Details of chemical oxygen demand attached to sediment fraction 1 mass concentration hydrographs.	
<model_name>.CK1	Details of total Kjeldahl nitrogen attached to sediment fraction 1 mass concentration hydrographs.	
<model_name>.CP1	Details of total phosphorus attached to sediment fraction 1 mass concentration hydrographs.	
<model_name>.C11	Details of user defined pollutant 1 attached to sediment fraction 1 mass concentration hydrographs.	
<model_name>.C21	Details of user defined pollutant 2 attached to sediment fraction 1 mass concentration hydrographs.	

<model_name>.C31	Details of user defined pollutant 3 attached to sediment fraction 1 mass concentration hydrographs.	ASCII
<model_name>.C41	Details of user defined pollutant 4 attached to sediment fraction 1 mass concentration hydrographs	
<model_name>.CBD	Details of biochemical oxygen demand dissolved mass concentration hydrographs.	
<model_name>.CCD	Details of chemical oxygen demand dissolved mass concentration hydrographs.	
<model_name>.CND	Details of ammoniacal nitrogen (NH4) dissolved mass concentration hydrographs.	
<model_name>.CKD	Details of total Kjeldahl nitrogen dissolved mass concentration hydrographs.	
<model_name>.CPD	Details of total phosphorus dissolved mass concentration hydrographs.	
<model_name>.C1D	Details of user defined pollutant 1 dissolved mass concentration hydrographs.	
<model_name>.C2D	Details of user defined pollutant 2 dissolved mass concentration hydrographs	
<model_name>.C3D	Details of user defined pollutant 3 dissolved mass concentration hydrographs.	
<model_name>.C4D	Details of user defined pollutant 4 dissolved mass concentration hydrographs.	
<model_name>.CBT	Details of biochemical oxygen demand total concentration hydrographs.	
<model_name>.CCT	Details of chemical oxygen demand total concentration hydrographs.	
<model_name>.CKT	Details of total Kjeldahl nitrogen total concentration hydrographs.	
<model_name>.CPT	Details of total phosphorous total concentration hydrographs.	
<model_name>.C1T	Details of user-defined pollutant 1 total concentration hydrographs.	
<model_name>.C2T	Details of user-defined pollutant 2 total concentration hydrographs.	
<model_name>.C3T	Details of user-defined pollutant 3 total concentration hydrographs.	ASCII
<model_name>.C4T	Details of user-defined pollutant 4 total concentration hydrographs.	
<model_name>.MS1	Details of sediment fraction 1 mass flow hydrographs.	
<model_name>.MB1	Details of biochemical oxygen demand attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.MC1	Details of chemical oxygen demand attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.MK1	Details of total Kjeldahl nitrogen attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.MP1	Details of total phosphorous attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.M11	Details of user-defined pollutant 1 attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.M21	Details of user-defined pollutant 2 attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.M31	Details of user-defined pollutant 3 attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.M41	Details of user-defined pollutant 4 attached to sediment fraction 1 mass flow hydrographs.	
<model_name>.MBD	Details of biochemical oxygen demand dissolved mass flow hydrographs.	
<model_name>.MCD	Details of chemical oxygen demand dissolved mass flow hydrographs.	
<model_name>.MND	Details of NH4 dissolved mass flow hydrographs.	
<model_name>.MKD	Details of total Kjeldahl nitrogen dissolved mass flow hydrographs.	
<model_name>.MPD	Details of total phosphorous dissolved mass flow hydrographs.	

<model_name>.M1D	Details of user-defined pollutant 1 dissolved mass flow hydrographs.	
<model_name>.M2D	Details of user-defined pollutant 2 dissolved mass flow hydrographs.	ASCII
<model_name>.M3D	Details of user-defined pollutant 3 dissolved mass flow hydrographs.	
<model_name>.M4D	Details of user-defined pollutant 4 dissolved mass flow hydrographs.	
<model_name>.MBT	Details of biochemical oxygen demand total mass flow hydrographs.	
<model_name>.MCT	Details of chemical oxygen demand total mass flow hydrographs.	
<model_name>.MKT	Details of total Kjeldahl nitrogen total mass flow hydrographs.	
<model_name>.MPT	Details of total phosphorous total mass flow hydrographs.	
<model_name>.M1T	Details of user-defined pollutant 1 total mass flow hydrographs.	
<model_name>.M2T	Details of user-defined pollutant 2 total mass flow hydrographs.	
<model_name>.M3T	Details of user-defined pollutant 3 total mass flow hydrographs.	
<model_name>.M4T	Details of user-defined pollutant 4 total mass flow hydrographs.	

2.1.2.1 Data import

Data may be imported from a variety of sources:

- InfoWorks supports the import of all network and event data from WASSP, WALLRUS and HydroWorks.
- The direct links between InfoWorks and MapInfo Professional enable data in MapInfo to be transferred directly into the InfoWorks database for model building. Such data includes subcatchment (contributing) areas, area breakdown (road/roof polygon areas) and population data (address point count).
- MapInfo Professional 5.0 supports map data import from AutoCAD DXF and DWG files, ESRI Shape and Intergraph/MicroStation Design. MapInfo also allows the import of data in delimited ASCII, dBase DBF and Microsoft Access file formats.
- InfoWorks is delivered with an importer that enables asset data held in a Microsoft Access database to be imported directly into the InfoWorks master database. The importer may be configured to map the data schema of the Access database to that within InfoWorks.
- InfoWorks can import specific data sets from manhole databases, including Thesis, STC25, STC26 and FastSTC.
- InfoWorks also supports the import of data from .CSV files.

2.1.2.2 Data export

Data may be exported in a number of formats:

- InfoWorks supports the export of network and event data required for simulations in HydroWorks: for instance DSD files (nodes/links), LUD files (land use), RPF files (runoff parameters), CRD files (catchment runoff data), SIM files (simulation parameters), SHP files (pipe cross-section data), RED files (rainfall), QIN files (inflow), LEV files (level), WWG files (wastewater), TWG files (trade waste) and RTC files (real time control).
- InfoWorks supports the export of network data and maximum results to specific layers in MapInfo Professional 5.0.
- InfoWorks allows the export of network and results data to CSV files. These may subsequently be imported into Microsoft Access, Excel or similar applications.
- InfoWorks can export PRN (text) files and HYD, HYQ and HYV (time-varying event) files.
- MapInfo Professional 5.0 supports data export to AutoCAD DXF and DWG files, ESRI Shape and Intergraph/MicroStation Design. MapInfo also allows the export of data in delimited ASCII, dBase DBF and Microsoft Access file formats.

2.1.2.3 Simulations

The behaviour of a network under particular conditions is modelled by running simulations. The simulations test the effects of a given flow of water through the network over a period of time and identify weaknesses in the system, such as nodes that are liable to flood under given conditions.

The details of particular simulations and their results are held on the database as *runs*. When you set up a run, you must specify one or more rainfall events to be used in the calculations. The system creates one *simulation* for each rainfall event; therefore each run will contain one or more simulations. Runs are grouped together within a catchment group as *run groups*. Each run group will contain one or more runs.

The simulations must be *scheduled*; this involves setting up the parameters for the simulations. The InfoWorks Simulation Controller then performs the simulations.

SIMULATION PARAMETERS

The simulation parameters govern how the engine performs the model calculations in the hydraulic simulation. You do not normally need to amend these parameters, as the default values have been chosen for optimum accuracy and performance. However, you can alter the way the engine behaves should it be necessary.

This section describes all the ways of viewing simulation results in InfoWorks, and of comparing results with observed data.

Reviewing the Progress of the Simulation

The progress of the simulation can be reviewed in one of three ways. The simplest is the Log Report. This contains the output from the simulation engine, and contains any warnings or other important information generated at each timestep during the model run.

The simulation can be replayed with the output displayed on either the GeoPlan View, the Long Section view, or the results versions of the Grid Views. The GeoPlan provides powerful tools for graphically displaying the changing state of the network over the period of the simulation.

Summaries of the Simulation

You can view two different text reports. The log report describes the progress of the simulation. The PRN report contains comprehensive results.

You can produce graph reports in several different configurations:

- single simulation
- multiple simulation
- multiple location
- observed versus predicted

There are other graphing options available.

Statistical Reports provide a way of reporting on selected parameters for selected objects. The reports are formatted to make them suitable for printing and distribution.

You can create grid reports for Flood Volume and Return Period Analysis.

More detailed results for specific Network Objects

You can display a grid report or graph of the time varying results for a single network object.

Results Export

You can export the results to MapInfo or ArcView. See Opening InfoWorks Data in MapInfo or Opening InfoWorks Data in ArcView.

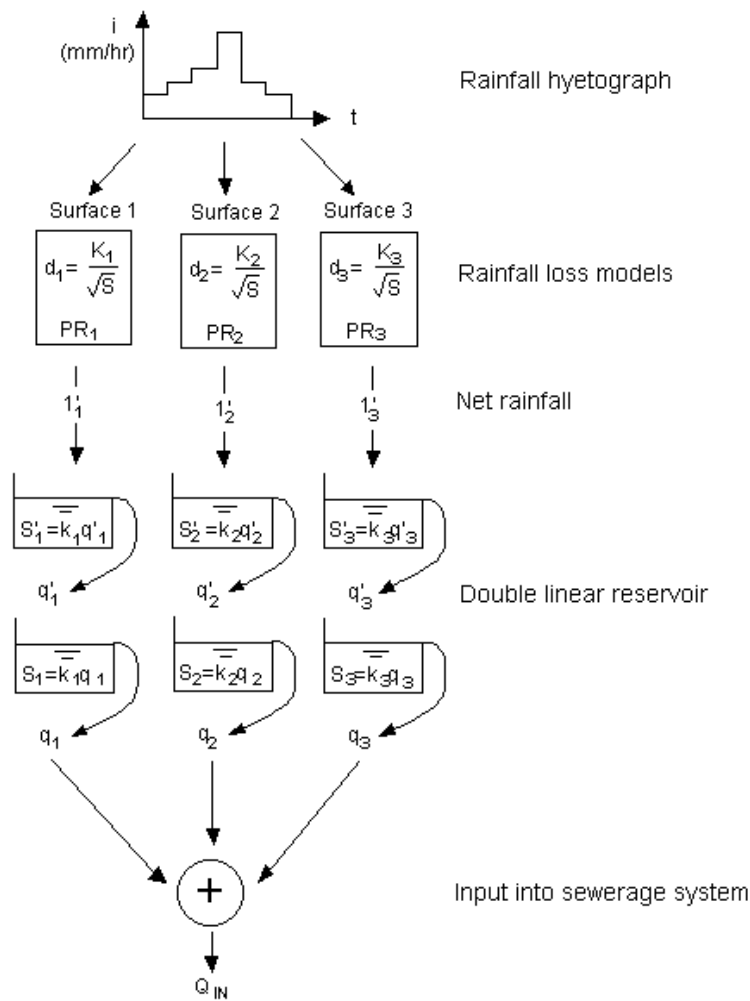
You can also export to a PRN, CSV or HydroWorks file.

Water quantity modelling is basically demanded to Runoff Model (rainfall-runoff transformation) and to Hydraulic model (flow propagation).

2.1.2.4 Runoff model

The runoff data describes the characteristics of the ground surfaces in the system, and the rainfall-runoff model which is used for each. This defines how much of the rainfall falling on the catchment becomes runoff and how quickly it enters the drainage system. The Wallingford

Procedure rainfall-runoff models comprise a set of models enabling engineers to represent the transformation of rainfall into runoff for both urban and rural catchment areas contributing to piped or channelled drainage systems. Irrespective of the model option selected there is a basic conceptual framework underpinning the use of the model. First, the overall catchment model is divided into a series of subcatchments. Each subcatchment can then be subdivided into surface types. A surface type is defined for each distinct category of surface. Usually between two and five surface types are sufficient to describe the different areas of a model. Each subcatchment can use up to 12 of the defined surface types.



Overview of surface hydrology model

Twenty-one surface types are defined by default in the database. The default surface types for the default land uses correspond to the three surfaces for the Wallingford model for UK urban runoff. The three surfaces are shown below:

Surface Type	Definition
10	Paved Surface
20	Pitched Roofs
21	Pervious Areas

These surface types will usually be used without alteration for UK catchments. However, in special circumstances, or for use in other countries surface, types can be redefined. Examples of alternative surface types that might be defined are:

- Pervious areas with dense vegetation
- Typical residential areas
- Road drainage
- Industrial areas

Three concurrent processes model the rainfall-runoff:

- Initial losses (depression storage)
- Continuing losses (infiltration)
- Overland flow routing

Conventionally the first two surface types represent impervious (road and roof) areas, whereas the remainder of the subcatchment represents pervious surfaces. The surface runoff model is divided into two major process models, representing rainfall losses and runoff routing, each of which can act independently on the surfaces in each subcatchment (see diagram above).

If you wish to modify the parameters, use the *Runoff Surfaces* tab in the *Sub-catchments Grid View* to define the required parameters.

INITIAL LOSSES

The quantity of rainfall required to just cause overland flow is the *initial losses*. The initial part of a rainstorm is assumed to cause no runoff because it is lost in wetting the ground surface and in forming puddles. Initial losses including wetting and depression storage depend on surface type and slope, and can be calculated using a regression equation or specified as an absolute value. Although on paved surfaces this is dominated by depression storage, other losses such as surface wetting, infiltration and evapotranspiration all have an impact on the value of the initial loss. On pervious surfaces interception is a further complicating process. See Depression Storage (Initial Losses) Model.

Initial losses are defined in the runoff parameters file. There are three available types of initial losses:

- Relative

- Absolute
- SCS

The value you give depends on the type.

Type	Value
Absolute	The depression storage depth
Slope	Depression storage is related to ground slope by the expression $D = \text{Value} / \sqrt{s}$ where s is the ground slope
SCS	The value is a proportion of the storage depth that is retained

Depression Storage (Initial Losses) Model

Initial losses are defined as the quantity of rainfall required to cause overland runoff. Hence, the initial loss for catchment surfaces can be taken as the intercept on the rainfall axis of a plot of rainfall verses runoff.

$$D_{\text{perv}} = D_{\text{pav}} = 0,71 S^{-0,48} \quad (1)$$

where:

D	Average depth of initial losses (mm) Note depression storage is defined in metres in runoff parameters files.
S	Average slope of the sub-catchment (%)

This model was included in the original Wallingford Procedure software (WASSP). For pitched roofs where less data was available an initial loss of 0.4mm was advised.

Subsequent experimental studies have shown that initial losses are also dependent on surface type, with variation within surface type dependent on surface slope as previously. Accordingly, the current approach uses an extension of Eq. 1 with units changed to SI:

$$D = \frac{K}{\sqrt{S}} \quad (2)$$

where:

D	Average depth of initial losses (m)
S	Slope (m/m)
k	Coefficient (m)

The value of k reflects such factors as the surface microtopography and layout. The default value used for paved areas is 0.000071 (m).

The initial state of the initial loss storage is defined as an equivalent depth of rainfall associated with a rainfall event. If higher than the depth D given by Eq. 2 this value is equated to D . Furthermore, the initial loss store can also be depleted by evapotranspiration at a constant rate.

It is also possible within the software to define an absolute initial loss value independent of surface slope. This facility is used to override the initial loss values defined for pervious surfaces when using the SCS and New UK Percentage runoff (variable PR) equation models.

The normal values for k are:

Paved and roof surface	0.000071 (m)
Pervious surface	0.000028 (m)

Pitched roof surfaces have the same coefficient as road surfaces but use a slope of 0.05.

Depression storage may be partly or wholly taken up by rainfall immediately before an event; that is, the antecedent rainfall.

RUNOFF VOLUME MODELS

The models that can be used are:

- Fixed PR Model
- Green-Ampt Model
- Horton Infiltration Model
- New UK PR Model
- Wallingford Procedure Model
- Constant Infiltration Model
- US SCS Model

The Wallingford model and UK model are global models, the others are individual models.

In this section we will describe the Wallingford model.

The Wallingford model is applicable to typical urban catchments in the UK. It uses a regression equation to predict the runoff coefficient depending on the density of development, the soil type and the antecedent wetness of each subcatchment.

The model predicts the total runoff from all surfaces in the sub-catchment, both pervious and impervious. This model should therefore not be mixed with another model within one subcatchment.

This is the standard model used to represent continuing losses for UK urban catchments and is applied with the initial losses model described previously. Runoff losses are assumed to be constant throughout a rainfall event and are defined by the relationship:

$$PR = 0.829 PIMP + 25.0 SOIL + 0.078 UCWI - 20.7 \quad (1)$$

where:

PR is the percentage runoff

PIMP is the percentage impermeability. This parameter is the percentage imperviousness of the catchment obtained by dividing the total directly connected impervious area (both roofs and roads) by the total contributing area

SOIL is an index of the water holding capacity of the soil.

The soil index (SOIL) is based on the winter rain acceptance parameter (WRAP) included in the Flood Studies Report and can be obtained from the revised soil map or from the 1:1000000 version covering the whole of the UK. The index describes broadly infiltration potential and was derived by a consideration of soil permeability, topographic slope, and the likelihood of impermeable layers. Five classes of soil are recognised as shown below:

SOIL	Soil Class	WRAP	Runoff	SOIL
1	Very high	Very low	0.15	
2	High	Low	0.30	
3	Moderate	Moderate	0.40	
4	Low	High	0.45	
5	Very low	Very high	0.50	

UCWI is the Urban Catchment Wetness Index

Eq. 1 was derived by statistical analysis. The derived relationship explained 57% of the observed variation of PR. However, when used to predict runoff volume the model provides a far greater explanatory power, as the variation of runoff volume is influenced markedly by the variation of rainfall volume. In fact, Eq. 1 explains the variability of runoff volume after the effect of rainfall volume has been removed. This latter point explains to a certain extent the marked success found in the use of Eq. 1 to predict runoff volume since the release of the Wallingford Procedure.

Inspection of Eq.1 indicates for low values PIMP, SOIL and UCWI low or even negative values of PR could be predicted. Consequently, a minimum value of PR of 20% together with a maximum of 100% is defined within the software. Eq. 1 is therefore used to define PR for separate events within this range. It should be appreciated that unrealistic PR values can be predicted with low values of SOIL (for example 0.15) in combination with both low values of PIMP (for example 30%) and UCWI.

This model predicts the total runoff from all surfaces in the sub-catchment, including both pervious and impervious. The model should therefore not be mixed with another model in one sub-catchment.

Runoff for the entire catchment is distributed between the different surfaces using weighting coefficients. All surfaces can therefore contribute some runoff even at low runoff rates, provided that initial losses have been satisfied.

The weighting is carried out as follows:

$$PR_i = \frac{f_i A_i}{\sum_{n=1,3} f_n A_n} * P \quad (2)$$

where:

f_i is the weighting coefficient for surface i

PR_i is the percentage runoff for surface i

A_i is the area for surface i

Default parameters for the weighting coefficients are:

Weighting coefficient	Surface	Value
f_1	Paved	1.0
f_2	Roofed	1.0
f_3	Pervious	0.1

RUNOFF ROUTING MODELS

Overland runoff on catchment surfaces is represented by the kinematic wave equation. However the direct solution of this equation in combination with the continuity equation is too time consuming for applications with distributed catchment models with a large number of contributing subcatchments.

It has been shown that simpler reservoir-based models, which are less onerous computationally, represent the physical processes as accurately as the more complex physically based approaches. Furthermore, in practice the models are applied to catchments comprising the amalgamated behaviour of a number of overland flow planes, gutters and feeder pipes. Therefore the parameters of a physically based approach (for example the roughness value) as applied would not relate directly to parameters representative of individual surfaces.

The overland flow of the runoff into the drainage system is modelled separately for each type of surface. You can amend the models to suit unusual conditions.

All surfaces for a land use (and therefore subcatchment) must use the same routing model.

Routing Models

Wallingford	The flow is routed using two equal linear reservoirs in series, whose routing coefficient depends on rainfall intensity, contributing area and surface slope (see routing reference at the end of this section)
Large Catchment	The flow is routed using two equal linear reservoirs in series, whose routing coefficient depends on rainfall intensity, contributing area, and surface slope as in the Wallingford model. The software also applies a timestep lag and routing factor multiplier. The timestep lag and routing multiplier are functions of sub-catchment area, ground slope and catchment length
SPRINT	The flow is routed using a single linear reservoir, whose routing coefficient depends on sub-catchment area, ground slope and percentage impermeable
Desbordes	The flow is routed using a single linear reservoir, whose routing coefficient depends on sub-catchment area, ground slope, percentage impermeable, catchment length, storm duration and storm depth
SWMM	The flow is routed using a single non-linear reservoir, whose routing coefficient depends on surface roughness, surface area, ground slope and catchment width

The overland routing transforms the net resulting rainfall for each sub-catchment into an inflow hydrograph at each node, using two notional linear reservoirs in a series to represent the storage which is available on the ground and in minor drains, and the delay induced between the peak rainfall and peak runoff. In this way, a reduced peak runoff is generated with a lag after the peak rainfall. The flow routing coefficient depends on the rainfall intensity, contributing area and slope. For these reasons overland flow in the Wallingford procedure models are modelled using reservoir-routing concepts. Initially a Non-linear reservoir (NLR) model was used; however a Double Linear Reservoir (Wallingford) Model has now superseded this.

The storage routing model is based on the use of a double quasi-linear reservoir model. Two reservoirs are applied in series for each surface type with an equivalent storage-output relationship for each reservoir. This relationship is given by

$$S = kq$$

Where:

$$k = Ci^{-0,39};$$

$i = 0,5(1 + i_{10})$; i_{10} = running ten minutes average of rainfall intensity.

Combination of the two storage relationships with the continuity equation leads to a second-order ordinary differential equation of the form:

$$k^2 \frac{d^2 q}{dt^2} + 2k \frac{dq}{dt} + q = i_n$$

Sarginson and Nussey in fact first proposed this form of model for use in the UK.

For each of the small catchments described in Table 1 the value of C has been derived by optimisation. An overall relationship relating this value to catchment characteristics was then developed and included in the software; the final relationship is

$$C = 0,117S^{-0,13} A^{0,24}$$

Within the software limits are placed on the parameters as described below:

If ($S < 0.002$), $S = 0.002$

If ($A < 1000$), $A = 1000$

If ($A > 10000$), $A = 10000$

Note A is the area of the surface for example the area of the paved surface.

2.1.2.5 Trade and Wastewater Events

Trade and Wastewater events are very similar in practice and the editing process is virtually the same. Both event types use the Wastewater Event Editor.

The main difference between the two is that the Wastewater event describes per capita flow, and the Trade event describes flow from a single source. Wastewater events have a Per Capita Flow field that is multiplied by a value for population to produce actual flow. The *Per Capita Flow* field is replaced by a Flow Scaling Factor field in the Trade event. This field is only retained for compatibility with previous versions and should always be set to one.

2.1.2.6 Rainfall Events

This section describes the way in which rainfall event data is stored, edited and used in InfoWorks.

There are two main types of rainfall data that you can use in InfoWorks:

observed (or recorded) rainfall data represents actual rainfall events. These events can cover a single storm or multiple storms over a long period of time.

Observed You would normally use observed rainfall data to calibrate and verify the model against flow survey data and flooding records for historical storm events.

synthetic rainfall (or design rainfall) represents a statistical event of known length and return period, derived from analysis of many years rainfall records.

Synthetic Synthetic rainfall will normally be used during analysis and design work with a calibrated model. It allows the rainfall characteristics of a region to be represented with a limited number of storms.

Observed Rainfall

Use recorded rainfall data to calibrate the model and verify its correct operation against data collected during flow surveys or against flood records for large historical storms.

Rainfall data will often be available for several rain gauges. This is a great benefit because rainfall intensities can vary significantly across a catchment.

Long Rainfall Records

In some cases rainfall data covering many years of rainfall events may be available. You could use these long term records in two ways within InfoWorks:

- As a continuous rainfall record including dry periods. If you include data as a single sub-event in this way, the timestep for dry periods will be the same as for rainfall events.

You could use a continuous rainfall record when modelling the flows to a treatment works

- As a series of sub-events, each one representing a storm. This method has several advantages
 - the timestep for each sub-event can be different so you can combine recorded data from different sources
 - subcatchment parameters - UCWI, Antecedent Rainfall Depth, Evaporation, and SCS Wetness Index - can be set separately for each sub-event
 - during periods without rainfall the simulation engine can switch to dry weather flow mode and use a longer timestep, thus reducing simulation times

Synthetic Rainfall

Synthetic storms represent the statistical characteristics of rainfall. They are derived from analysis of many years of actual rainfall records. They are easier to use than observed rainfall and can approximate a region's rainfall in just a few storms.

You will normally use synthetic rainfall during normal analysis and design work. However, long time-series rainfall may have advantages in some cases as it is likely to contain a wider range of conditions. In order to cover the same conditions using synthetic storms, you will have to generate a large number of storms covering this wide range.

You can edit synthetic data in two ways:

- import an existing synthetic hydrograph, or build a synthetic hydrograph from scratch. You will edit the synthetic event in the same way as an observed event.
- use one of the synthetic rainfall generators supplied with InfoWorks. The generators currently available are the UK Rainfall Generator and the Hong Kong Rainfall Generator.

Time-Series Rainfall

In the UK, typical synthetic time-series are available for several regions of the country. These are generated artificially by analysing past events. They reproduce the random variations in the timing and magnitude of actual rainfall events in the region.

Time-series rainfall generally includes a wide range of conditions and is more likely to contain the conditions that are critical for each catchment. In some cases it might be more appropriate for modelling work than synthetic rainfall events.

You might use this type of data to study the operation of overflows and pumping stations.

Rainfall Initial Conditions

Both Observed and Synthetic rainfall events can contain initial conditions for sediment on the subcatchment surface (water quality only) and runoff.

You can import the initial conditions at the same time as a rainfall file or import into an existing rainfall event

Spatially Varying Rainfall

InfoWorks supports spatially varying rainfall by allowing rain-gauge regions to be stored with each profile in a rainfall event.

You cannot directly use generated synthetic rainfall generated within InfoWorks for spatially varying rainfall because these events only contain one profile. You can, however, export the synthetic event and then import it again so it can be edited like an observed event. Additional profiles can then be added if required.

Editing Rainfall Data

Observed rainfall data is edited on the Rainfall Event Editor. This is a special type of the generic *Event Editor*.

Generated synthetic rainfall data is edited in two different places. The data needed to generate the rainfall profile is edited using the appropriate rainfall generator. Rainfall generators currently available are:

- UK Rainfall Generator
- Hong Kong Rainfall Generator

Generated synthetic rainfall profile data can be viewed on the Rainfall Event Editor.

Synthetic rainfall events can be created and imported in the same way as observed rainfall. If you have done this then edit the synthetic rainfall using the Rainfall Event Editor.

Initial conditions for both observed and synthetic rainfall can be viewed and edited on the Rainfall Event Editor.

Initial State of the Catchment

The rainfall data defines the rainfall intensities during a storm. The wetness of the catchment at the start of each sub-event must also be defined to calculate the rainfall losses and rates of runoff. The antecedent conditions will be either standard conditions for use with synthetic events or recorded conditions for use with recorded events.

You can also define evapotranspiration for the model.

2.1.2.7 Hydraulic Model

- Conduit Model

A conduit is represented as a link in the network, of defined length, between two nodes. The boundary condition between the link and a node is either of the outfall or headloss type. The gradient of a conduit is defined by invert levels at each end of the link; this does not preclude discontinuities in level at nodes or negative gradients.

A variety of pre-defined cross-sectional shapes may be selected for both closed pipes and open channels. Circular pipes are defined by one dimension - the diameter - and all others by the height and width; in the case of open channels the height will be to the top of the channel lining. Non-standard cross-sectional shapes may be modelled by defining a non-dimensional height/width relationship.

Two different values of hydraulic roughness may be assigned; one for the bottom third of the conduit and one for the remainder. A permanent depth of sediment may be defined in the invert of the conduit; no erosion or deposition is considered.

The governing model equations are the Saint-Venant equations (see Yen, 1973). These are a pair of conservation equations of mass and momentum:

$$\frac{\delta A}{\delta t} + \frac{\delta Q}{\delta x} = 0$$

$$\frac{\delta Q}{\delta t} + \frac{\delta}{\delta x} \left(\frac{Q^2}{A} \right) + gA \left(\cos \vartheta \frac{\delta y}{\delta x} - S_0 + \frac{Q|Q|}{K^2} \right) = 0$$

The model equations governing pressurised pipe flow differ in that the free surface width is replaced conceptually by the relatively small term:

$$B = g^* A_f / C_p^2$$

The solution of the Saint-Venant equations may be retained in pressurised flow by introducing a suitably narrow slot, the Preissmann slot, into the pipe soffit (see Cunge and Wegner, 1964). A smooth transition between free surface and surcharged conditions is thus enabled.

The effect of placing the Preissmann slot directly onto the pipe soffit may be an abrupt change in surface width derivative and wave celerity in the transition to pressurised conditions. A transition region is included within the model defined by a monotonic cubic between the true pipe geometry and the width of the Preissmann slot.

The slot width itself is defined such that the wave celerity in the slot is ten times that at half the conduit height. This allows accurate modelling of pressurised flow (see Gomez et al., 1992) and results in a slot width that is 2% of the conduit width.

In the case of an open channel the geometry is extrapolated if the level in the conduit exceeds the height of the channel lining.

Note that in a closed pipe the maximum conveyance will actually occur below the pipe soffit and will be greater than the 'pipe full' value. The conveyance approximates this by imposing monotonicity to avoid the turning point and the possibility of multiple numerical solutions.

- Pressurised Pipe Model

You can use the pressurised pipe model instead of the full (Saint-Venant) solution on selected pipes: for example, if you wish to model rising mains or inverted siphons.

The model equations governing pressurised pipes are:

$$\frac{\delta Q}{\delta x} = 0$$

$$\frac{\delta Q}{\delta t} + gA \left(\cos \vartheta \frac{\delta y}{\delta x} - S_0 + \frac{Q|Q|}{K^2} \right) = 0$$

The pressurised pipe model more accurately predicts velocities and storage than the full (St. Venant) model because it does not assign base flow or a Preissmann slot to a pipe.

2.1.2.8 Headloss Curves

A headloss condition must be specified at each end of a conduit. The headloss condition represents the extra energy loss due to turbulence at a manhole or junction. You can specify any of the following built in headloss conditions:

- No headloss
- Normal manhole headloss
- High manhole headloss
- Fixed headloss

or you can define your own headloss curves using the Grid Headloss Curve View and Headloss Curve Property Sheet.

The default for an individual conduit is the global value specified for the drainage system; this has a default value of normal manhole headloss.

The *normal manhole headloss* is appropriate for well-constructed manholes on pipe systems, or for small open channel systems. There is negligible headloss for links flowing less than half full and small headloss up to just less than full. The actual values for well-constructed manholes are 0.001 up to half pipe full, 0.05 at 3/4 pipe full and 0.1 at pipe full. The headloss varies with the depth of surcharge, but is equivalent to 0.15 times the velocity head ($0.15 \frac{v^2}{2g}$) for large depths of surcharge.

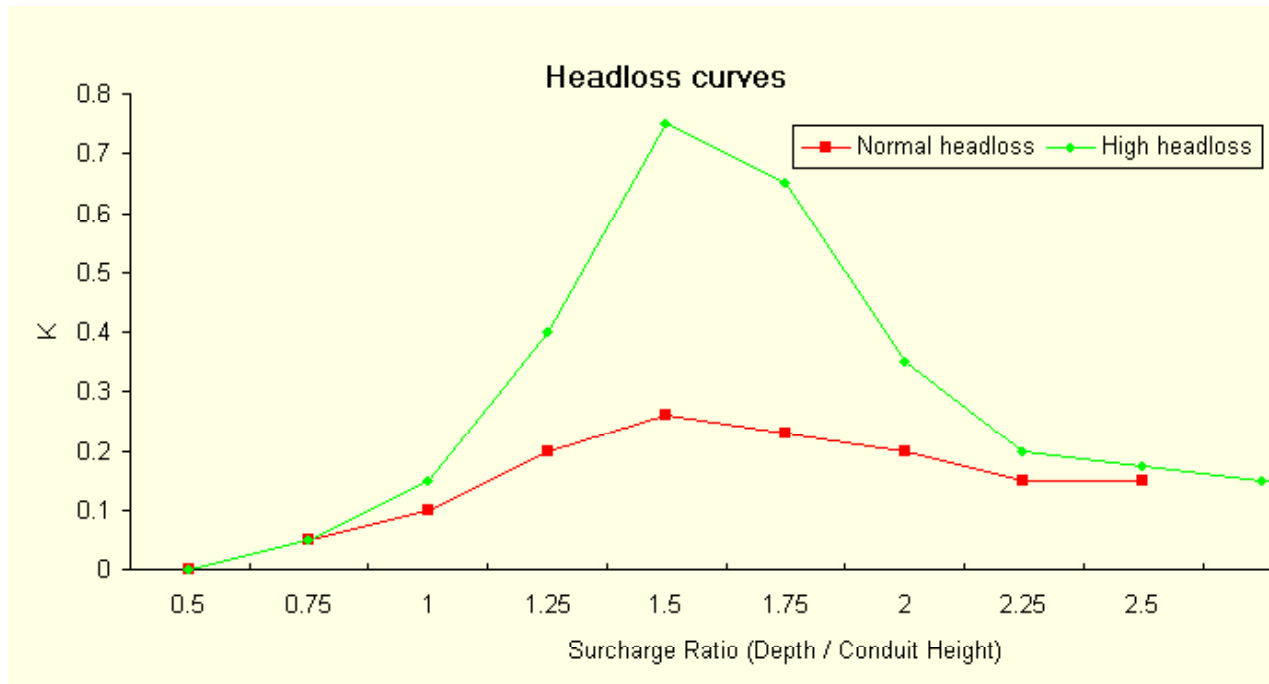
The *high manhole headloss* is appropriate for badly constructed manholes that are benched only to half pipe height. There is more headloss for depths less than pipe full than for normal manholes, but at large depths headloss is $0.15 \frac{v^2}{2g}$ (as for a normal manhole).

The *fixed headloss* is appropriate for situations when you have a known headloss, such as entry into a reservoir or an open channel with bends. The headloss factor represents the k value and does not vary with surcharge ratio or flow direction.

Linear interpolation is used between given values. Linear extrapolation is used outside the given range.

Headloss curves

The diagram and table below show the headloss curves that InfoWorks uses.



Surcharge ratio Normal headloss factor (headloss type 1) High headloss factor (headloss type 2)

0.50	0.001	0.001
0.75	0.05	0.05
1.00	0.10	0.15
1.25	0.20	0.40
1.50	0.26	0.75
1.75	0.23	0.65
2.00	0.20	0.35
2.25	0.15	0.20
2.50	0.15	0.175
2.75	0.15	0.15
3.00	0.15	0.15

The following relationship defines the surcharge ratio:

$$\text{Surcharge ratio} = (\text{level in mandole} - \text{Invert level at the end of the link}) / (\text{conduit height})$$

The headloss is calculated using the following equation:

$$\Delta h = kv^2 / 2g$$

Note that the losses for flow from a conduit into a manhole (exit) are calculated as 0.1 of the losses for flow from a manhole into a conduit (entry).

Angle of approach

You can apply a factor to both the normal and high headloss conditions. This may represent the extra headloss due to the angle of approach of a conduit to a manhole. It may also represent headloss at additional intermediate manholes that the modeller has not included in the model. The default value is 1.0.

The suggested factors for the angle of approach are:

Angle	Value
30	3.3
60	6.0
90	6.6
>90	8.0

2.1.2.9 Infiltration Module

Flow in sewer systems frequently exceeds the sum of stormwater runoff and domestic and trade inflows. This residual flow is usually attributed to infiltration, which enters the sewer system through cracks. Unlike runoff, which responds to a rainfall event in minutes, infiltration inflows have a much slower response. There are two main types of infiltration:-

Rainfall-induced infiltration results from soil water infiltrating directly into the sewer network. This has an effect on flow within hours or days of the storm.

A proportion of rainfall percolates deeper into the groundwater reservoir and in the weeks or months following the storm the groundwater level may be sufficiently high to cause groundwater infiltration.

For single-event simulations, infiltration can be modeled as a constant inflow. For time-series simulations, however, some account of antecedent hydrological conditions is required; this data is included in the InfoWorks CS infiltration model.

The infiltration model uses bulk mass balance equations and simplified flow equations to approximate the physical processes. Because the model is simplified it requires some degree of calibration.

Rainfall runoff has three model components: initial loss (depression storage), runoff volume and runoff routing. Incident rainfall is initially stored in surface depressions, which are subject to evaporative loss (defined in the rainfall event). When rainfall exceeds depression storage a proportion of the excess rainfall goes to runoff according to the particular volume model used. The remaining rainfall is directed into the soil storage reservoir. When the soil reaches a given saturation threshold (the percolation threshold), water starts to percolate downwards. A proportion of this percolation flow (the percolation percentage infiltrating) infiltrates directly into the sewer network while the remainder penetrates deeper to feed the groundwater storage reservoir. Note that the volume in the soil storage reservoir is also subject to evapotranspiration, though at a reduced rate.

When the groundwater storage reservoir reaches a particular threshold water loss due to baseflow occurs. When the groundwater level reaches a further infiltration threshold, groundwater infiltration occurs.

The infiltration model can be calibrated so that the ground water storage level relates to the actual groundwater table level. In this case, the infiltration threshold type and baseflow threshold types are set to levels that are relative to the chamber floor of the node that the particular subcatchment drains to. This is a reasonable estimate of the realistic level at which infiltration may occur.

In networks where infiltration is dominated by tidal influences, you can create a time-varying profile for the groundwater storage level. This profile will override the level calculated by the infiltration model and groundwater infiltration is then based on this level.

2.1.3 SWMM –(Appendix 3 - The Mondello case study in Palermo, Italy)

SWMM is a dynamic rainfall-runoff simulation model, primarily but not exclusively designed for urban drainage systems analysis and for single-event or long-term (continuous) simulation.

SWMM can be considered as a complete suite of tools covering all the aspects of urban drainage simulation: runoff generation and propagation, water quality analysis on catchment surface, in the drainage system and in the receiving waters.

An overview of the model structure is shown in fig. 1. In simplest terms the program is constructed in the form of “blocks” as follows:

- Runoff Block;
- Transport Block;
- Extended Transport (Extran) Block;
- Storage/Treatment Block;
- Receive Block.

Quality constituents for simulation may be arbitrarily chosen for any of the block, although the different blocks have different constraints on the number and type of constituents that may be modelled. The Extran Block is the only block that does not simulate water quality.

Flow routing can be performed in the Runoff, Transport and Extran Blocks, in increasing order of sophistication.

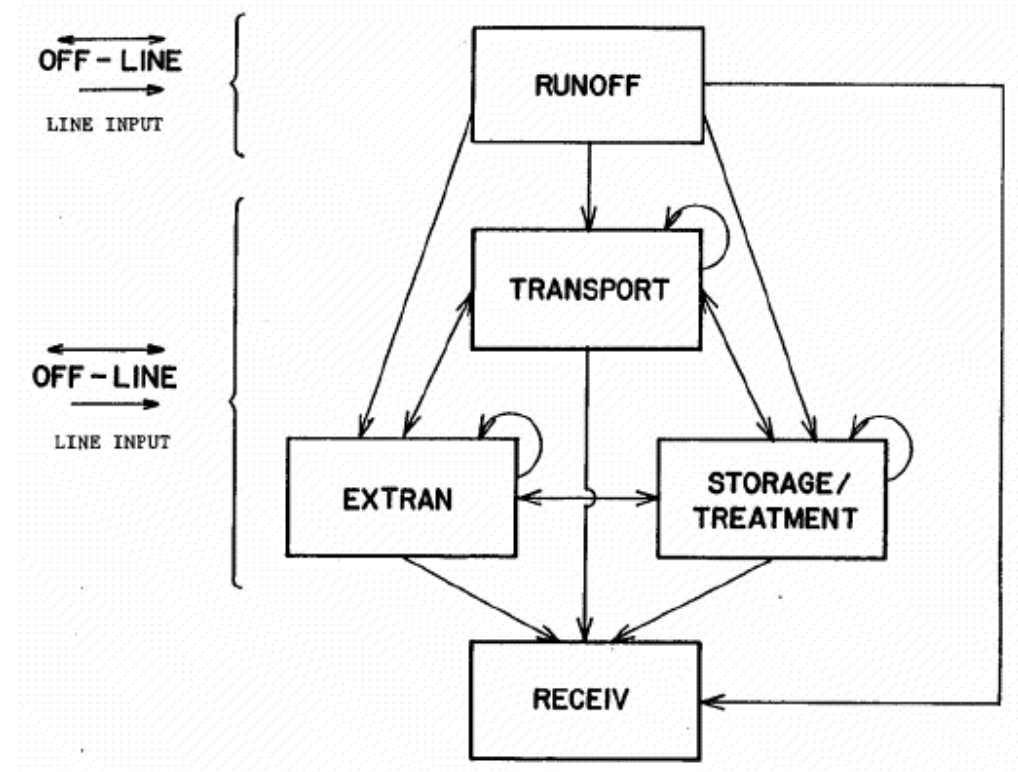
SWMM continues to be widely used throughout the world for analysis of quantity and quality problems related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well.

The model may be used for both planning and design. The planning mode is used for an overall assessment of the urban runoff problem and proposed abatement options. This mode is typified by continuous simulation for several years using long-term precipitation data.

At design-level, event simulation also may be run using a detailed catchment schematization and shorter time steps for precipitation input.

Both single-event and continuous simulation may be performed on catchments having storm sewers, or combined sewers and natural drainage, for prediction of flows, stages and pollutant concentrations.

The current version is SWMM5 and it is under beta testing (BETA E – 03 february 2004). In CARE-S research activities the second newest version (4.4H – 2002) has been used because it



has a wider

Figure 1: Overview of SWMM model structure, indicating linkages among the computational blocks.

Water quantity modelling is basically demanded to Runoff Block (rainfall-runoff transformation) and to Extran Block (flow propagation).

2.1.3.1 Runoff block

The Runoff Block has been developed to simulate both the quantity and quality runoff phenomena of a drainage basin and the routing of flows and contaminants to the major sewer lines. It represents the basin by an aggregate of idealized subcatchments and gutter or pipes. The

program accepts an arbitrary rainfall or snowfall hyetograph and makes a step by step accounting of snowmelt, infiltration losses in pervious areas, surface detention, overland flow, and channel flow leading to the calculation of a number of inlet hydrographs that can be passed as input to other SWMM blocks. This Block may be run for periods ranging from minutes to year. Surface flow routing is accomplished using four types of elements:

- Subcatchment elements (overland flow);
- Channel elements (trapezoidal or parabolic channel flow);
- Pipe elements (circular channel flow);
- Control structures (weir and orifices).

Each subcatchment is schematized as in Figure 2.2.2, in which three or four subareas (depending on whether snowmelt is simulated) are used to represent different surface properties. The slope of the idealized subcatchment is in the direction perpendicular to the width. The width of the pervious subarea, A2, is the entire subcatchment width, whereas the widths of the impervious subareas, A1, A3, A4, are in proportion to the ratio of their area to the total impervious area, as implied in Figure 1.1.2. Of course, real subcatchments seldom exhibit the uniform rectangular geometries shown in Figure 1.1.2. In terms of the flow routing, all geometrical properties are merely parameters and no inherent “shape” can be assumed in the non-linear reservoir technique. However, in terms of parameter selection, the conceptual geometry is useful because it aids in explaining the flow routing.

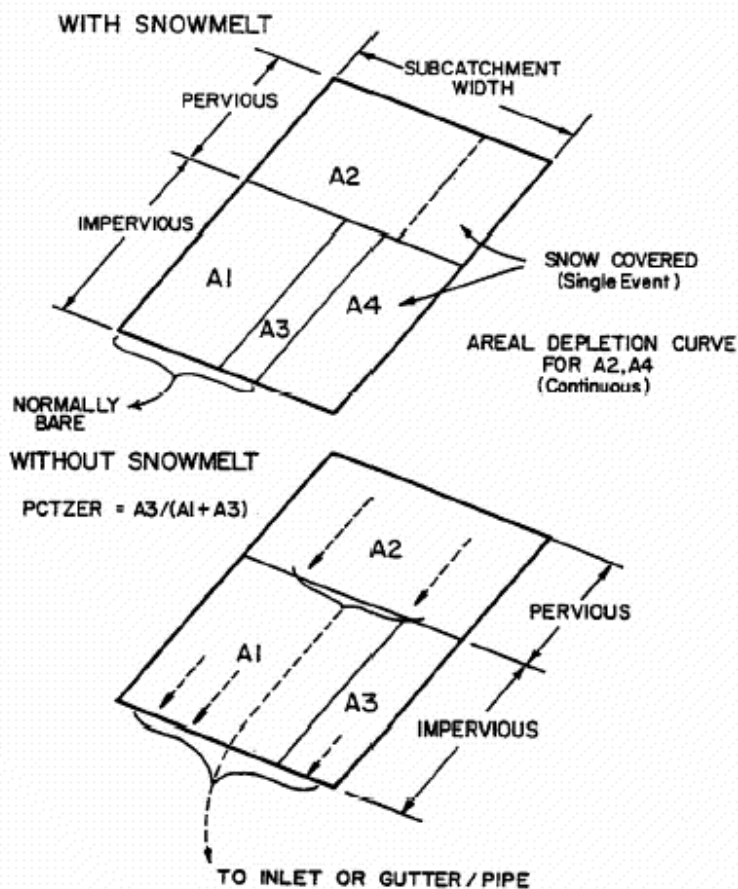


Figure 1.1.2: Subcatchment schematization with or without Snowmelt

Subcatchment elements receive rainfall and snowmelt, account for losses due to evaporation and infiltration (via Horton's or Green-Ampt equations), and permit surface depression storage to account for losses such as ponding or retention on grass or pavement (pervious or impervious areas). In fact, depression (retention) storage is a volume that must be filled to the occurrence of runoff on both pervious and impervious areas. "Losses" from infiltration may optionally be routed through a subsurface pathway, first into an unsaturated zone storage, then to a saturated zone storage from which baseflow into an inlet or channel/pipe may be generated. Surface flow from subcatchment is always into channel/pipe elements or inlets. Flow routing for both subcatchments and channel/pipes is accomplished by approximating them as non-linear reservoirs. This is simply a coupling of a spatially lumped continuity equation with Manning's equation. Should the capacity of a channel/pipe be exceeded, "surcharge" is indicated, and excess water is stored at the upstream end until the channel/pipe can accept it.

SWMM snowmelt routines are based on earlier work done on the Canadian SWMM study by Proctor and Redfern and James F. MacLaren (1976a, 1976b, 1977). Most techniques are drawn from Anderson's (1973) work for the National Weather Service (NWS). For continuous simulation, daily max-min temperatures are converted to hourly values to sinusoidal interpolation, as explained earlier. Urban snow removal practices may be simulated through "redistribution fractions" input for each subcatchment, through alteration of the melt coefficients and base temperatures for the regions of each subcatchment, and through the areal depletion curves used for continuous simulation. Anderson's temperature-index and heat balance melt equations are used for melt computations during dry and rainy periods, respectively. For continuous simulation, the "cold content" of the pack is maintained in order to "ripen" the snow before melting. Routing of melt water through the snow pack is performed as a simple reservoir routing procedure, as in the Canadian study.

The presence of a snow pack is assumed to have no effect on overland flow processes beneath it. Melt is routed in the same manner as rainfall.

Simulation time step can not be changed during simulation but it is possible to define different values for the "wet" period (during rainfall events), the "transition" period (after the end of rainfall events, when runoff is still present on the catchment surface), the "dry" period (when no rainfall and no runoff is present on the catchment). The time step only influences the approximation of simulation outputs but not the stability of the model.

The depth of depression storage is an input parameter (WSTORE) for the impervious areas of each catchment. The impervious area without depression storage is specified for all subcatchments by parameter PCTZER (as a percent):

$$A_3 = \frac{PCTZER}{100}(A_1 + A_3)$$

Overland flow is generated from each of the three subareas by approximating them as non-linear reservoirs, as sketched in figure 1.1.3.

The non-linear reservoir is established by coupling the continuity equation with Manning's equation. Continuity may be written for a subareas as:

$$\frac{dV}{dt} = A \frac{dd}{dt} = A \cdot i^* - Q$$

where:

V = A · d = volume of water on the subarea

d = water depth

t = time

A = surface area of subcatchment, ft²

i* = rainfall excess = rainfall/snowmelt intensity minus evaporation/infiltration

Q = outfall rate

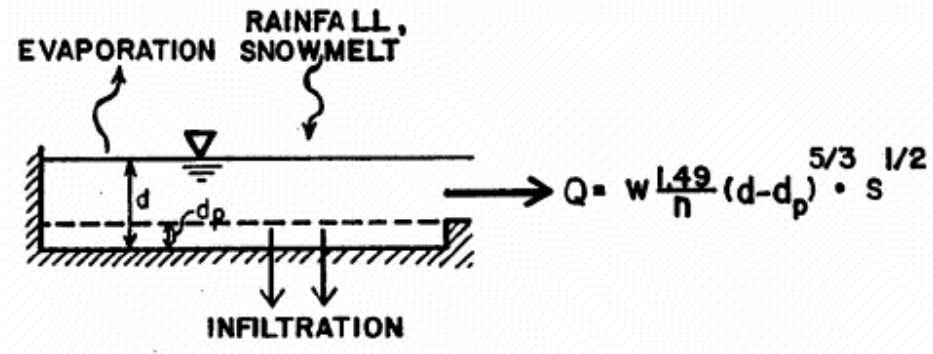


Figure 1.1.3: Non – linear reservoir model scheme

The outflow is generated using Manning’s equation:

$$Q = W \frac{1.49}{n} (d - d_p)^{5/3} S^{1/2}$$

where:

- W = subcatchment width
- n = Manning’s roughness coefficient,
- dp = depth of depression storage, and
- S = subcatchment slope.

These two equations may be combined into one non-linear differential equation. This produces the non-linear reservoir equation that can be solved at each time step by means of a simple finite difference scheme and can be approximated by:

$$\frac{d_2 - d_1}{\Delta t} = i^* + WCON \cdot \left[d_1 + \frac{1}{2}(d_2 - d_1) - d_p \right]^{5/3}$$

where

Δt = time step, sec

$$WCON = \frac{1.49 \cdot W \cdot S^{1/2}}{A \cdot n}$$

Infiltration from pervious areas may be computed by either the Horton (1933, 1940) or Green-Ampt (1911) equation described below.

Infiltration capacity as a function of time is given by Horton as:

$$f_p = f_c + (f_0 - f_c)e^{-kt}$$

where:

f_p = infiltration capacity into soil

f_c = minimum or ultimate value of f_p (WLMIN)

f_0 = maximum or initial value of f_p (WLMAX)

t = time from beginning of storm and

k = decay coefficient (DECAY)

This equation describes the familiar exponential decay of infiltration capacity evident during heavy storms.

The second infiltration option is the Green-Ampt equation that, has the advantage of physically based parameters that, in principle, can be predicted a priori. The Mein-Larson (1973) formulation of the Green-Ampt equation is a two-stage model. The first step predicts the volume of water, F_s which will infiltrate before the surface becomes saturated. From this point onward, infiltration capacity, f_p , is predicted directly by Green-Ampt equation. Thus,

$$\text{For } F < F_s : f = I \text{ and } F_s = \frac{S_u IMD}{i/K_s - 1} \text{ for } i > K_s$$

$$\text{For } F \geq F_s : f = f_p \text{ and } f_p = K_s \left(1 + \frac{S_u IMD}{F} \right)$$

where:

f = infiltration rate

f_p = infiltration capacity

i = rainfall intensity

F = cumulative infiltration volume, this event

F_s = cumulative infiltration volume required to cause surface saturation

S_u = average capillary suction at the wetting front (SUCTION)

IMD = initial moisture deficit for this event (SMDMAX) and

K_s = saturated hydraulic conductivity of soil, (HYDCON).

Infiltration is thus related to the volume of water infiltrated as well as to the moisture conditions in the surface soil zone.

Depression (retention) storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas; a good discussion is presented by Viessman et al. (1977). It represents a loss or “initial abstraction” caused by such phenomena as surface ponding, surface wetting, interception and evaporation. In some models, “depression storage” also includes infiltration in pervious areas. In the Runoff Block, water stored as depression storage on pervious areas is subject to infiltration (and evaporation), so that it is continuously and rapidly replenished. Water stored in depression storage on impervious areas is depleted only by evaporation. Hence, replenishment typically takes much longer.

When snowmelt is simulated, a fourth subarea is added to each subcatchment as illustrated in Figure 1.1.4. The main purpose of the fourth subarea is to permit part of the impervious area (subarea A4) to be continuously snow covered (e.g., due to windrowing or dumping) and part (subareas A1 plus A3) to be “normally bare” (e.g., streets and sidewalks that are swept).

During single event simulation, subarea A4 retains 100 percent snow cover until it has all melted. During continuous simulation, an areal depletion curve is used.

Similarly, for single event simulation, a fraction of the pervious area remains 100 percent snow covered. During continuous simulation, the whole pervious area is subject to areal depletion curve.

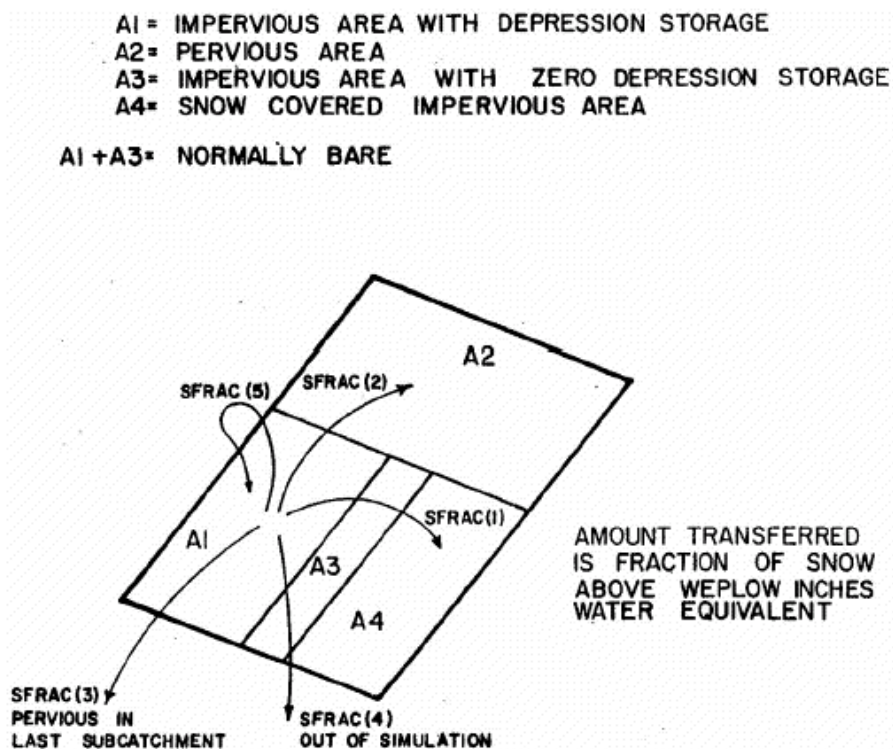


Figure 1.1.4: Snow redistribution during ploughing

Initial snow depths (inches/mm water equivalent) may be entered using catchment characterization parameters. This is likely to be the only source of snow for a single event simulation although snowfall values may be entered as negative precipitation in rain data group.

During continuous simulation, the effect of initial conditions will die out, given a simulation of a few months. No liquid runoff will leave the snow pack until its free water holding capacity (due to its porosity) has been exceeded. The available volume is a constant fraction, of the snow depth. During periods of no rainfall, snowmelt is computed by a degree-day or temperature index equation.

Melt coefficients and base melt temperatures may be determined both theoretically and experimentally. Considering the former, it is possible to first write a snowmelt equation from a heat budget formulation that includes all relevant terms: change in snow pack heat storage, net short wave radiation entering pack, conduction of heat to the pack from underlying ground, net (incoming minus outgoing) long wave radiation entering pack, convective transport of sensible

heat from air to pack, release of latent heat of vaporization by condensation of atmospheric water vapour, and advection of heat to snow pack by rain. (It is assumed here that the pack is “ripe”, i.e., just at the melting point, so that rain will not freeze and release its latent heat of fusion.) The equation may then be linearised about a reference air temperature. Alternatively, observed melt, in inches per time interval, may be plotted against temperature for that time interval, and a linear relationship developed.

The program allows (during continuous simulation) snow that falls on the normally bare impervious areas to be redistributed according to the other parts of the catchment. This is intended to simulate ploughing and other snow removal practices in urban areas. Snow depths above the ploughing water equivalent are thus redistributed according to Figure 1.1.4.

The value of ploughing water equivalent depends upon the level of service given the particular impervious area. The five fractions SFRAC, should sum to 1.0 and are defined on the basis of the ultimate fate of the removed snow. For instance, if snow is ploughed from a street onto an adjacent impervious or pervious area, fractions SFRAC(1) or SFRAC(2) would be appropriate. It may also be transferred to the last subcatchment (e.g., a dumping ground) or removed from the simulation (i.e., removed from the total catchment) altogether. Finally, it may be converted to immediate melt. Should variations in snow removal practices need to be simulated, different subcatchments can be established for different purposes and the fractions varied accordingly.

2.1.3.2 Flow Routing

A conceptual overview of Extran Block is shown in Fig. 1.1.5. As shown here, the specific function of Extran is to route inlet hydrographs through the network of pipes, junctions, and flow diversion structures of the main sewer system to the treatment plant interceptors and receiving water outfalls.

As shown in Fig. 1.1.5, Extran simulates pipes, manholes (pipe junctions), weirs, orifices, pumps (on-line or off-line pump station), storage basins (on-line enlarged pipes or tunnels, on-line or off-line arbitrary stage-area relationship) and outfall structures (transverse with or without tide gate, side-flw weir with tide gate, outfall with tide gate, free outfall without tide gate).

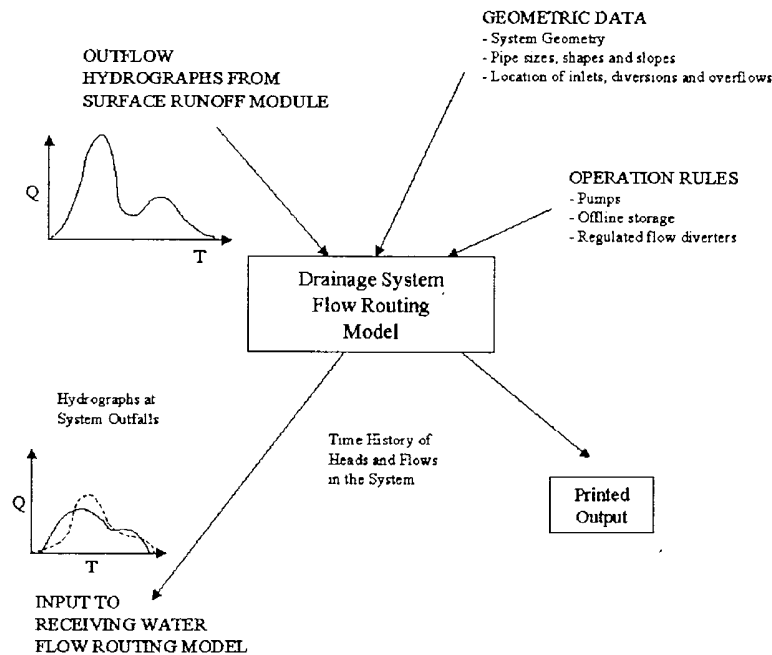


Figure 1.1.5: Extran module functional scheme

Output from Extran takes the form of:

- discharge hydrographs and velocities in selected conduits in printed and plotted form;
- flow depths and water surface elevations at selected junctions in printed and plotted form. Hydrographs may be supplied to a subsequent module on the output interface file.

Extran uses a link-node description of the sewer system which facilitates the discrete representation of the physical prototype and the mathematical solution of the gradually-varied unsteady flow (St. Venant) equations which form the mathematical basis of the model. As shown in Fig. 1.1.6, the conduit system is idealized as a series of links or pipes which are connected at nodes or junctions. Links transmit flow from node to node. Properties associated with the links are roughness, length cross-sectional area, hydraulic radius, and surface width. The last three properties are functions of the instantaneous depth of flow. The primary dependent variable in the links is the discharge, Q . The solution is for the average flow in each link, assumed to be constant over a time step. Velocity and cross sectional area of flow, or depth, are variable in the link in the numerical solution.

Nodes are the storage elements of the system and correspond to manholes or pipe junctions in the physical system. The variables associated with a node are volume, head, and surface area. The primary dependent variable is the head, H , which is assumed to be changing in time but constant throughout any one node. Inflows, such as inlet hydrographs, and outflows, such as weir diversions, take place at the nodes of the idealized sewer system. The volume of the node at any time is equivalent to the water volume in the half-pipe length connected to any one node. The change in nodal volume during a given time step, Δt , forms the basis of head and discharge calculations as discussed below.

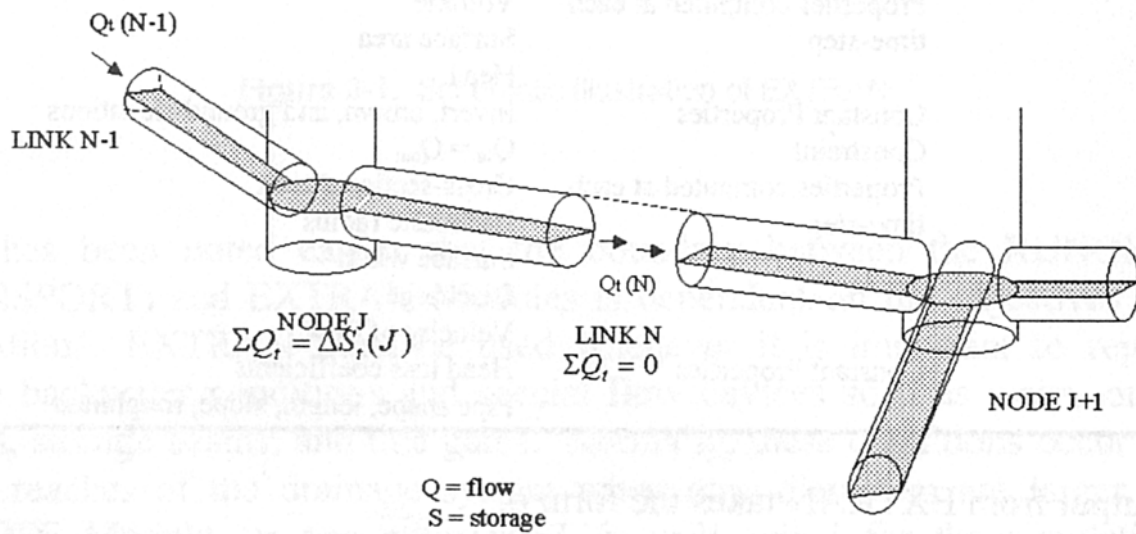


Figure 1.1.6: Node – Link computational scheme

The basic differential equations for the sewer flow problem come from the gradually varied, unsteady flow equations for open channels, otherwise known as the St. Venant or shallow water equations. The unsteady flow continuity equation with surface area flow as dependent variables (Yen, 1986; Lai, 1986) is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

where:

- A = cross sectional area,
- Q = conduit flow,
- X = distance along the pipe/channel, and
- T = time.

The momentum equation is (Lai, 1986):

$$\frac{\partial Q}{\partial t} + \frac{\partial(Q^2/A)}{\partial x} + gA \frac{\partial H}{\partial x} + gAS_f = 0$$

where:

- g = gravitational constant,
- H = z + h = hydraulic head,
- z = invert elevation,
- h = water depth, and
- S_f = friction (energy) slope.

(The bottom slope is incorporated into gradient of H).

Extran uses the momentum equation in the links and a special lumped continuity equation for the nodes. Thus, momentum is conserved in the links and continuity in the nodes.

For use in Extran, the momentum equation is combined with the continuity equations to yield an equation to be solved along each link at each time step:

$$\frac{\partial Q}{\partial t} + gAS_f - 2V \frac{\partial A}{\partial t} - V^2 \frac{\partial A}{\partial x} + gA \frac{\partial H}{\partial x} = 0$$

where:

Q = discharge along the conduit,

V = velocity in the conduit,

A = cross-sectional area of the flow,

H = hydraulic head (invert elevation plus water depth), and

Sf = friction slope.

The friction slope is defined by Manning's equation, i.e.:

$$S_f = \frac{k}{gAR^{4/3}} Q|V|$$

where:

k = $g(n/1.49)^2$ for U.S. customary units and gn^2 for metric units

n = Manning roughness coefficient,

g = gravitational acceleration,

R = hydraulic radius.

Use of the absolute value on the sign on the flow term makes Sf a directional quantity and ensures that the friction force always opposes the flow.

Solving for $Q_{t+\Delta t}$ gives the final finite difference form of the dynamic flow equation:

$$Q_{t+\Delta t} = \frac{1}{1 + \frac{k\Delta t}{R^{4/3}|V|}} \left[Q_t + 2\bar{V} \left(\frac{\Delta A}{\Delta t} \right) \Delta t + \bar{V}^2 [(A_2 - A_1)/L] \Delta t - g\bar{A} [(H_2 - H_1)/L] \Delta t \right]$$

Another equation is required relating Q and H. This can be obtained by writing the continuity equation at a node:

$$\frac{\partial H}{\partial t} = \sum \frac{Q_i}{A_{s_i}}$$

Or, in finite difference form

$$H_{t+\Delta t} = H_t + \sum Q_i \Delta t / A_{s_i}$$

where:

As = surface area of node.

Flow and continuity equation can be solved sequentially to determine discharge in each link and head at each node over a time-step Δt . The numerical integration of these two equations is accomplished by the improved polygon or modified Euler method. The results have proven to be

relatively accurate and, when certain constraints are followed, stable. Fig. 1.1.7 shows how the process would work if only the discharge equation were involved.

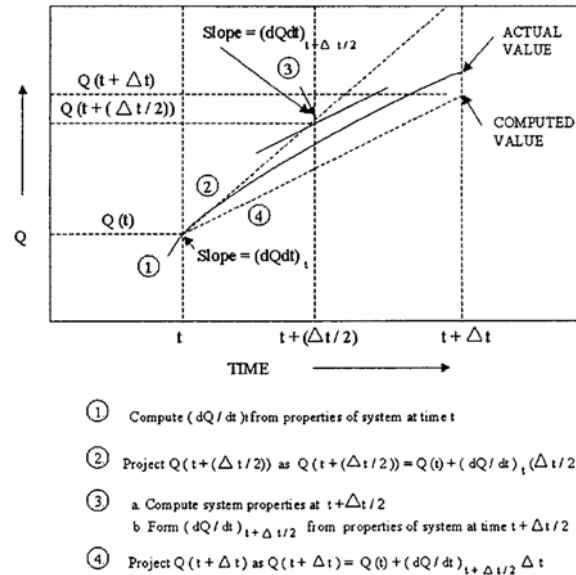


Figure 1.1.7: Computational approximation scheme.

The link-node computations can be extended to include devices which divert sanitary sewage out of a combined sewer system or relieve the storm load on sanitary interceptors. In Extran, all diversions are assumed to take place at a node and are handled as inter-nodal transfer. The special flow regulation devices treated by Extran include: weirs (both side-flow and transverse), orifices, pumps, and outfalls. Each of these is discussed below.

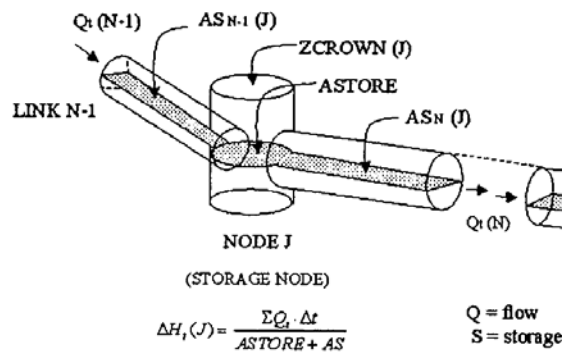


Figure 1.1.8: Storage node computational approach.

In-line or off-line devices act as flow control devices by providing for storage of excessive upstream flows thereby attenuating and lagging the wet weather flow hydrograph from the upstream area. The conceptual representations of a storage junction and a regular junction are illustrated in Fig. 1.1.8. An arbitrary stage-area-volume relationship may be also be input (data group E2), e.g. to represent detention ponds. Routing is performed by ordinary level-surface reservoir methods. This type of storage facility is not allowed to surcharge.

As it is shown in Fig. 1.1.9, Extran also simulates orifices by converting the orifices to an equivalent pipe. The conversion is made as follows. The standard orifice equation is:

$$Q_0 = C_0 A \sqrt{2gh}$$

where:

C_0 = discharge coefficient,

A = cross-sectional area of the orifice,

g = gravitational acceleration, and

h = the hydraulic head on the orifice.

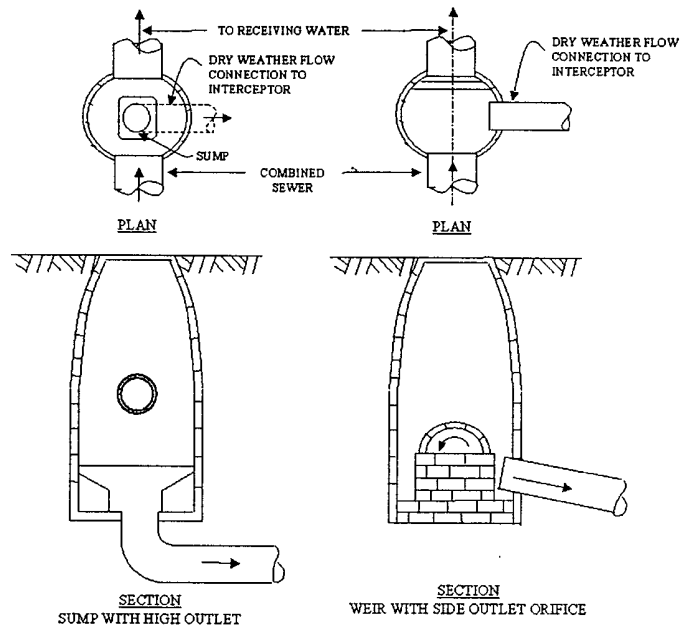


Figure 1.1.9: SWMM orifice scheme.

To convert the orifice to a pipe, the program equates the orifice discharge equation and the Manning pipe flow equation, i.e:

$$\frac{m}{n} AR^{2/3} S^{1/2} = C_0 A \sqrt{2gh}$$

where:

$m = 1,49$ for U.S. customary units and $1,0$ for metric units,

S = slope of equivalent pipe.

Extran is able to simulate weirs. A schematic illustration of flow transfer by weir diversion between two nodes is shown in Fig. 1.1.10. Flow over a weir is computed by:

$$Q_w = C_w L_w \left[\left(\frac{h + V^2}{2g} \right)^a - \left(\frac{V^2}{2g} \right)^a \right]$$

where:

C_w = discharge coefficient,

L_w = weir length (transverse to overflow),
 h = driving head on the weir,
 V = approach velocity, and
 a = weir exponent, $3/2$ for transverse weirs and $5/3$ for side flow weirs

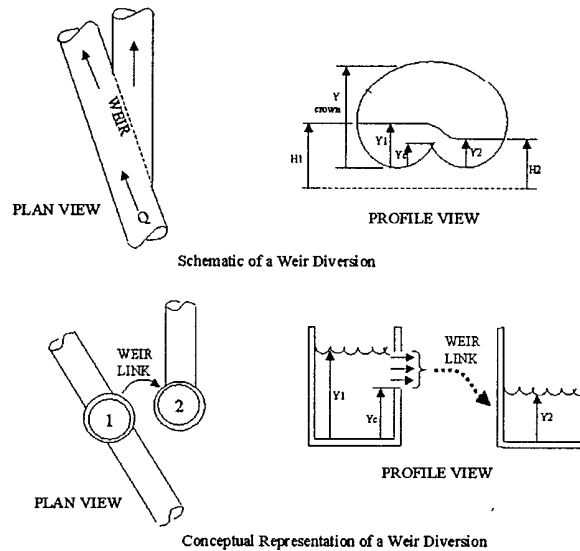


Figure 1.1.10: Weir computational scheme.

If the weir is submerged, the flow is computed as:

$$Q_w = C_{SUB} C_w L_w (Y_1 - Y_c)^{3/2}$$

The submergence coefficient, $CSUB$, is taken from Roessert's Handbook of Hydraulics and is a function of $CRATIO$ defined as:

$$C_{RATIO} = \frac{Y_2 - Y_c}{Y_1 - Y_c}$$

The values of $CRATIO$ and $CSUB$ are computed automatically by Extran and no input data values are needed.

If the weir is surcharged it will behave as an orifice and the flow is computed as:

$$Q_w = C_{SUR} L_w (Y_{TOP} - Y_c) \sqrt{2gh'}$$

where:

Y_{TOP} = distance to top of weir opening shown in Fig.1.1.10

$h1 = Y_1 - \text{maximum}(Y_2, Y_c)$, $CSUR$ = weir surcharge coefficient.

The weir surcharge coefficient, $CSUR$, is computed automatically at the beginning of surcharge.

Extran simulates also pump stations as follows. A pump station is conceptually represented as either an in-line lift station, or an off-line node representing a wet-well, from which the contents

are pumped to another node in the system according to a programmed rule curve. Alternatively, either in-line or off-line pumps may use a three-point pump curve (head versus pumped outflow). For an in-line lift station, the pump rate is based on the water depth, Y , at the pump junction. The step-function rule is as follows:

$$\begin{aligned} \text{Pump Rate} &= R1 \text{ for } 0 < Y < Y1 \\ &= R2 \text{ for } Y1 \leq Y < Y2 \\ &= R3 \text{ for } Y2 \leq Y < Y3 \end{aligned}$$

For $Y = 0$, the pump rate is the inflow rate to the pump junction.

Inflows to the off-line pump must be diverted from the main sewer system through an orifice, a weir, or a pipe. The influent to the wet-well node must be a free discharge regardless of the diversion structure. The pumping rule curve is based on the volume of water in the storage junction. A schematic presentation of the pump rule is shown in Fig.1.1.11.

The step-function rule operates as follows: up to three wet-well volumes are pre-specified as input data for each pump station: $V1 < V2 < V3$, where $V3$ is the maximum capacity of the wet well; three pumping rates are pre-specified as input data for each station.

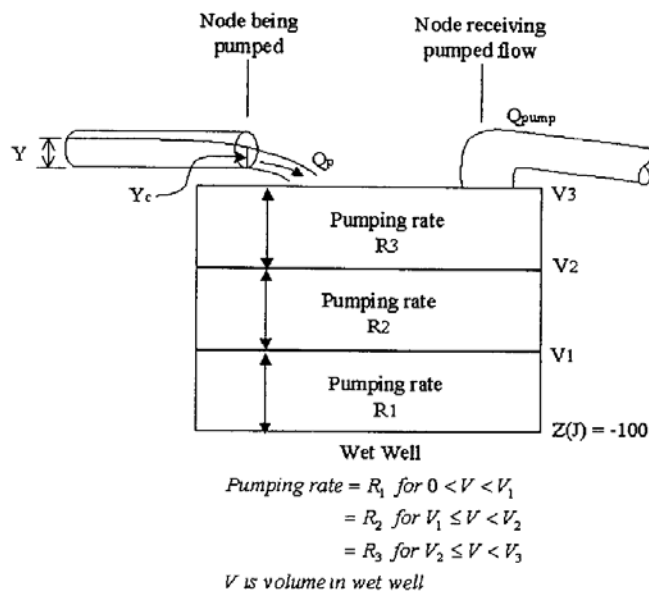


Figure 1.1.11: Pumping station scheme.

A mass balance of pumped outflow and inflow is performed in the wet-well during the model simulation period.

If wet-well goes dry, the pump rate is reduced below rate $R1$ until it just equals the inflow rate. When the inflow rate again equals or exceeds $R1$, the pumping rate goes back to operating on the rule curve.

If $V3$ is exceeded in the wet-well, the inflow to the storage node is reduced until it does not exceed the maximum pumped flow. When the inflow falls below the maximum pumped flow, the inflow “gates” are opened. The program automatically steps down the pumping rate by the operating rule described at point 2 as inflows and wet-well volume decrease.

2.2 Modeling the effects of deterioration processes (Appendix 4: 3D simulations using FLUENT Model)

2.2.1 Deterioration processes

Sewer systems fail in a number of ways. Failures are generally be categorized by the effects they have on their surrounding environment, rather than the condition of the system itself.

According to WP2 classification two general failure categories for sewer systems can be defined:

- **Technical pipe state or failure**
- Structural collapse (due to physical or microbial/chemical deterioration)
- **Operational pipe state or failure**
- Infiltration of groundwater
- Exfiltration of sewage
- Sewer blockages or “chokes” (sediment build up, ragging, root intrusion)
- Design deficiencies 2 (e.g. negative slope, “sags”, bottlenecks, etc.)

Blockages cause a reduction in the transport capacity of the system, backflow (and in turn flooding) and damage to infrastructure. This often inconveniences customers and reduced customer perception of the company; Infiltration increases the volume carried to treatment plant, requiring the plant to be designed with a larger capacity than otherwise required, increasing capital investment and maintenance costs; Exfiltration, while not as readily observable as the other failure modes, results in soil and groundwater contamination. This is a large concern to cities which rely on groundwater for potable water; Structural collapse can occur suddenly and catastrophically resulting in not only interruption to service (similar to blockages) but can also cause large scale destruction of the surrounding environment. This an issue in large size mains, e g located underneath streets.

A model will be required for each of these failure types (see WP2-Deterioration Modelling Approach-Revision5.0). These models will enable a sewer network to be analyzed in detail and the probability of failure for each asset to be predicted based upon its installation/operational environment and its condition.

2.2.2 Developing a transmission mode between deterioration processes / failures coded in CCTV inspection systems and the commercial UDM software

2.2.2.1 Characterization and quantification in accordance with EN 13508 – 2:2001 (*Establishment of the condition of drain and sewer systems outside buildings – Visual inspection coding system*)

The European Standard 13508 – 2:2001 is applicable to the establishment of the condition of drain and sewer systems by inspection, status codification and consideration of external factors and other information.

It is applicable to drain and sewer systems, which operate essentially under gravity, from the point where the sewage leaves a building or roof drainage system, or enters a road gully, to the point where it is discharged into a treatment works or receiving water. The coding system for

drains and sewers comprises a series of codes which shall be used to describe the defects and features found in the drain or sewer. A separate report shall be prepared for each pipeline length. The information recorded is of two main types.

- Header information - relating to the pipeline as a whole.
- Information about individual observations within the drain or sewer.

Drains and sewers - Header information

Requirements

Header information is entered at the start of the inspection. The following information shall be recorded.

- a) The pipeline length identification, by pipeline reference and/or the two node references.
- b) Alternatively, where the pipeline length is a lateral connected to another pipeline without a chamber,
- c) the node at the junction with the main pipeline may be defined using the reference of the main
- d) pipeline and the distance from the start.
- e) The direction of the inspection.
- f) A textual description of the location.
- g) The coding system.
- h) The longitudinal reference point (if recording longitudinal location).
- i) The method of inspection.
- j) The date of inspection.
- k) Whether the drain or sewer was pre-cleaned.
- l) Any other information required by the employing authority.

Other header information

- a) Other information may include:
- b) type of location;
- c) name of employing authority;
- d) name of town, village, district or sewer system;
- e) land ownership.
- f) original coding system (where older data is converted)
- g) time of inspection;
- h) name of inspector;
- i) job reference;
- j) video storage details;
- k) photograph storage details;
- l) purpose of inspection;
- m) cross section;
- n) material;
- o) lining details;
- p) pipe unit length;
- q) depth to invert of upstream and downstream nodes ,
- r) type of drain or sewer (e.g. gravity sewer or rising main);
- s) type of effluent (e.g. wastewater or surface water);
- t) year of construction;

- u) strategic importance;
- v) precipitation;
- w) temperature;
- x) water level;
- y) flow control measures;
- z) atmosphere within the pipeline.

Drains and Sewers - Codes

General

Each observation shall be recorded using an Observation type code which is a main code that broadly describes the feature, together with the following additional information where required.

- Characterisation - up to two codes which describes the feature in more detail.
- Quantification - up to two values which quantify the feature.
- Circumferential location - up to two clockface references which locate the position of the observation
 - around the circumference.
- Joint - identifies when the observation is associated with a joint
- Longitudinal location - the distance from the stated reference point including a method of recording
 - observations which continue over a significant length
- Photograph reference
- Video reference
- Remarks - text which describes aspects of the observation which cannot be described any other way.

The employing authority may specify which observations are to be recorded and the amount of detail that is recorded for each observation.

An example of the record for a longitudinal crack at the top of the pipe 10.5 metres from the start point is shown below:

Main Code	Characterisation		Quantification		Circumferential Location		Joint	Longitudinal location	Video ref.	Remarks
BAB	B	A			12			10.5	00:10:30	

An example of the record for an 100 mm diameter intruding lateral connection (intruding half of the diameter of the main pipe) 16.4 metres from the start point is shown below (Note two codes are necessary to describe the feature):

Main Code	Characterisation		Quantification		Circumferential Location		Joint	Longitudinal location	Video ref.	Remarks
BCA	E	A	100		9			16.5	00:12:20	
BAG			50		9			16.5	00:12:20	

Main code

The main codes have been grouped under 4 headings and the second letter indicates the respective group:

- Codes relating to the fabric of the pipeline (codes BA..),

- Codes relating to the operation of the pipeline (codes BB..),
- Inventory codes (codes BC..),
- Other Codes (codes BD..).

Characterisation

Codes are given to further describe the observation. Up to two types of characterisation are given for each observation type. They shall be recorded in the order in which they are given.

When required by the employing authority, and where an observation cannot be observed (for example if it is not possible to see corrosion in an uncleaned sewer) the code YY shall be used as a first characterisation code.

Only characterisation codes listed in this standard may be used.

Quantification

Up to two values shall be recorded.

Unless these clauses specify the use of the two quantification values differently, the two values may be used to record a band, by specifying the lower and upper limits (e.g. 10% to 15%).

Circumferential location

Where specified, the position of the observation shall be recorded using the clockface reference. The clockface is determined from the angle subtended at the centre of the cross section (the point defined by half the height and half the width) between the observation and the soffit of the pipeline.

The clockface reference shall be determined by reference to Figure 1.

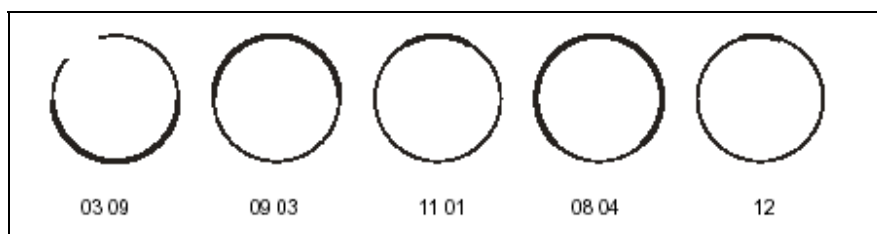


Figure 1 : Example of clockface references

Observation at joint

Where an observation occurs at a joint between two adjacent pipe units this shall be recorded using the code (A) where required.

Longitudinal location

The location of each observation shall be specified by stating the distance in metres from the reference point. The reference point shall be one of the following.

- The inside face of the wall of the starting node, (manhole, inspection chamber or outfall etc.) at the point where the drain or sewer passes through the wall.
- The soffit of the end of the pipeline length inside the starting node. This will be the same point at described in (a) above except where the pipe projects into the manhole.
- The centre of the starting manhole or inspection chamber.
- The midpoint of the incoming and outgoing pipes, measured along the channel.

Where observations continue over a length of more than 1 m, the start and finish of the observation shall be recorded separately, using a continuous observation code containing A (start) or B (finish) and a numeric label which identifies all references to the same observation.

Measurements shall be recorded in metres to one decimal place.

Photograph reference

A reference to identify any still photographs or still computer images shall be recorded against an observation wherever a photograph is taken. If the photograph is of no coded feature the General Photograph (BDA) code shall be used.

Video location reference

Where the inspection is recorded on video, a reference which allows the observation to be located on the video sequence shall be recorded. The method of locating used shall be stated in the header information in accordance with clause 7. Where a time based method is used, the time shall be recorded in accordance with ISO 8601 in a hh:mm:ss format.

Remarks

Where an observation cannot be fully described by a code, further information should be recorded as a remark.

A remark should be as short and descriptive as possible.

2.2.2.2 Failure hydraulic characterization (k , λ , M) generated by 3D simulations

The hydraulic performance is the most important parameter for the analysis of the level service of the wastewater collection system. It depends on the dimension of the pipes, but also on temporal effect of structural deterioration, blockages, roots, sags, etc, which affect the capacity to transport wastewater and runoff, and to avoid local floods and excessive pollution discharges.

WP3.2 assesses the effects of an expected future structural deterioration of hydraulic performance.

Failures, on the CCTV inspection data coding system (European Standard – prEN 13508-2 – *Condition of Drain and Sewer systems outside buildings – Visual inspection coding system*) are divided into groups. The hydraulic capacity of sewer affected by failures is simulated using a mathematical 3D model. Each failure is simulated separately. Through the comparison of a new/clean pipe and a pipe with failures, hydraulic parameters are evaluated. Those parameters describe the real pipe condition better than the default values usually applied in 1D models such as MOUSE, SWMM or InfoWorks. Thus, the model of the sewer system is closer to the reality and the mathematical 1D model can produce more accurate results that, hopefully, require less effort and data for calibration.

The methodology is based on pressure losses on fully filled pipe compared to clean new pipe without any obstacles. The differences between pressure losses of those two pipes gives head loss due by failure.

The inclusion of sewer components temporal decline inside the usual hydraulic simulations permits to know real performances of the system and environmental impacts of hydraulic deficiencies. The main objective consists in developing a rational framework for a model-based investigation of hydraulic, structural and operational performance of a sewer system in accordance with the procedure in the EU 752/5 Rehabilitation standard. The modelling framework will be based on a commercial UD modelling package and CCTV inspection/failure rate database. With the hypothesis that CCTV inspections data are available, and considering that those data are archived following the European coding system by the utilities, the research plan started using the EN classification of failure as input.

A numerical 3D model was used for simulating the hydraulic effects of each failure on the sewer flow. The main output of those simulations are the values of hydraulic parameters that could produce the same effects of the failures in the system; those parameters will be used as input, instead of the usual default ones, on 1D deterministic models commonly used by the end – users.

The 3D model used is FLUENT. The 1D models considered are MOUSE, InfoWorks and SWMM. Simulations using FLUENT were performed on fictitious pipes, trying to represent all the possible failure conditions. Comparing a new/clean pipe with a pipe affected by failure we derived hydraulic parameters that describe the real pipe condition better than the default values. The methodology is based on pressure losses in pipes with full section compared to clean new pipe without any obstacles. Differences between pressure losses of these two pipes can be interpreted as a local head loss due to failure.

Simulated failures – Formula defined

Simulations done with the 3D model allowed to define the formulae able to evaluate the effects of failures on hydraulic performances of the system. Formulae obtained are described in the next pages using the same symbols introduced by the EN 13508 classification code system.

▪ **BAA - Deformation**

Characterization of failure

The cross sectional shape of the pipeline has been deformed from its original shape.

The employing authority may specify whether this code is to be used either for flexible pipes only, or for pipes of all materials.

Characterization of Characterization

The orientation of the deformation:

(A) vertical – the height of the pipe has been reduced.

(B) horizontal - the width of the pipe has been reduced.

Quantification of failure

The percentage change in the dimension which reduces.

Circumferential location

It should correspond with quantification of failure

Formula

$$A_{reduced} = A_{projected} - C1 \cdot Q1 \cdot A_{projected}$$

OR

Exact shape change taken from characterization, quantification and circumferential location

Comments

A = area

▪ **BAC – Break / Collapse**

Characterization of failure

Characterization of Characterization

The nature of the observation:

(A) break – pieces of pipe visibly displaced;

(B) missing – missing pieces of wall;

(C) collapse – complete loss of structural integrity.

Quantification of failure

The length of the observation in millimetres where this is less than 1000 millimetres.

Note: Where the length is longer than 1 meter longitudinal locations of the start and finish of the observation are recorded in accordance with 8.1.7 of E.N.13508-2:2001.

Circumferential location

It should correspond with quantification of failure

Longitudinal location

The location of each observation shall be specified by stating the distance in metres from the reference point. The reference point shall be one of the following.

a) The inside face of the wall of the starting node (manhole, inspection chamber or outfall etc.) at the point where the drain or sewer passes through the wall.

- b) The centre of the starting manhole or inspection chamber.
 c) The midpoint of the incoming and outgoing pipes, measured along the channel.

Where observations continue over a length of more than 1 m, the start and finish of the observation shall be recorded separately, using a continuous observation code containing A (start) or B (finish) and a numeric label which identifies all references to the same observation.

Formula

$$A_{reduced} = C1 \cdot A_{projected}$$

Comments

A = area

▪ **BAD – Defective Brickwork / Masonry**

Characterization of failure

Individual bricks or masonry units from the fabric of a brick or masonry drain or sewer have moved from their original position.

Characterization of Characterization

Characterization 1

The extent of displacement:

- (A) Displaced – bricks or masonry units still present but displaced from their original position.
- (B) Missing – bricks or masonry units missing from their original position.
- (C) Dropped invert - A section of the invert of a brick or masonry pipeline has dropped in relation to the walls leaving a gap of more than 20 mm.
- (D) Collapse – complete loss of structural integrity.

Characterization 2

Where bricks or masonry units are missing:

- (A) Another layer of brickwork or masonry visible – though the hole left by the missing brickwork.
- (B) Nothing is visible – It is not possible to determine what is exposed by the missing brickwork or masonry.

Where soil is visible, or a void is visible the codes BAO or BAP shall also be used.

Quantification of failure

For a dropped invert, the depth of drop in millimetres.

Circumferential location

It should correspond with quantification of failure

Formula

$$A_{reduced} = C1 \cdot C2 \cdot A_{projected}$$

Comments

A = area

▪ BAJ – Displaced Joint

Characterization of failure

Adjacent pipes are displaced from their intended position in relation to each other.

Longitudinal displacements of less than 10 mm shall not be recorded.

Characterization of Characterization

Characterization 1

The type of displacement:

- (A) longitudinal - the pipes are displaced parallel to the line of the sewer;
- (B) radial – the pipes are displaced in a direction at right angles to the line of the sewer;
- (C) angular – the axes of the pipes are not parallel.

Quantification of failure

The quantification measured as:

- (A) For longitudinal displacement - the distance between the end of the spigot and the inside of the socket of the adjacent pipe in mm.
- (B) for radial displacement – the distance of displacement in mm.
- (C) For angular displacements - the angle of displacement between the axes of the two pipes in degrees.

Formula

We can use local head loss coefficient or replace friction during of the pipe. Better way is of course is to use local head losses.

Local head loss coefficient

$$k = C1 \cdot 0,0122 \cdot e^{12,295 \cdot \frac{Q1}{D}}$$

Friction replacement

Replacement of manning number

$$n_{new} = \sqrt{n_{orig}^2 + \frac{k \cdot D^{\frac{4}{3}}}{124,5 \cdot l}}$$

▪ BBA - Roots

Characterization of failure

Roots of trees or other plants growing into the pipeline through joints, defects or connections.

Characterization of Characterization

Characterization 1

The type of root:

- (A) tap root;
- (B) independent fine roots;
- (C) complex mass of roots.

Quantification of failure

The reduction in cross-sectional area expressed as a percentage.

Formula

$$k = C1 \cdot (2,3913 \cdot Q1^2 + 0,0971 \cdot Q1 + 0,0889)$$

▪ BBE – Other Obstacles

Characterization of failure

Objects in the pipeline, obstructing the cross-sectional area. This code shall only be used where none of the other codes BBA to BBD are applicable.

Characterization of Characterization

Characterization 1

The description of the observation:

- (A) dislodged brick or masonry unit lying in invert (A);
- (B) broken piece of pipe material lying in invert (B);
- (C) other object lying in the invert (C);
- (D) protruding through the wall (D);
- (E) wedged in the joint (E);
- (F) entering through a connection/junction pipe (F);
- (G) external pipes or cables built through pipeline (G);
- (H) built into the structure (H).

Quantification of failure

The reduction in cross-sectional area expressed as a percentage.

Formula

For (A)

$$k = C1 \cdot 0,0294 \cdot e^{7,9799 \cdot Q1}$$

For (B), (C), (H)

$$k = C1 \cdot e^{14,673 \cdot Q1^3 - 17,544 \cdot Q1^2 + 12,324 \cdot Q1 - 3,6518}$$

For (D) – (G)

$$k = C1 \cdot 0,026 \cdot e^{6,8158 \cdot Q1}$$

Friction replacement

Replacement of manning number

$$n_{new} = \sqrt{n_{orig}^2 + \frac{k \cdot D^{\frac{4}{3}}}{124,5 \cdot l}}$$

Comments

Brick 240 × 115 × 71 mm

Recalculation for percentage for brick.

2.2.2.3 *Extension of the hydraulic dimension of 1D hydraulic simulator to cover temporal decline in sewer system hydraulic performance - The Nordberg case study #2*

The main objective of the CARE-S WP 3.2 is to provide the end-users of CARE-S Rehabilitation Manager with a possibility for a model-based investigation of temporal decline of sewer system hydraulic performance in accordance with the procedure in the EN752-5 Rehabilitation standard. This chapter describes the modelling framework in which the results of translation of failures derived from CCTV inspection data to 3D CFD numerical software for representation and simulation of the hydraulic behavior of failures has been embedded into the methods of 1D commercial hydraulic simulator. The modelling framework is tested and evaluated by means of 1D MOUSE hydraulic model and the failure database in the Nordberg catchment area in Oslo, Norway. The procedure which was carried out using an already established MOUSE model of the Nordberg sewer system in Oslo, involves three main steps:

- Identification and "quantification" of the classified failures in the Norwegian CCTV inspection system
- Attribution of the hydraulic characterisation of failures (k , λ , M) computed using 3D CFD modelling software (FLUENT) to the existing methods of 1D hydraulic simulator (MOUSE).
- Hydraulic performance modelling and reporting the effects of failures using surface flooding as performance indicator.

Defects/failures data in the Nordberg case study in Oslo

Detection of interior defects is the first step in assessing the condition of pipelines and developing rehabilitation strategies. Oslo municipality uses WinCan software to report CCTV survey results and the Norwegian CCTV classification system. Based on the inspection data to values are calculated:

- Sd – operational pipeline state
- Sm – structural pipeline state

The failures that effect Sd and Sm value are:

Sd - **Joint misalignment** (#, EN13508-2 code BAJ-Displaced joint), **Root intrusion** (%/#, EN13508-2 code BBA-Roots), **Encrustation** (%/#, EN13508-2 code BBB-Attached deposits), **Pipeline intrusion** (#, EN13508-2 code BAG-Intruding connection), **Infiltration** (#/%, EN13508-2 code BBF-Infiltration) and **Visible seal** (#, EN13508-2 code BAI-Intruding sealing material).

Sm – **Crack formation** (%/#, EN13508-2 code BAB-fissure), **Corrosion** (%), EN13508-2 code BAF-Surface damage/Characterisation 2/Chemical attack/ Biochemical attack due to sulfuric acid /Attack by wastewater).

It the Nordberg catchment area approximately half of the sewer system was inspected in 2001. A total of 543 failures were identified from the CCTV inspection database. Of these failures 62 % (338) refer to structural and 38% (205) to operational failures as illustrated in Figure 1.

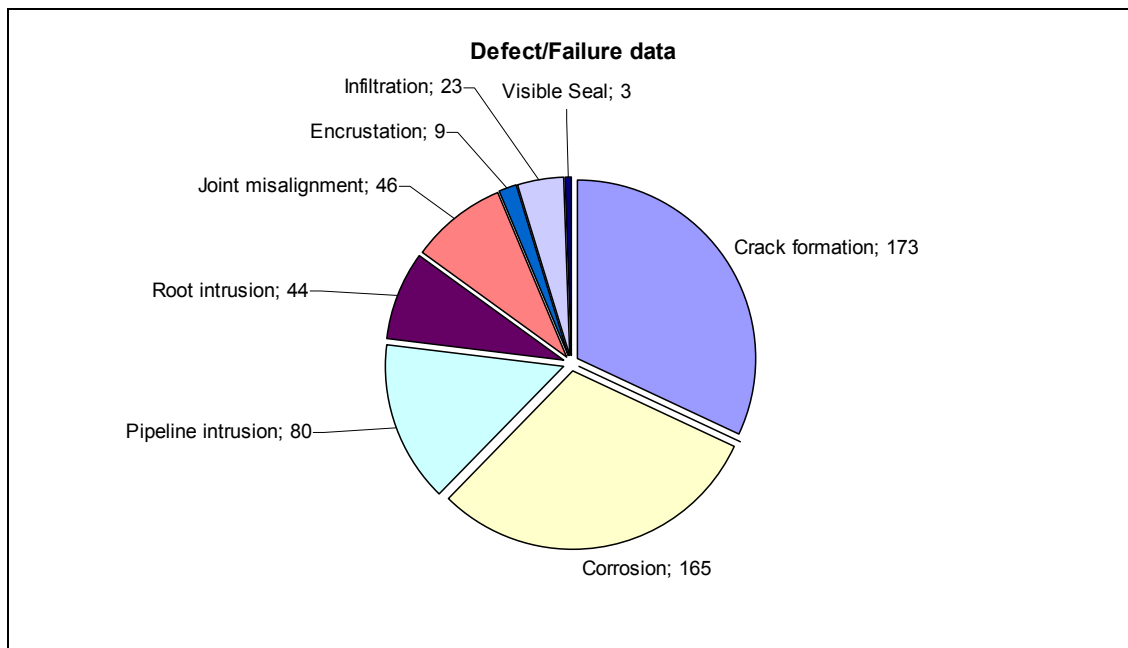


Figure 1 Failure records in the Nordberg catchment area.

Failure raw data "quantification" and parameterization with hydraulic loss

The information about recorded defects/failures in the Nordberg catchment area was defined according to the Norwegian CCTV standard and coding system which is still in use in Norway for CCTV inspection reports, though it has to be replaced by EN13508-2 coding system. Within the CARE-S WP2 frame, the compliance of CCTV classification model put in test in task 2.1 with the new European standard EN13508-2 will allow a precise coding of the severity and extension of the various defects/failures.

According to the Norwegian CCTV standard the structural and operational conditions of the pipeline are classified into 4 classes, ordered to reflect the relative degrees of deterioration of the pipeline. Figure 2 a) shows raw data coded using the Norwegian CCTV standard to express severity and extension of the various defects/failures. Appendix 6.5 provides insight into spatial distribution of the failures.

In order to model the effects of various types of failures on the hydraulic function of sewer system, the value assigned to each class for each failure is translated into hydraulic loss. It is assumed that the value assigned to class 1, 2, 3 and 4 for each failure correspond to 10%, 20%, 30% and 40% of interaction area occupied by obstacle in pipe, respectively. Figure 2 shows four steps in preparing raw input data (a) for use in MOUSE hydraulic model: the "quantification" procedure used to quantify raw failure data coded using the Norwegian CCTV coding system (b1), the parameterisation of "quantified" failures with hydraulic loss calculated using the 3D FLUENT model (b2), the calculation of Manning's number as a sum of all hydraulic losses due to a set of failures which exist in the pipeline description file (c).

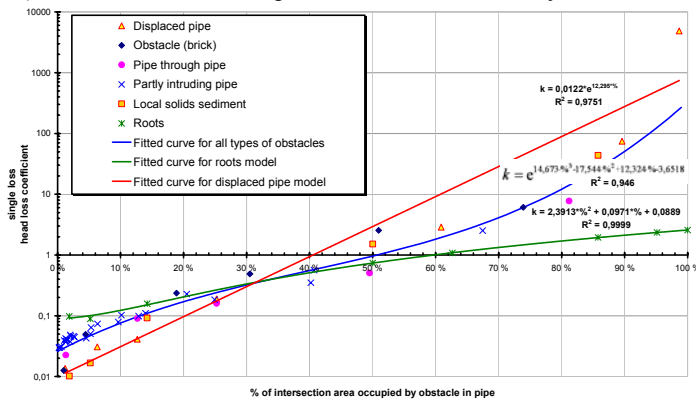
a) Raw input data – The Norwegian coding system

Pipe ID number	Joint misalignment				Pipeline Intrusion				Root intrusion				Encrustation			
	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
260166			1		1											
260891									15 %							
248567					1	2			2 %	22 %	10 %					
242532					2											
260686																
260876			1													
260878						2										
260877					1											
260145			2													
260144			5		2											

b1) "Quantification" procedure used to quantify raw failure data coded using the Norwegian CCTV coding system (grey rows)

Pipe ID number	Length	Diameter	NO class				NO class				NO class				NO class													
			0	1	2	3	4	0	1	2	3	4	0	1	2	3	4	0	1	2	3	4						
			k for BAG characterisation				k for BAJ characterisation				k for BBB characterisation				k for BBA characterisation													
			0 %	10 %	20 %	30 %	40 %	0 %	10 %	20 %	30 %	40 %	0 %	10 %	20 %	30 %	40 %	0 %	10 %	20 %	30 %	40 %						
260166	55.80	300				0,06						0,03										0,03						
260891	49.26	230		0,03								0,03																0,06
248567	79.63	300				0,14						0,03																0,06
242532	58.80	230				0,06						0,03																0,03
260686	61.47	230		0,03								0,03																0,03
260876	73.45	230		0,03								0,03																0,03
260878	60.53	230				0,14						0,03																0,03
260877	54.21	230				0,06						0,03																0,03
260145	81.00	230		0,03								0,03																0,03
260144	88.80	230				0,06					0,06																	0,03

b2) Parameterisation of "quantified" failures with hydraulic loss calculated using the 3D FLUENT model (c),



c) Calculation of Manning's number

Pipe ID number	ALL FAILURES			
	$k = \frac{\Delta h_{loc} \cdot 2g}{v^2} = \frac{2\Delta p_{loc}}{\rho v^2}$	$\lambda = \frac{\Delta p \cdot D \cdot 2}{\rho \cdot l \cdot v^2}$	$n = \sqrt{\frac{\lambda \cdot D^3}{124,5}}$	
	k1+k2+k3+k4	LAMBDA	MANNING n	MANNING M=1/n
260166	0,15	0,00081	0,02545	39
260891	0,15	0,00070	0,02077	48
248567	0,26	0,00098	0,02805	36
242532	0,15	0,00059	0,01901	53
260686	0,12	0,00045	0,01663	60
260876	0,12	0,00038	0,01521	66
260878	0,23	0,00087	0,02320	43
260877	0,15	0,00064	0,01980	51
260145	0,12	0,00034	0,01448	69
260144	0,18	0,00047	0,01694	59

Figure 2 Four steps in preparing raw input data for use in MOUSE hydraulic model.

Modelling the effects of various type of failures on the sewer system hydraulic performance by means of MOUSE model

The parameter which characterize the pipe material in 1D UD modelling tools is the pipe friction, usually expressed as Manning's number (n or $M=1/n$). Table 1 shows the available pipe materials with MOUSE default values for Manning's number. The default Manning's number for any individual pipe can be overwritten by a user-specified, pipe-specific value.

Head losses in pipes	
Type of material	Manning's number
Smooth concrete	85
Normal concrete	75
Rough concrete	68
Plastic	80
Iron	70
Ceramics	70
Stone	80
Other	50

Table 1 MOUSE default values for Manning's numbers for pipes.

Energy losses in manholes are of the same order of magnitude as those caused by the pipe wall friction. Table 2 shows nine different options for calculation of energy losses in manholes (junctions) and default values for the head losses coefficient. Per default the specific value is interpreted as the outlet shape coefficient K_m . In addition to the default interpretation of the head loss coefficient K_m , two alternative interpretations which ignore the geometric relations between the manhole and the outlet pipe (outlet shape) and apply the specific value directly as the ζ_{cont} can be selected.

Head losses in manholes defaults values for K_m	
Round edged outlet	0.25
Sharpe edged outlet	0.50
Orificing outlet	0.50
No cross section changes	0.00
Energy loss	0.50
No cross section changes (new)	0.00
Effective flow area (1)	0.25
Effective flow area (2)	0.25
Mean energy approach	0.25

Table 2 MOUSE default values for the head losses in manholes (K_m).

Analysis of the effects of various types of failures on the hydraulic function of sewer system in the Nordberg case study

In the analysis of the effects of various types of failures on the hydraulic function of sewer system in the Nordberg case study, the default values of Manning's number for any individual pipe was overwritten by the calculated values of Manning's number using the procedure of failure hydraulic parameterization described in the pervious chapter. This approach is to be superior to the approach which assigns the calculated head looses due to failure to manholes

because the knowledge about the magnitude of energy losses in manholes, based on experimental data, is very limited.

Figure 3 shows how the individual failures calculated by the 3D CFD software FLUENT in terms of Manning's number were included in the MOUSE input table for network. (Network/Specific Hydraulic Parameters/Friction Loss: link ID, Manning's number).

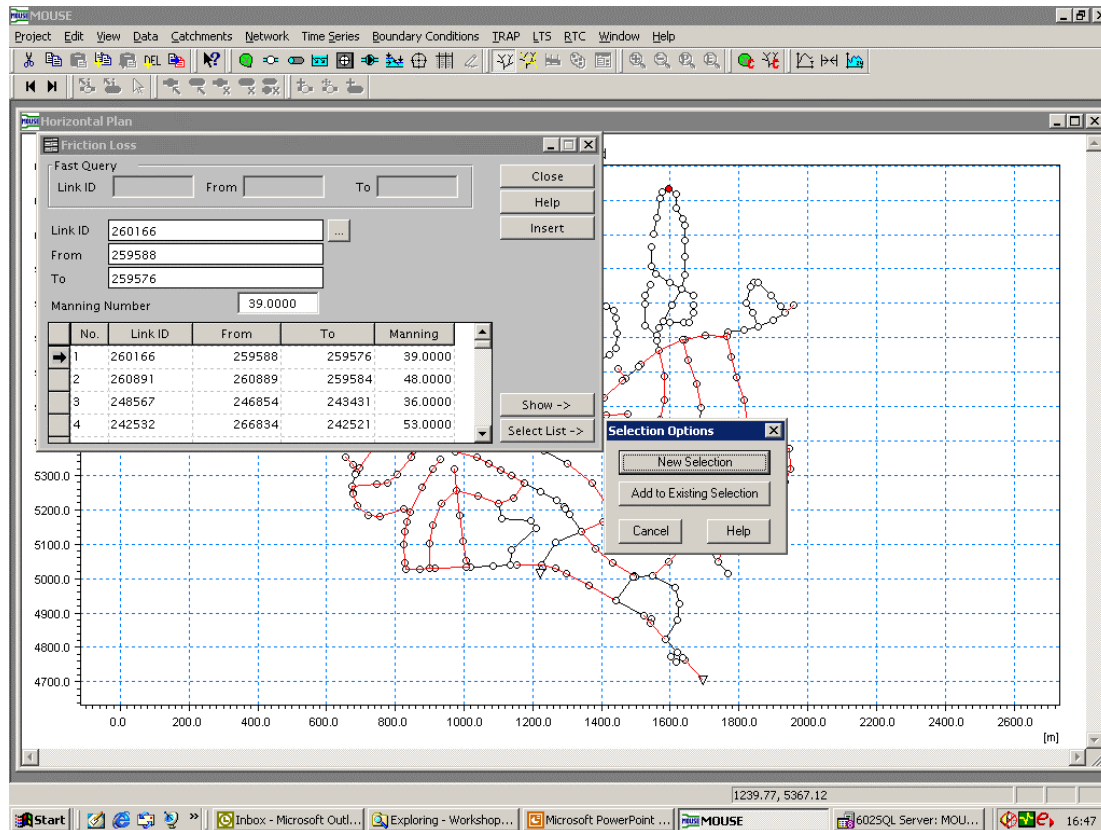
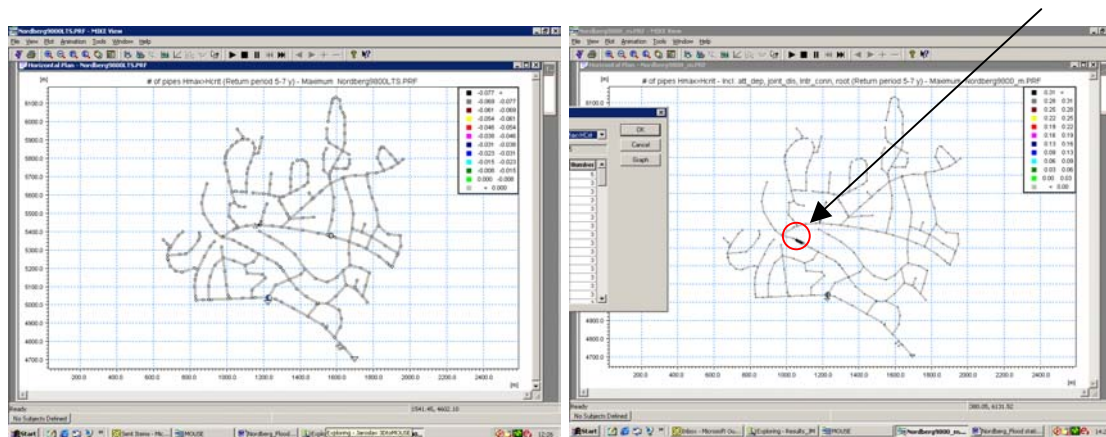


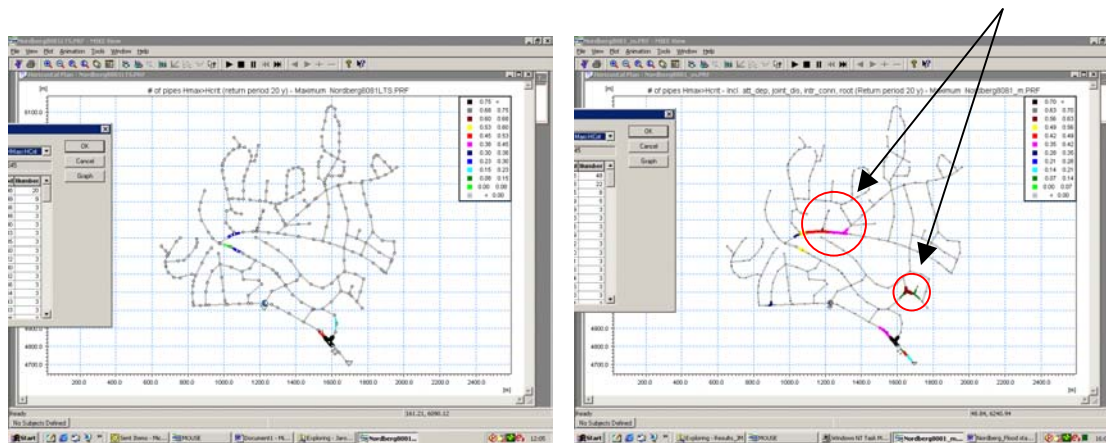
Figure 3 Input of the hydraulic characteristic of failures calculated by 3D CFD software in term of Manning's number in the MOUSE hydraulic model of sewer system in the Nordberg catchment area in Oslo (Network/Specific Hydraulic Parameters/Friction Loss: link ID, Manning's number).

Figure 4 summarized the results of the simulation run in the Nordberg sewer system in Oslo according to procedure which is required for evaluation of the hydraulic performance of an existing sewer system. The results of the analysis are reported using surface flooding as performance indicator. The hydraulic consequences of the modelled failures are presented on the right side of Figure 4 while the hydraulic consequences of a new sewer system are shown on the left side of Figure 4. While the new sewer system can cope with event which is only exceeded in severity once in 5 years, the temporal decline of the "old" system leads to one localised surface flooding incident. For event which is only exceeded in severity once in 20 years leads the "old" system to increase number of pipelines causing surface flooding and the extend of flooding incidents.



a1) Model without failures -
There is no surface flooding

a2) Failures included into model -
There is surface flooding



b1) Model without failures -
There is surface flooding

b2) Failures included into model -
Surface flooding is severer

Figure 4 Effects of a set of failures on frequency of surface computed using 1D simulator (MOUSE). a) Events with 5 years return period, b) Events with 20 years return period.

2.3 Hydraulic performance based on hydraulic model simulation of sewer at the future state

The combination of WP3 deterministic models and WP2 probabilistic models gives information about the possible future conditions of the system.

Work Package 2 will develop models to define the current state of the pipe assets and the relationship between the current state and the expected service time of sewer systems using CCTV classification system developed in 2.1 as the main source of input. The models developed within this task will be based on empirical and statistical analysis of asset data as well as physical functions to define the failure probability or failure frequency (if possible), taking state variables into account. This task will interact heavily with task 2.1 to allow the use of CCTV systems to characterize the current state probability or frequency of failure and task 2.3 to allow the current state classification to be linked with a degradation model to predict future current

CCTV coding system developed by the EN 13508 is the starting point common to WP3.2 and WP2 for the analysis of system deteriorated states.

Task 3.2 through 3D simulations found how to simulate the effects of failure on hydraulic performances of the system. Changing the Manning coefficient or introducing a k – coefficient of local headloss for the pipe affected by failure, instead of the default values, it is possible to run 1D UD models for a deteriorated system. Manning coefficient or the k – coefficient has to be calculated with the formulas developed after 3D simulations (chapter 6.4.7).

Known the current state of the system, WP2 models allow to establish the probability that a specific pipe will be affected by failure after a defined period of time. WP2 models already developed, base the prediction of future conditions of the network as the probability of transition from a condition class to another in a given period of time. Condition classes are defined as groups of failure named following the EN13508 code classification. All the condition classes include one failure code, at least, that was simulated by the 3D model of task 3.2, so, for that failures, and as consequence, for all the condition classes, it is possible to calculate how will change the Manning coefficient or the k – coefficient of the pipe passing from one to another condition class. Using the roughness value calculated as input of the 1D UD model for that pipe, it is possible to run simulations taking in account the future assessment of the system as described by the WP2 models.

2.3.1 Developing a transmission mode between the future structural state of sewer pipelines and the hydraulic characterization of structural failures

Overview of the Structural model developed by Cemagref “Degradation Models for Drain and Sewer Pipelines Parameterised with CCTV Inspection Data” – Yves Le Gat:

At any time of the service life of the pipeline, its structural condition can be classified into of a finite number m of states, 1, 2, . . . , m , ordered to reflect the relative degrees of deterioration of the pipeline. It is proposed to consider the following successive four

Deterioration states:

- 1 = perfectly good,
- 2 = presence of at most surface damage and/or cracks,
- 3 = presence of fractures,
- 4 = collapse.

It is proposed to define the deteriorated states 2, 3 and 4 according to the EN13508-2 coding system in the way specified in table 1.

Condition Classes	Defect Codes
2	BABA (Surface cracks) or BABB (Thin fissure) or BAE (Missing mortar *) or BAF (Surface damage, BAFI excluded) or BAK (Defective lining)
3	BABC (Fracture with wall pieces still in place) or BACA (Break with wall pieces still in place) or BADA (Masonry units displaced *) or BAFI (Surface damage with missing wall)
4	BAAA (Vertical deformation **) or BACB (Break with displacement) or BACC (Complete loss of structural integrity) or BADB (Missing masonry units *) or BADC (Dropped invert *) or BADD (Complete loss of structural integrity *)

* masonry pipelines
** flexible materials excluded

Table 1: Definition of Condition Classes using EN13508-2 Defect Coding

Moreover a continuous observation code enables to record the pipeline length concerned by a longitudinal defect that extends over more than 1 meter. It is then proposed to attribute a default length extension of 1 meter to any punctual defect, e.g. a circumferential fissure or break. The EN13508-2 coding system enables to split the total length L of the pipeline into partial lengths $L_j, j = 1, \dots, m$, with $\sum_{j=1}^m L_j = L$ and $L_j \leq L$, L_j being the pipeline partial length in state j.

An example of input data

The table 2 shows an example of CCTV inspection report coded with the EN13508-2 system. This information is translated into structural condition classes and relative length concerned, leading to the table 3. The condition class data of table 3 must then be crossed with the pipeline description exemplified in table 4, in order to add to each row of the condition class file as many columns of covariates value as there exist in the pipeline description file ; the covariates are defined in table 5. The file obtained, as in table 6, by merging the condition class file and the pipeline description file by pipeline identification code is then usable for performing an Ordered Probit Analysis.

Pipeline Identification Code	Longitudinal Location (m)	Continuous Defect Code	Observation Code	CCTV Inspection Date
1	0		BCDA	21/04/2001
	5	A1	BABC	
	8	B1		
	15	A2	BACC	
	20	B2		
2	53		BCEA	07/09/1999
	0		BCDA	
	11		BABA	
3	24		BCEA	02/05/1993
	0		BCDA	
	6	A1	BAFC	
	18	B1		
	19		BCEA	

Table 2: Example of Raw Input Data - Defects Coded Using the EN13508-2 System

Id ¹	Y ²	w ³	Date
1	1	0.85	21/04/2001
	2	0.00	
	3	0.06	
	4	0.09	
3	1	0.96	07/09/1999
	2	0.04	
	3	0.00	
	4	0.00	
2	1	0.21	02/05/1993
	2	0.79	
	3	0.00	
	4	0.00	

¹ Pipeline Id.

² Condition Class

³ Relative Length

Table 3: Example of Condition Class Input Data

Id	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈
1	Co	200	53	2.20	W	1947	M	P
2	CI	100	24	2.80	C	1905	H	S
3	AC	150	19	1.20	R	1967	L	G

Table 4: Example of Pipeline Description File

Covariate	Label	Unit	Values
X ₁	Material		Co = Concrete AC = Asbestos-Cement CI = Cast Iron
X ₂	Diameter	mm	
X ₃	Length	m	
X ₄	Depth	m	
X ₅	Effluent		W = Waste Water R = Rain Water C = Combined
X ₆	Construction Year		
X ₇	Traffic Level		H = High M = Medium L = Low
X ₈	Located under		P = Pavement S = Sidewalk G = Green Space

Table 5: Covariates Labels, Units and Values

Id	Y	w	t	X ₁	X ₂	X ₃	X ₄	X ₅	X ₆	X ₇	X ₈
1	1	0.85	54	Co	200	53	2.20	W	1947	M	P
	2	0.00	54	Co	200	53	2.20	W	1947	M	P
	3	0.06	54	Co	200	53	2.20	W	1947	M	P
	4	0.09	54	Co	200	53	2.20	W	1947	M	P
2	1	0.96	94	CI	100	24	2.80	C	1905	H	S
	2	0.04	94	CI	100	24	2.80	C	1905	H	S
	3	0.00	94	CI	100	24	2.80	C	1905	H	S
	4	0.00	94	CI	100	24	2.80	C	1905	H	S
3	1	0.21	26	AC	150	19	1.20	R	1967	L	G
	2	0.79	26	AC	150	19	1.20	R	1967	L	G
	3	0.00	26	AC	150	19	1.20	R	1967	L	G
	4	0.00	26	AC	150	19	1.20	R	1967	L	G

Table 6: Inspection Data merged with Pipeline Description

An example of output data

Table 7 shows an example of output data related to forecast computation with the end of year 2005 as horizon date.

Id	t	Y	w
1	58	1	0.80
		2	0.02
		3	0.08
		4	0.10
2	100	1	0.91
		2	0.06
		3	0.03
		4	0.00
3	38	1	0.15
		2	0.72
		3	0.13
		4	0.00

Table 7 : Example of output Data – Forecast Horizon Date: 31/12/2005.

An example of graphical results

The graph of figure 1 is a fictitious example of the Weibull distribution functions that could be obtained with a sufficiently large sample of CCTV-Inspection reports. The Weibull distribution is indeed very flexible with respect to the shape of its distribution function. In the graph of figure 1, the power of t in formula (4.1) has been chosen > 1, which leads to a S-shaped cumulative distribution function.

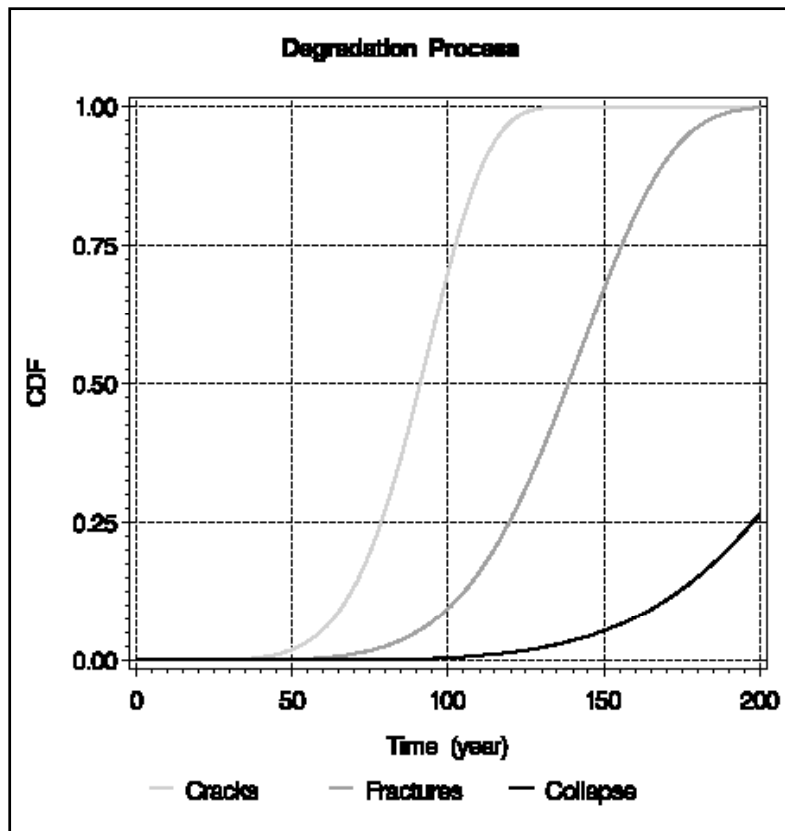


Fig.1 : Fictitious example of Weibull CDF related to successive degradation classes

How to predict hydraulic performance after a period of time.

Starting from the “Structural Model” results it is possible to know the probability that a pipe will go from one condition class to another after a given period of time.

Using the “recalculation matrix” developed by task 3.2 (see chapter 6.4.7) for the evaluation of roughness coefficient for pipe affected by specific kind of failure, it is possible to calculate the Manning coefficient or the K – coefficient of local head loss for each condition class.

Combining results coming from the two models, the probability of being in one condition class and the new roughness coefficient to use as input to the 1D models if the probability is considered high enough are known.

Running the 1D model with those new input it is possible to simulate the hydraulic performance of the network after a given period of time considering changes in structural condition, at least.

If possible the simulation can consider other different data from the ones used in the simulation of the current condition of the system, such as precipitation information, catchment evolution, population, depending on the availability of those information.

2.3.2 Developing a transmission mode between the future probability of blockages and the hydraulic characterization of blockages

Waiting for WP2.2 Blockages Model.

3 CARE-S - Cross-package development and integration

3.1 WP3 Internal links

3.1.1 WP3.2-WP3.3 (Water balance at CSO and outlet)

Data and information required by task 3.3 from task 3.2 include all the results regarding Volume balance, Hydrograph in outlet , frequencies of spills for CSO's, and some of the input data used for the Hydraulic simulations input data (i.e. population, area, rain series,... etc). The following table lists all the information available for task 3.3 from task 3.2 output data:

Table 1 MOUSE Summary file (*.HTM). Continuity balance – Infiltration.

Continuity Balance			
Start volume in Pipes, Manholes and Structures			0,951 m3
End volume in Pipes, Manholes and Structures			35,605 m3
Total inflow volume			
3.1 Runoff :	6746,502 m3		
3.2 Boundary :	0,000 m3		
3.3 DWF :	1834,402 m3		
3.4 Outlets (inflow) :	0,000 m3		
3.5 Infiltration :	320,640 m3		
	8901,544 m3	-->	8901,544 m3
Total diverted volume			
4.1 Weirs :	0,000 m3		
4.2 Pumps :	0,000 m3		
4.3 Spilling nodes :	0,000 m3		
4.4 Outlets :	8867,876 m3		
	8867,876 m3	-->	8867,876 m3
Water generated in empty parts of the system :			3,816 m3
Continuity balance = (2-1) - (3-4+5) :			-2,829 m3

Results coming from task 3.2 simulations are resumed in a text file as described in chapter 3.2.2.1.

3.1.2 WP3.2-WP3.4 (introductory study for combining hydraulic and reliability model)

Task 3.4 will produce, as final result, probability maps.

The probability maps will be:

- hydraulic maps
- environmental maps
- maybe operational maps, depending on accordance with WP2.

Hydraulic maps will define the probability to reach or to exceed a critical water level for each pipe and for each manhole. The critical level will be not define by CARE-S but every end – user will use his national / local standard.

For producing those maps task 3.4 needs from task 3.2 Long Term Simulations results in term of water level inside the pipes.

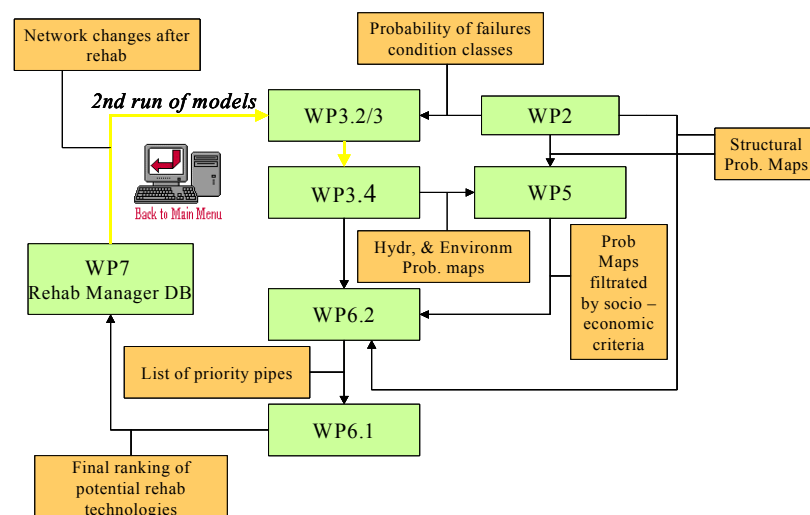
Task 3.4 will collect water level results from 3.2 for each pipe in x years of simulations and graphically will find “every how many years the specific pipe reaches or exceeds the critical level” .

If required task 3.4 will have velocity inside the pipes from task 3.2 to see pipe where velocity is < of... m/s.

Results coming from task 3.2 simulations are resumed in a text file as described in chapter 3.2.2.1.

3.2 WP3 External links

Tasks 3.2 and 3.3 transmit results to task 3.4 which will produce the Hydraulic and Environmental probabilistic Maps. Structural probabilistic Maps will come from WP2 in addition. All the probabilistic Maps become input to both WP5 and task 6.2. From WP5 the probabilistic maps filtrated by socio – economic criteria go to task 6.2 too. From the analysis of the maps, task 6.2 produces the list of priority pipes in terms of rehabilitation request and gives this list to task 6.1 which define the final ranking of potential rehabilitation technologies. The effect of rehabilitation technologies on the network conditions is recorded on the Rehabilitation Manager. Changes produced in the system are defined and the 1D UDM models input corrected. With the new input files is possible to run again the procedure once developing the new probability maps after rehabilitation.



3.2.1 WP5 and WP6 (data transfer formats)

Results coming from task 3.2 simulations are resumed in a text file as described in chapter 3.2.2.1.

For WP5 and 6 use critical pipes, or pipes showing high levels of rehabilitation needs, can be highlighted by task 3.2 results in different ways.

Task 3.2 output useful for produce the task 3.4 Maps which will be processed by WP5 and 6 are flood relating problems and velocity values coupled with sedimentation problems.

- flood at pipe level:

Critical pipes can be defined with the following possible alternatives:

- 1) Yes or not ? (is the pipe responsible for flood problems?)
- 2) Pipe filling maximum inside the pipe (ratio between pipe water level and pipe diameter inside the pipe)

- 3) Classification of pipes in 4 classes defined by the comparison of water level inside the pipe and a critical level. Critical level must be defined by the end – user. (1st class: full pipe, 2nd class: up to critical level, 3rd class: from critical level to ground level, 4th level: above ground level. The maximum critical level will be the ground level).
- 4) For each pipe after Long Term Simulation will be defined the probability of reaching the critical level.

- Velocity coupled with sedimentation problems

ID of all pipes

ID of pipes where velocity is minor than a fixed value (British Standard)

Other hydraulic results will be produced as additional help for end user for rehabilitation plan and visualized by Rehab Manager GIS system.

3.2.2 WP7 (data transfer formats)

MOUSE - InfoWorks - SWMM data structure

This chapter aims to define data produced by the Urban drainage models considered by the CARE-S project : MOUSE, InfoWorks and SWMM.

The following table presents the hydraulic results provided by task 3.2 to the other WP3 tasks and CARE-S WPs. In the same table, simulation typologies from which deriving the specific result, are listed.

Simulation			
Type	Example	Final Results	Required by
-Single Event <i>synthetic or real event</i> <i>(Assigned Return Period)</i>	Chicago	Pipe “Level” <i>max flow</i> <i>max velocity</i> <i>water level</i>	Task 3.4 Wp5 Wp6
		Node “Level” <i>max level</i>	
-Long Term Simulations	One year of rain	Weir “Level” <i>CSOs Frequency</i> <i>Spilled Volume</i> <i>Mean velocity</i>	Task 3.3
-Dry wether flow simulation	DW profile	Pipe “Level” <i>Mean velocity</i>	Wp5 Wp6

MOUSE, InfoWorks and SWMM are able to produce the hydraulic results as binary code, in this way results can be analysed directly inside the simulation program. Some of the results can be produced by InfoWorks and SWMM as a ASCII report file automatically and by MOUSE as a htm file. For all the three model is possible the exportation of the Times series as text file manually (Table 2 and 3)

Output				
Type Out		InfoWorks	Mouse 2003	Swmm 5.0
Output file Format		<i>Binary</i>	<i>Binary</i>	<i>Binary</i>
It is possible the Output exportation to text file?	Aut.	<i>Only the report file</i>	<i>Only the report file</i>	<i>Only the report file</i>
	Man.	<i>Time Series</i>	<i>Time series</i>	<i>Time Series</i>

Output data could be analysed at two different levels:

- 1) the first level as a report in ASCII format for InfoWorks and SWMM or html format for MOUSE, where the main results of the simulations are list in terms of maximum values or global values of the hydraulic parameters.
- 2) The second level as a binary code which helps in knowing the temporal variation of the hydraulic parameters. The binary systems unable any automatic access from external tools because this operation can be done only working inside the specific model considered. The results written in binary code can be copied on the ASCII file but this operation could not be done from external tools.

In the next pages the two levels are described; a description of the input file required by the model as text file useful for the calculation of the hydraulic parameters are defined.

Output as ASCII REPORT FILE

In the next tables the ASCII report file of each UDM model are list. Only the hydraulic parameters required by the CARE-S tasks are described (quality parameters are not included)

Infoworks:

Report file extension: *.prn

Global data Results	
Total rainfall	Rainfall volume on the catchment during the simulation
Total runoff	Rain volume due to surface drainage ;
Total inflow	Total inflow;
Total outflow	Total outflow;
Total lost	Hydrological lost

Node data Results								
Node Reference	Ground Level	Max Level	Flood Volume	Flood Depth	Flood Area	Max Stored	Inflow	Vol Balance
N1	N2	N3	N4	N5	N6	N7	N8	N9
N1	A unique name for the node, up to 30 letters, numbers and underscore characters;							
N2	Ground level;							
N3	Max level in the simulation at the node;							
N4	Cumulated Flood volume at the node in the simulation;							
N5	Max flood depth at the node in the simulation;							
N6	Flood area;							
N7	Max volume stored in the node;							
N8	Volume of inflow in the node;							
N9	Volume balance = total outflow - total inflow - change in storage.							

Link data Results															
Link Reference	D/S Node	Pipe Len	Pipe Hgt	Sed Dpth	P.Full Flow	Invert Level	Upstream				Downstream				
							Max Depth	Max Flow	Max Vel	Total Flow	Invert Level	Max Depth	Max Flow	Max Vel	Total Flow
L1	L2	L3	L4	L5	L6	L7	L8	L9	L10	L11	L12	L13	L14	L15	L16
L1	Link reference;														
L2	Downstream node ID;														
L3	Pipe length;														
L4	Pipe height;														

L5	Height of the sediment layer;
L6	Pipe full flow design;
L7	Invert level at the upstream section;
L8	Max depth of water on the invert level;
L9	Max flow at the upstream section;
L10	Max velocity at the upstream section;
L11	Total volume of water through the section in the simulation;
L12	Invert level at the Downstream section;
L13	Max depth of water on the invert level;
L14	Max flow at the Downstream section;
L15	Max velocity at the Downstream section;
L16	Total volume of water through the section in the simulation.

Link data Results "WEIR"														
Link Reference	D/S Node	Pipe Len	Sed Dpth	P.Full Flow	Upstream					Downstream				
					Invert Level	Max Depth	Max Flow	Max Vel	Total Flow	Invert Level	Max Depth	Max Flow	Max Vel	Total Flow
L1	L2	L3	L5	L6	L7	L8	L9	L10	L11	L12	L13	L14	L15	L16
L1	Link reference;													
L2	Downstream node ID;													
L3	Pipe length;													
L5	Height of the sediment layer;													
L6	Pipe full flow design;													
L7	Invert level at the upstream section;													
L8	Max depth of water on the invert level;													
L9	Max flow at the upstream section;													
L10	Max velocity at the upstream section;													
L11	Total volume of water through the section in the simulation;													
L12	Invert level at the Downstream section;													
L13	Max depth of water on the invert level;													
L14	Max flow at the Downstream section;													
L15	Max velocity at the Downstream section;													
L16	Total volume of water through the section in the simulation.													

SWMM:

Input file extension: *.inp

Node data Input [JUNCTIONS]					
Name	Invert Elevation	Max. Depth Ground lev-Invert elev.	Init. Depth	Surcharge Depth	Ponded Area
IN1	IN2	IN3	IN4	IN5	IN6
IN1	A unique name for the node;				
IN2	invert elevation of the node;				
IN3	Maximum Depth = Ground level - Invert elevation;				
IN4	Depth of water at the junction at the start of the simulation;				
IN5	Additional depth of water beyond the maximum depth that is allowed before the junction floods. This parameter can be used to simulate bolted manhole covers.				
IN6	Area occupied by ponded water atop the junction after flooding occurs. If the Allow Ponding analysis option is turned on, a non-zero value of this parameter will allow ponded water to be stored and subsequently returned to the conveyance system when capacity exists.				

Link data input [CONDUITS]							
Name	Inlet Node	Outlet Node	Length	Manning N	Inlet Height	Outlet Height	Init. Flow
IL1	IL2	IL3	IL4	IL5	IL6	IL7	IL8
IL1	Link reference;						
IL2	inlet node name (upstream);						
IL3	Outlet node name (downstream);						
IL4	Length of the pipe;						
IL5	Manning's roughness coefficient.;						
IL6	Height of the conduit invert above the node invert at the upstream end of the conduit;						
IL7	Height of the conduit invert above the node invert at the downstream end of the conduit;						
IL8	Initial flow in the conduit at the start of the simulation.						
Link data input [XSECTIONS]							
Link Name	Type	Geom1	Geom2	Geom3	Geom4		
IL9	IL10	IL11	IL12	IL13	IL14		
IL9	Link reference;						
IL10	Type of section shape;						
IL11	Maximum depth;						
IL12	Bottom width;						
IL13	Left side slope;						
IL14	Right side slope.						

Report File Extension: *.rpt

Global data Results	
Flow Units	Unit System adopted;
Dry Weather Inflow	
Wet Weather Inflow	
Groundwater Inflow	
External Inflow	
External Outflow	
Initial Stored Volume	
Final Stored Volume	

Node data Results						
Node	Average Depth	Maximum Depth	Time of Max Occurrence days hr:min	Average Depth Change	Total Minutes Flooded	Fraction Courant Critical
N1	N2	N3	N4	N5	N6	N7
N1	A unique name for the node;					
N2	Average depth of water on the invert level;					
N3	Max depth of water on the invert level;					
N4	Time of Max depth Occurrence;					
N5	Average Depth Change					
N6	Total Minutes Flooded;					
N7	Fraction Courant Critical.					

Link data Results						
Conduit	Maximum Flow	Time of Max Occurrence days hr:min	Maximum Velocity	Time of Max Occurrence days hr:min	Maximum /Design Flow	Total Minuts Surcharged
L1	L2	L3	L4	L5	L6	L7
L1	Link reference;					
L2	Max flow in the pipe;					
L3	Time of Max flow Occurrence;					
L4	Max velocity in the pipe;					
L5	Time of Max velocity Occurrence;					
L6	Pipe full flow design;					
L7	Total Minutes Surcharged in the simulation.					

Mouse

Input file extension: *.und.txt

Node data Input [NUMBER CIRCULAR MANHOLES]						
NODE	X-COOR	Y-COOR	BOTTL	TOPL	SHP	DIAM
IN1	IN2	IN3	IN4	IN5	IN6	IN7
IN1	A unique name for the node;					
IN2	X coordinate of the node;					
IN3	Y coordinate of the node;					
IN4	Bottom level of the manhole;					
IN5	Top level of the manhole;					
IN6	Shape of the manhole;					
IN7	Diameter of the manhole.					

Link data input [CONDUITS]											
	NODE-U	NODE-D	M	A	BL-U	BL-D	A	FLOW	GW-LE	A	DIAM
	IL1	IL2	IL3	IL4	IL5	IL6	IL7	IL8	IL9	IL10	IL11
IL1	inlet node name (upstream);										
IL2	Outlet node name (downstream);										
IL3											
IL4											
IL5	Bottom level of the pipe upstream section;										
IL6	Bottom level of the pipe downstream section;										
IL7											
IL8	Additional Flow;										
IL9	Ground water level?										
IL10											
IL11	Diameter of the pipe's section.										

Report file extension: *.Htm

Global data Results
Start volume in Pipes, Manholes and Structures;
End volume in Pipes, Manholes and Structures;
Total inflow volume
Total diverted volume, (Weirs, Pumps, Spilling nodes, Outlets)

Node data Results						
Nodes - Water level						
Node	Minimum	Maximum	Ground Level	Ground Level - Maximum	Time - Minimum	Time - Maximum
N1	N2	N3	N4	N5	N6	N7
N1	A unique name for the node;					
N2	Minimum water level;					
N3	Maximum water level;					
N4	Ground Level;					
N5	Maximum Ground level;					
N6	Time of Minimum water level Occurrence;					
N7	Time of Maximum water level Occurrence.					

Node data Results “Discharge”					
Nodes - Water level					
Node	Minimum	Maximum	Flow accumulated	Minimum	Maximum
N1	N2	N3	N4	N5	N6
N1	A unique name for the node;				
N2	Minimum flow;				
N3	Maximum flow;				
N4	Flow accumulated;				
N5	Time of Minimum flow Occurrence;				
N6	Time of Maximum flow Occurrence;				

Link data Results									
LinkID	From Node	To Node	Hmax	Qmax	Hmax/D	Qmax/Qf	Flow - Accumulated	Time - Hmax	Time - Qmax
L1	L2	L3	L4	L5	L6	L7	L8	L9	L10
L1	Link reference;								
L2	inlet node name (upstream);								
L3	Outlet node name (downstream);								
L4	Max water level at the upstream section of the pipe;								
L5	Max flow in the pipe;								
L6	Max degree of pipe fill;								
L7	Rate beetwin Qmax and Q Pipe full flow design;								
L8	Flow – Accumulated;								
L9	Time of Max level Occurrence;								
L10	Time of Max flow Occurrence.								

The following table compares the Report files of all the three UDM’s defining when it is possible to evaluate the hydraulic parameters required by the CARE-S tasks

Node data Results from report file + input file				
Entity	Task	InfoWorks	Mouse	SWMM
Single Event				
Node Reference	All interested	Y	Y	Y
Ground level	All interested	Y	Y	Y
Invert level	wp3.4 wp5 wp6	Y	Y	Y
Max water level	wp3.4 wp5 wp6	Y	Y	Y
Spilled Volume	Wp3.3	N	Y	N
Long Term Simulations				
Node Reference	All interested	Y	Y	Y
Ground level	All interested	Y	Y	Y
Invert level	wp3.4 wp5 wp6	Y	Y	Y
Max water level	wp3.4 wp5 wp6	Y	Y	Y
CSOs Frequency	Wp3.3	N	N	N
Total Spilled Volume	Wp3.3	N	Y	N

Link data Results from report file + input file				
Entity	Task	InfoWorks	Mouse	SWMM
Single Event				

Link Reference	All interested	Y	Y	Y
Max velocity	All interested	Y	Y	Y
Min velocity	All interested	N	N	N
Max flow	All interested	Y	Y	Y
Max water level	wp3.4 wp5 wp6	Y	Y	Y
Invert level	wp3.4 wp5 wp6	Y	Y	Y
Long Term Simulations				
Link Reference	All interested	Y	Y	Y
Max velocity	All interested	Y	Y	Y
Min velocity	All interested	N	N	N
Max flow	All interested	Y	Y	Y
Max water level	wp3.4 wp5 wp6	Y	Y	Y
Invert level	wp3.4 wp5 wp6	Y	Y	Y

Output as BINARY SYSTEM FILE

The results written in binary system code can be copied on a ASCII file but this operation could not be done from external tools.

In the next pages example files exported from the binary system to the ASCII format are presented for each model with the distinction between results at node and link level.

Infoworks:

Export File extension: *.csv

At the node level is created a file for the Water Level (*_D.csv).

<i>Time series</i>					
<i>Time</i>	<i>Start Simulation</i>	<i>Node1</i>	<i>Node2</i>	<i>Node i</i>	<i>Node n</i>
Time	Seconds	243411.1	243430.1
01-08-00 00:00	0	0.03065	0.02000
01-08-00 00:15	900	0.03064	0.02000
.....
End simulation	End simulation

For the links are created three files with the same format (csv): one file for the depth (*_D.csv), one for the flow Q (*_Q.csv) and one for the velocity V (*_V.csv).

<i>Time series</i>					
<i>Time</i>	<i>Start Simulation</i>	<i>Link 1</i>	<i>Link 2</i>	<i>Link i</i>	<i>Link n</i>
Time	Seconds	243411.1	243430.1
01-08-00 00:00	0	0.03065	0.02000
01-08-00 00:15	900	0.03064	0.02000
.....
End simulation	End simulation

Mouse:

Export File extension: *.* (the user can chose the extension, anyway the file will be considered a text file)

The exported file could be only one with all the information included or more.
In the following pages are described two different files, one for the node and one for the links.

At the node level is created a file for the Water Level (*.out).

<i>Title file</i>	<i>*M11 CHAN</i>	<i>Nodo 1</i>	<i>Nodo 2</i>	<i>Nodo i</i>	<i>Nodo n</i>
	1 2 Time:	<>			
<i>Var. (WL) Element</i>	1 103 Node WL:	<242716>			
<i>Var. (WL) Element</i>	1 103 Node WL:	<242714>			
	<i>*M11 DATA</i>	"<242716>"	"<242714>"	"<i>"	"<n>"
<i>Timeo</i>	1994-01-01 00:00:00	212.511	210.8011
	1994-01-01 00:05:04	212.5134	210.8021
	<i>End simulation</i>

For the links are created one file for the Water level, one for the flow Q and one for the velocity V (*.out).

<i>Title file</i>	<i>*M11 CHAN</i>	<i>Link 1</i>	<i>Node start</i>	<i>Node end</i>	<i>Link 2</i>		
<i>Link 1</i>	1 2 Time:	<>					
<i>Var. (WL) Elem.</i>	1 100 Link WL	<242760>	242702	242547			
<i>Var. (Q) Elem.</i>	1 200 Link Q:		242702	242547			
<i>Var. (V) Elem.</i>	1 300 Link V:		242702	242547			
<i>Var. (WL) Elem.</i>	1 100 Link WL:		242702	242547			
<i>Link 2</i>							
<i>Var. (WL) Elem.</i>	1 100 Link WL	<242548>	242547	242472			
<i>Var. (Q) Elem.</i>	1 200 Link Q:	<242548>	242547	242472			
<i>Var. (V) Elem.</i>	1 300 Link V:	<242548>	242547	242472			
<i>Var. (WL) Elem.</i>	1 100 Link WL:	<242548>	242547	242472			
		<i>WL</i>	<i>Q</i>	<i>V</i>	<i>WL</i>	<i>Q</i>
	<i>*M11 DATA</i>	<242760>	<242760>	<242760>	<242760>	<242548>
<i>Time</i>	1994-01-01 00:00:00	209.5411	0.0000	0.1100	208.7911
	1994-01-01 00:05:04	209.5426	0.0000	0.0517	208.8000.
	<i>End simulation</i>

SWMM:

Export File Extension: *.txt

While for MOUSE and InfoWork it is possible to create files with hydraulic parameters for more than one location (link), the SWMM model allows to create text files with all the information but only for one link or Node.

Node data Results							
Table - Node 245071							
	Days	Hours	Depth	Head	Volume	Inflow	Overflow
	N1	N2	N3	N4	N5	N6	N7
N1	Days;						
N2	Hours;						
N3	Depth (Head relative to bottom);						
N4	Head (absolute);						
N5	Volume in the node;						
N6	Inflow to node;						
N7	Overflow.						

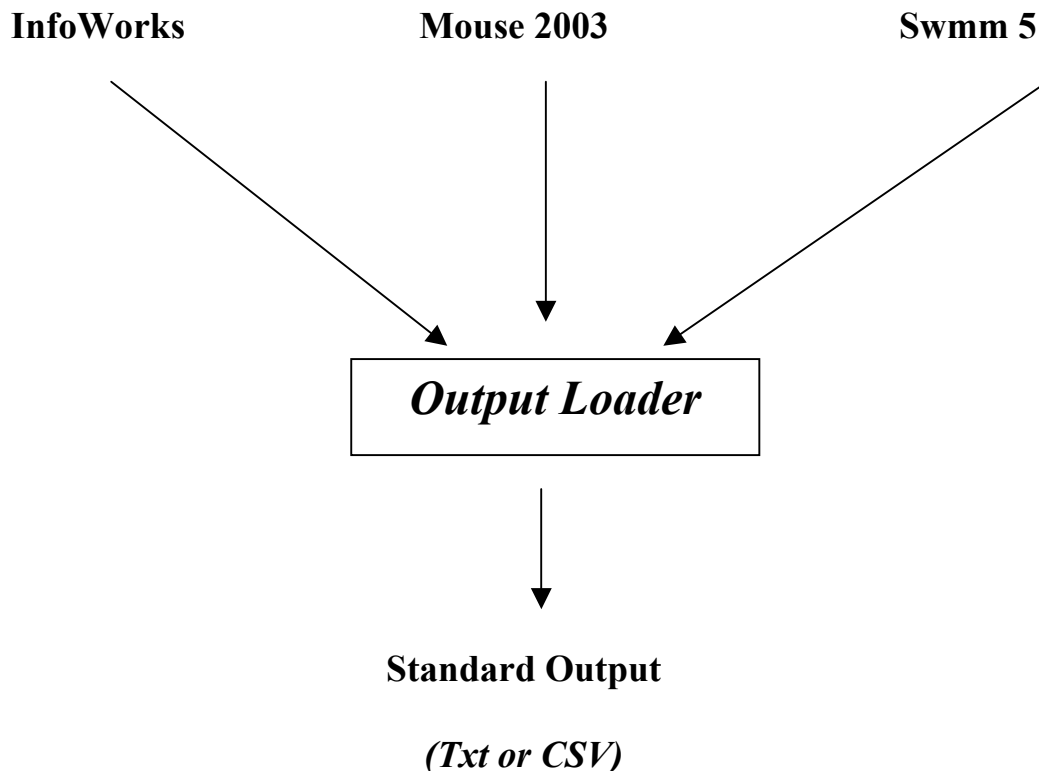
Link data Results						
Table - Link DES_4.1						
	Days	Hours	Depth	Flow	Velocity	Capacity
	N1	N2	N3	N4	N5	N6
N1	Days;					
N2	Hours;					
N3	Depth (Head relative to bottom);					
N4	Flow;					
N5	Velocity;					
N6	Max full flow;					

The report files (ASCII or htm) coming from the three models have different format. WP3.2 considered useful the creation of a tool able to analyse all the output data in order to make hydraulic results available for the other CARE-S tasks /WPs in a standard format which is independent from the specific UDM used by the end user.

This tool or subroutine is written in Visual Basic 6.0.

The final output is a text file.

3.2.2.1 Text Output from (hydraulic simulations)



The “Standard output” file has to be generated for each simulation, and for each simulation it resumes the data required by the others WPs/tasks.

The subroutine uses as input file the report files of the models and if necessary (SWMM and MOUSE) the file with the network information.

The files involved on this translation are:

for Infoworks:

*.prn

for Mouse:

*.htm for the simulation

*.txt for the network

for Swmm:

*.prt for the simulation

*.inp for the network

The “standard output” file is in ASCII format, and it will include the following information:

<i>N1</i>	<i>N2</i>	<i>N3</i>	<i>N4</i>	<i>N5</i>
Node Reference	Ground Level	Max Level	Max Depth	Flood Index
	(m AD)	(m AD)	(m)	(%)
<241681>	190.670	188.316	0.001	0.
.....

N1	Node reference ;
N2	Ground level ;
N3	Max Level nel nodo;
N4	Max depth (Max Level - Bottom Level)
N5	Max depth /(Ground level – Bottom Level)

<i>L1</i>	<i>L2</i>	<i>L3</i>	<i>L4</i>	<i>L5</i>	<i>L6</i>	<i>L7</i>	<i>L8</i>	<i>L9</i>	<i>L10</i>
Link Reference	US Node	DS Node	Pipe Dpth	UPInvert Level	DWInvert Level	Max Depth	Max Vel	Max Flow	Max h/D
			(mm)	(m AD)	(m AD)	(m)	(m/s)	(m3/s)	
<241682.1>	241682	241681	230	192.620	188.290	0.0260	0.2215	0.0010	0.11
.....
L1	Link reference;								
L2	Up-stream node reference;								
L3	Down-stream node reference;								
L4	Pipe height								
L5	Up-stream invert level;								
L6	Down-stream invert level;								
L7	Max depth in the pipe;								
L8	Max velocity in the pipe;								
L9	Max flow in the pipe;								
L10	Max ratio of Max depth on the height of pipe.								

G1	
Total outflow =	m3

G1	Total outflow at the outlet section
----	-------------------------------------

Example of a standard output file:

Node Reference	Ground Level (m AD)	Max Level (m AD)	Max Depth (m)	Flood Index (%)
<241681>	190.670	188.316	0.001	0.00
<241682>	195.010	192.646	0.026	1.01
<241683>	198.670	196.386	0.026	1.01
<241684>	202.530	200.315	0.025	1.01
<DES_4>	131.880	128.588	0.074	2.02
<DES_5>	131.000	128.285	0.066	2.04

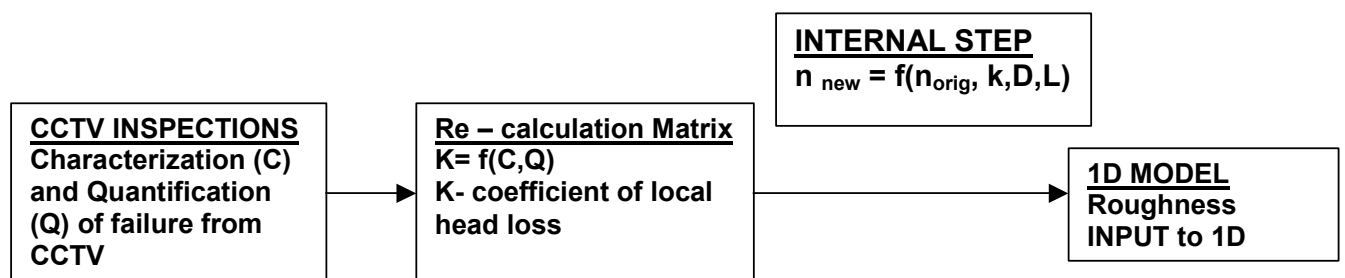
Link Reference	US Node	DS Node	Pipe Dpth (mm)	UPInvert Level (m AD)	DWInvert Level (m AD)	Max Depth (m)	Max Vel (m/s)	Max Flow (m3/s)	Max h/D
<241682.1>	241682	241681	230	192.620	188.290	0,180555556	1,538194444	0.0010	0.11
<DES_2.1>	DES_2	DES_5	110	129.000	128.200	0,364583333	0.0015	0.0000	0.48
<DES_3.1>	DES_3	DES_4	110	129.300	128.500	0,375	0.0015	0.0000	0.49
<DES_4.1>	DES_4	DES_5	300	128.500	128.200	0,552083333	6,552083333	0,097222222	0.27
<DES_5.1>	DES_5	250551	300	128.200	127.970	0,444444444	12.815	0,097222222	0.21

Total outflow = 1459.8 mc

3.2.2.2 Modeling temporal decline

In chapter 2.2.2.2 and Appendix A4 the results of 3D simulations to model temporal decline are described. Results are given as Formulae for the calculation of the K – coefficient of local headloss for each kind of failure analyzed. Those formulae are grouped in the “Re – calculation Matrix” which is an Excel file. Formulae are calculated using CCTV data as input. If the commercial model used by the end – user is unable to introduce a new K coefficient, without changing the network topology, the local headloss can be converted in a distributed head loss applying a new roughness coefficient (“ n ” coefficient defined by Manning) (see formula (6), chapter 6.4.1).

The connection from the CCTV inspection data, to the Re – calculation Matrix and roughness values to the 1D models as presented in the following picture, should be done automatically, but if not possible, depending on the complexity of the problem for the Rehab Manager structure, it can be done manually.



REFERENCES

1. Claudepierre, C. (2003) Hydraulic, structural, operational and risk investigation in sewer rehabilitation planning–The Nordberg case study in Oslo, Norway. Master theses. Department of Hydraulic and Environmental engineering, Norwegian University of Science and technology, Trondheim; SINTEF, Trondheim; INSA de Lyon, France.
2. Developing of guidelines for the structural, hydraulic and environmental rehabilitation of sewers. EU Project nr. MAS0303. <http://www.hi.ihe.nl/srguide/>
3. DHI
MOUSE User Guide
MOUSE Pipe Design User Guide
MOUSE RDII User Guide
MOUSE LTS User Guide
MOUSE GM User Guide and Tutorial
MIKE View User Guide
MOUSE Reference Manual (Pipe Flow)
MOUSE Reference Manual (Runoff)
MOUSE RDII Reference Manual
DHIAPP INI and ADP Files Reference Manual
UND File Specification
HGF File Specification
4. European Code of Practice EN752.
5. ISBN 87-7511-974-9 Rehabilitation of sewer systems, guidelines for choice of method, design and installation. Norway.
6. Milina, J., Köning, A., Selseth, I., Schilling, W. (2003) RISURSIM – Risk Management for Urban Drainage Systems – Simulation and Optimisation. EUREKA project. SINTEF Applied Chemistry. STF66 A03011.
7. Water and Sewage Works Department (VAV) in Oslo municipality. Local design standards and codes of practice.
8. Yves Le Gat “*Degradation Models for Drain and Sewer Pipelines Parameterised with CCTV Inspection Data*”
9. Jaroslav Pollert “*The Hydraulic Capacity of Deteriorating Sewer Systems*”, final report.
- 10 InfoWorks User Guide
11. G. Freni "Catchment-scale efficiency evaluation of distributed stormwater management practices"- Ph.D Dissertation Thesis (XIVth Cycle) – Università "Federico II" di Napoli - Tutors: Prof. Ing. Goffredo La Loggia, Prof. Ing. Giacomo Rasulo - Coordinator: Prof. Ing. Carlo Montuosi.
12. Valente Stefania, "*Studio del comportamento idraulico e qualitativo della rete fognaria di Correggio (RE) tramite il codice di calcolo InfoWorks*", Thesis, DISTART- Bologna University - Italy, In italian. 2003.

4 APPENDICES

4.1 A1 The Nordberg catchment case study#1

The municipality of Oslo in Norway initiated extensive sewer system rehabilitation program in the late 1990's. The large number of measurements, studies and analyses have been performed in order to describe the sewer network's functionality. CCTV equipment has been used to gather data necessary to analyse true pipe condition and defects, while MOUSE modules have been used to calculate capacity and flooding problems, Figure A1. Data has been analysed with the help of GIS software.

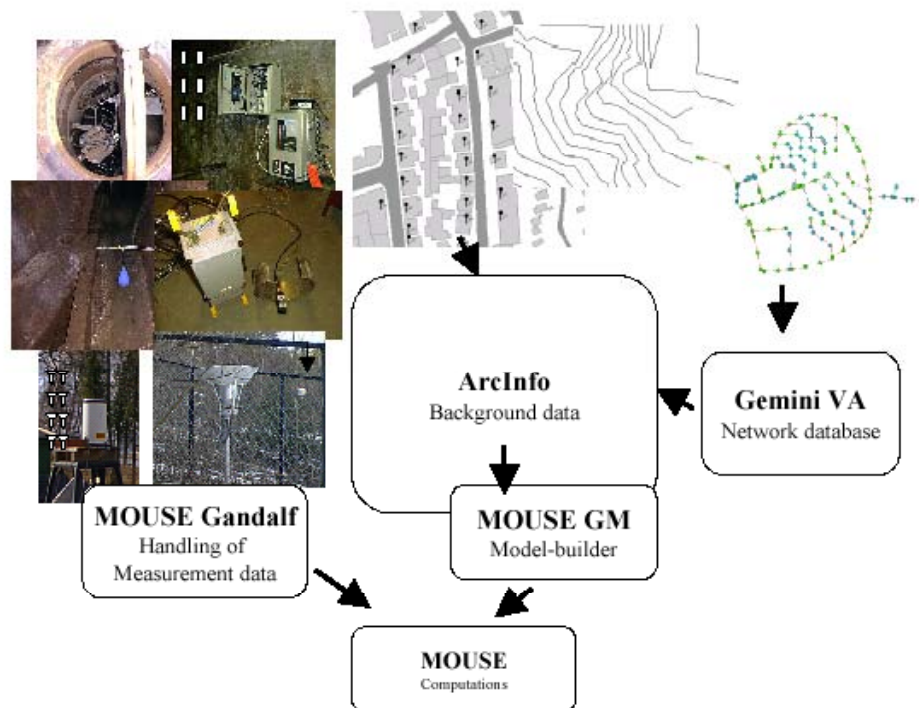


Figure A1 The main 'pillars' of the sewer rehabilitation process in Oslo, Norway

For the needs of CARE-S WP3.2 task, a MOUSE model of the northern part of the Nordberg catchment area in Oslo was provided by the Water and Sewage Works department (VAV) in Oslo municipality. The Nordberg catchment area is encircled on the Figure A2.

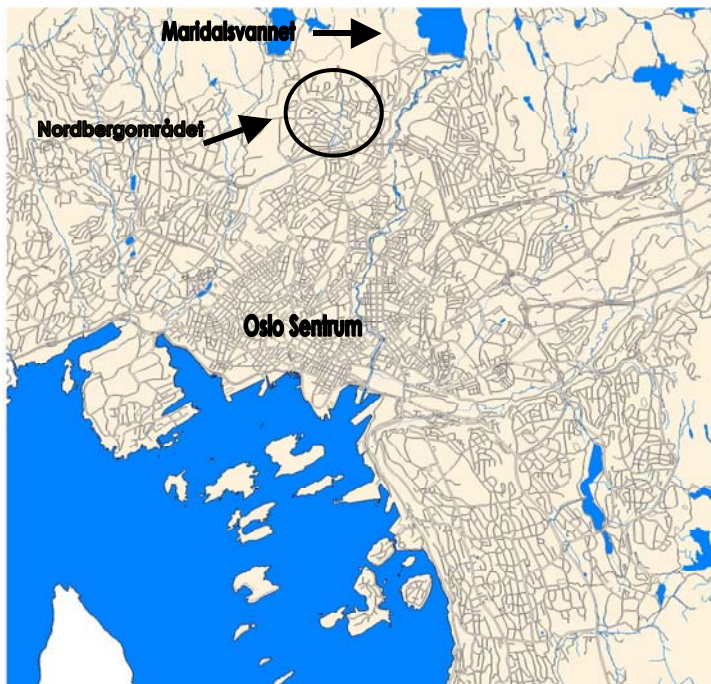


Figure A2 Location of the Nordberg catchment area in Oslo, Norway.

The Nordberg case study covers a residential area of 125 ha with 4813 inhabitants.

Catchment and sewer system data and mouse model building

The importance of GIS in urban drainage management which relies very heavily on a huge amount of data has increased in the past years. In the late 1980's – early 1990's, the Norwegian municipalities mapping departments began constructing a new set of digital baseline maps. The availability of these data enabled other departments to begin using digital maps and GIS for their own work. Each of the departments chose a software package that best fit their needs for constructing and maintaining geo-data.

In Oslo municipality ArcInfo and ArcView installation was established to allow sharing of data between departments and applications. The Water and Sewage Works Department (VAV) decided to build an Arcview GIS application in order to support and enhance the process of building a complete MOUSE model for the city. The development of this software named GM (an abbreviation for GIS Model Management) started in 1996 by VAV, the company ROSIM AS and the Norwegian ESRI distributor GEODATA AS. Later on the joint effort of the mentioned parties and DHI Water and Environment resulted in MOUSE GM as a commercial software package.

MOUSE GM is developed to support and enhance the process of building a model of a sewershed and its sewer network and to export the model to MOUSE. Besides, after the simulation, results from MOUSE can be displayed and thus complete the model with new information.

Figure A3 shows the structure of MOUSE GM folders that include all data on catchment and sewer system in the Nordberg case study.

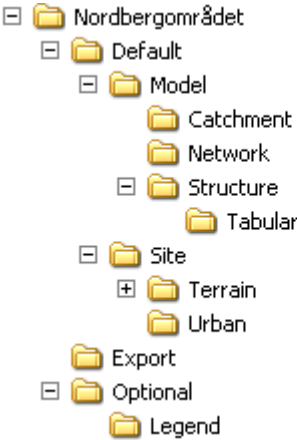


Figure A3 Folder structure in MOUSE GM.

The **DEFAULT** folder contains sub-folder, *.SHP and *.DBF files with structure and names which must not be changed. The sub-folders under DEFAULT/MODEL, i.e. CATCHMENT, NETWORK and STRUCTURE, contain the *.SHP files that together represent the complete physical network and catchment data in MOUSE. Table A1 shows relation between the sub-folders and the predefined *.SHP file names.

Table A1 Predefined shape files in the DEFAULT folder.

Site	Grid	Grid data				
Urban	Shape	Building	Population	Road		
Catchment	Shape	Combined	Separate	Storm	Gully_trap	HgF*
Network	Shape	Node	Pipe			
Structure	Shape	Basin	Outlet	Pump	Weir	

* The HGF.SHP in the CATCHMNET folder is only present if the user has created an "Hgf" table in the catchment mode.

To store MOUSE tabular data (cross section, basin geometries, weir curves, pump capacity, etc.), MOUSE GM generates *.DBF files with predefined file names as needed. These *.DBF files are stored in the TABULAR folder, a sub-folder to the STRUCTURE folder. Table A2 shows the predefined table names for DBF files used to store MOUSE tabular data and cross-section data. The tables are generated with a serial number as the _xx indicates. The LINK.DBF file provides a link between the data set ID and the MOUSE GM table name.

Table A2 Tabular data.

Tabular data	GM table name	Cross section data	GM table name
Capacity Curve QH	CapQH_xx	H,W closed	HWclo_xx
Capacity Curve QdH	CapdH_xx	H,W open	HW_ope_xx
Pump acceleration curve	PACur_xx	X,Z closed	XZ_clo_xx

Basin geometry	BasGe_xx	Link file	link.dbf
.....			

In addition to the listed *.DBF files a file with the predefined name RDII_PARAM.DBF is saved in the CATCHMENT folder and used to store the MOUSE -> RDII data sets.

The PROJECT.INI file is saved in the MODEL folder. The INI file contains information from the project creation like MOUSE GM version and units as well as optional settings last time the project was closed.

Population, buildings, roads and terrain levels which belong to data that typically can be found in public databases and GIS systems can be represented in MOUSE GM by the SITE themes. Population is a theme with a point shape. The table should contain a filed A, type number, with numbers that correspond to number of persons at each point. Building and road are used as polygon shapes, no specific fields are needed. The terrain theme should be a grid theme.

The EXPORT folder contains files for the data exchange, that is:

- *.UND, *.HGF and *.MEX files for import from and export to MOUSE
- *.BMW/BMP files for the background image in MOUSE and MIKE View
- *.CLA file for opening of MIKE View.

It is practical to use the EXPORT folder as the directory for the appurtenant MOUSE project and MIKE View files.

The OPTIONAL folder contains two types of files:

- Optional themes saved by the user
- Files generated by MOUSE GM

In the LEGEND folder (the sub-folder of the OPTIONAL folder), the legends for the default project themes are stored with the same name as the appurtenant project theme i.e. BASIN.AVL, BUILDING.AVL, COMBINED.AVL, MANHOLE.AVL, PIPE.AVL, POPULATION.AVL etc.

4.1.1 MOUSE GM tool - The Nordberg case study

Catchment

The main functions of MOUSE GM tool CATCHMENT enable catchment delination and calculation of catchment parameters. Three catchment types/themes are included: Combined theme, Separate theme and the Storm theme, Table A3.

Table A3 The three catchment type included into the CATCHMENT tool and type of data exported to MOUSE.

	Area of buildings and roads	Number of persons
Combined	x	x

Storm	x	
Separate		x

The three catchment types differ according to the type of site data calculated and exported to MOUSE for each catchment type. It can be observed that for storm catchments only the impervious area is calculated (surface runoff), while for the separate catchment only the population data area include (wastewater flow). Combined catchments include both types of data.

Catchment table consists of a number of fields with one row for each catchment polygon, Table A4. Some of the fields are not intuitive and require explanation.

Table A4 The Catchment table.

ID	Catchment	IMPER_PCT	9.870	CATCH_LEN	0
M_ID	900002	PER_AREA	0.1550	DES_BUI	0
BUILD_AREA	0.0113	PER_PCT	90.116	DES_BUIPCT	0.0000
BUILD_PCT	6.5753	PER_FACT	1.00	DES_ROA	0
BUILD_FACT	1.00	TOT_AREA	0.172	DES_ROADPCT	0.0000
ROAD_AREA	0.0057	PE_HA	116.246	DES_PE	0
ROAD_PCT	3.2946	CATCH_TYPE	Combined	DES_PE_HA	0
ROAD_FACT	1.00	ADD_FLOW	0.000	DES_PERV	0
IMPERV_TOT	0.0170	SLOPE	0	DES_PERPCT	0.0000

The file M_ID is the name of the connected manhole. IMPER_PCT is the impervious area (%) exported to MOUSE surface runoff model A. BUILD_AREA is the area of building that is included in the calculation of IMPER_PCT. Likewise, BUILD_PCT is the percentage of buildings inside the polygon as included in the calculation of IMPER_PCT.

BUILD_AREA, BUILD_PCT and IMPER_PCT area values that MOUSE GM calculates. The BUILD_FACT is a user-specified factor (default value 1), which is multiplied with the actual building area for the calculation of the above mentioned fields. The BUILD_FACT is therefore the factor to use for calibration purposes. For example, when measures of flow have indicated that only 50% of the building area contributes to the flow, the BUILD_FACT would be adjusted to 0.5 and the calculation of catchments performed again. The same principle as described above applies for ROAD_AREA and PER_AREA.

The PER_PCT is calculated as the total area minus the calculated impervious area and is exported to the 'RDII Area' field in the MOUSE catchment dialog.

The PE_HA field is the population density in persons/hectares for the SI units and persons/acre for the US unit version. Based on this and the total area of the catchment polygon, number of inhabitants is calculated when writing the MOUSE text files.

HGF Shape

Opposite to the catchment shapes, the HGF shape includes all the parameters which constitute the MOUSE *.HGF file and all the data can be edited with standard ArcView functionality.

A HGF.SHP file can be created from two sources:

- 1 Import from MOUSE *.HGF file
- 2 From Catchment drawn in MOUSE GM

When imported from a MOUSE *.HGF file, there is no shape description and the HGF catchment polygons are drawn as squares with size illustrating the size of the area of the catchment and located with the connected manhole in the center of the square.

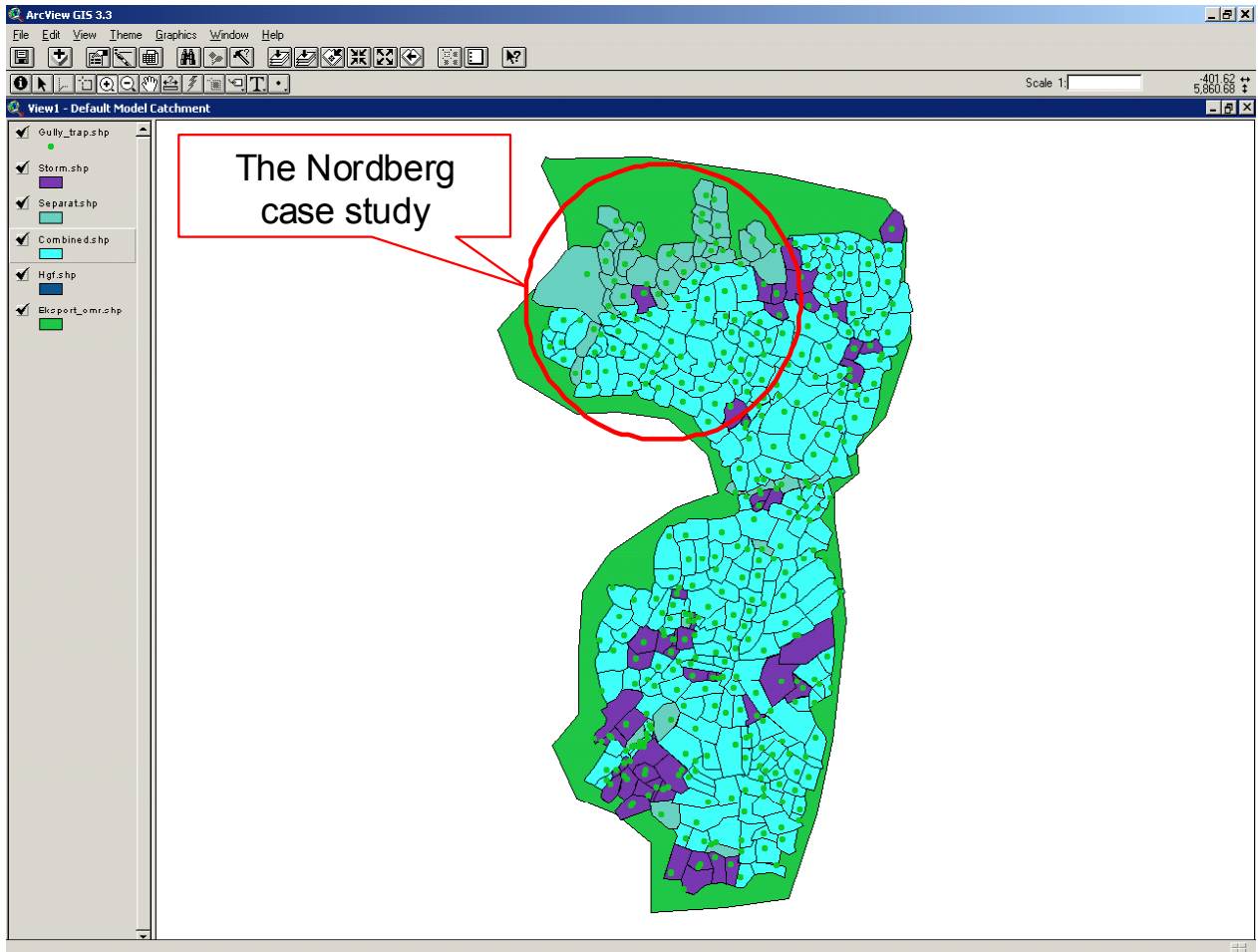


Figure A4 The Nordberg case study. MOUSE GM View – Default/Model/Catchment.

Catchment	Combined	Separate	Storm	Gully_trap	HgF
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The catchment area of the Nordberg case study is divided into 91 sub-catchments. Each sub-catchment is connected to a node in MOUSE model and defined by its coordinates, area, number of inhabitants, percentage of impervious area, additional flow, and a hydrologic parameter set for the runoff computation using MOUSE Surface Runoff module A and Rainfall Dependent Inflow& Infiltration module (RDII). The combined sewer system covers 73 ha, the separated sewer system 50,5 ha.

Network/Structure

The main functions of MOUSE GM tools NETWORK and STRUCTURE enable representation of the complete physical sewer system in MOUSE.

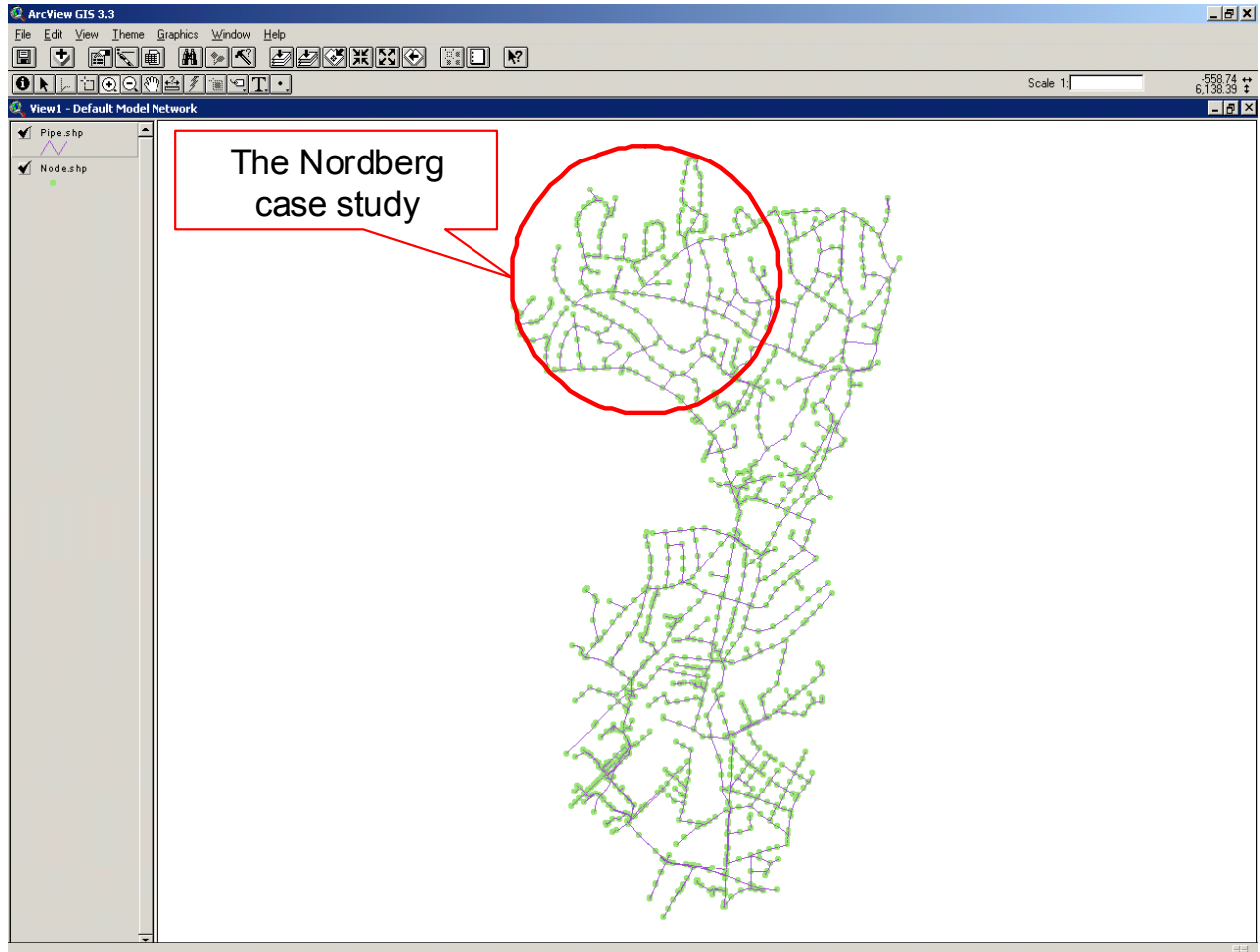


Figure A5 The Nordberg case study. MOUSE GM View – Default/Model/Network.

Network	Node	Pipe
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The Norwegian GEMINI database for storm and wastewater collection systems was used to develop the Nordberg sewer model. The network is constituted by 284 nodes among which one overflow, one outlet and 282 manholes. The storm water and wastewater is transported under gravity to the sewershed outlet. Manholes are defined by their coordinates, invert level, top level and diameter; whereas the outlets are defined only by their coordinates, invert level but also water level. Between these nodes, 295 links or pipes are defined by their starting and ending nodes, their material (concrete), their cross section (circular), their invert level and their diameter.

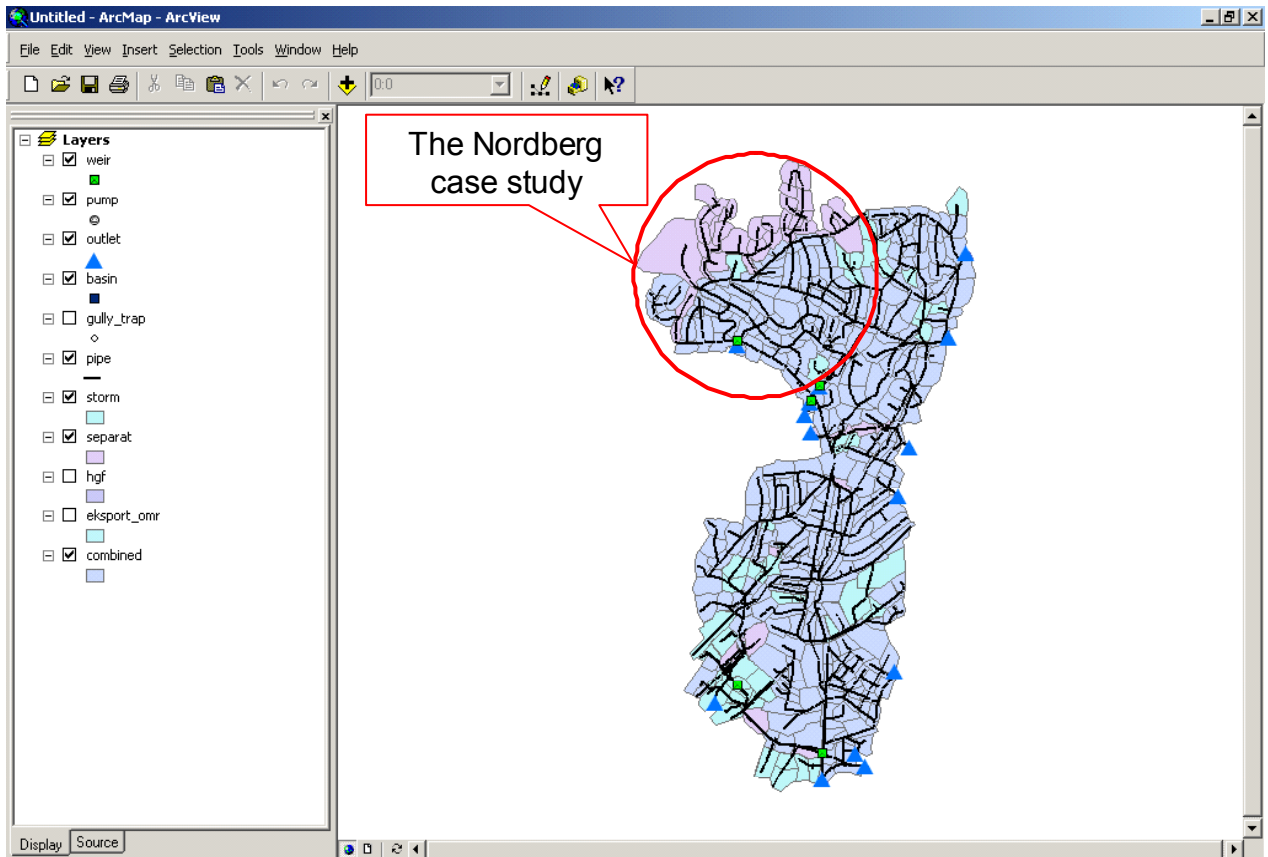


Figure A6 The Nordberg case study. MOUSE GM View – Default/Model/Structure.

Structure	Basin	Outlet	Pump	Weir
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A weir associated with an outlet diverts water away from the sewer network, when the capacity of the network is exceeded.

Site themes

Population, buildings, roads and terrain levels belong to data that typically can be found in public databases and GIS systems. This type of data can be represented in MOUSE GM by the SITE themes, with reserved names 'Population', 'Building', 'Road' and 'Terrain'.

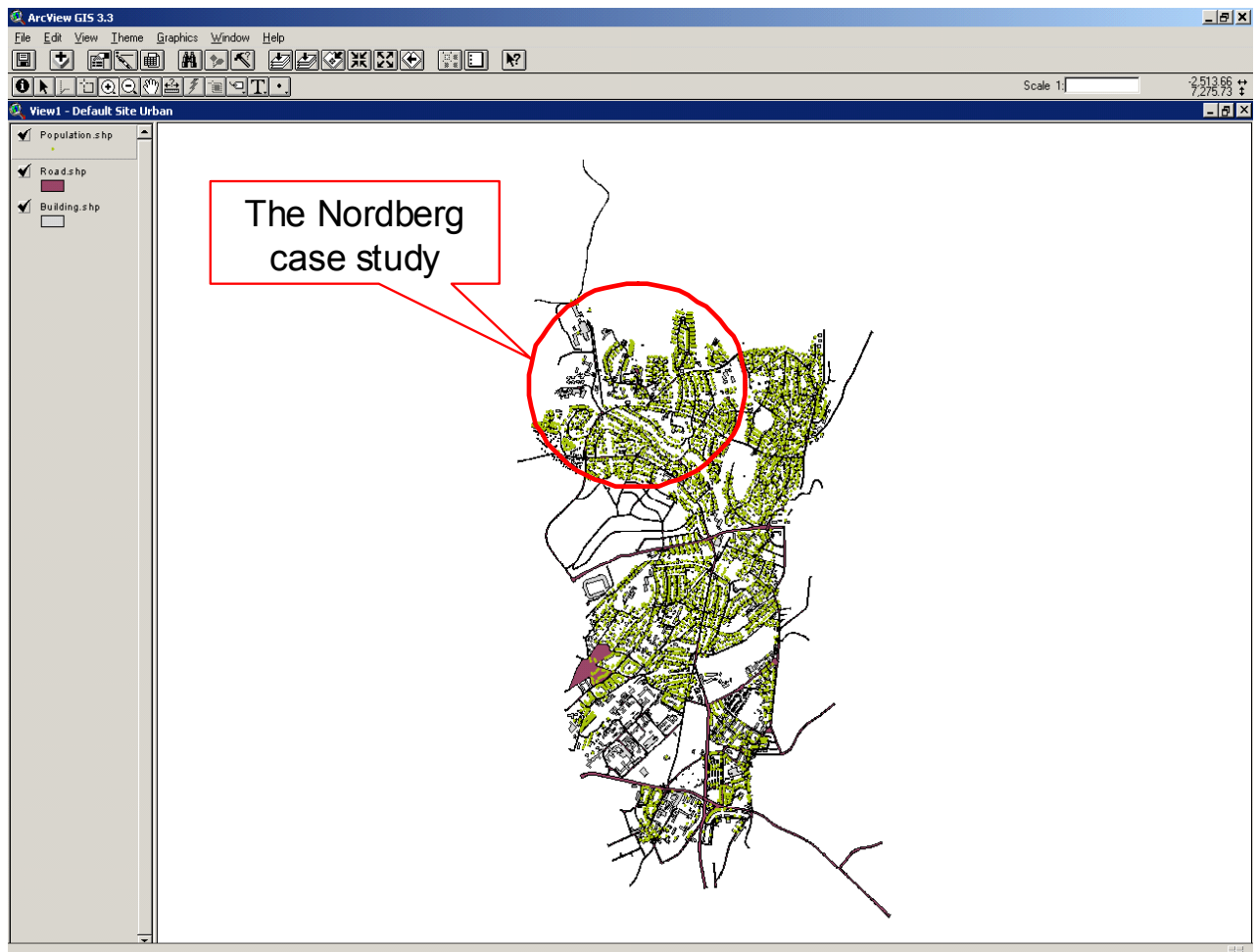


Figure A7 The Nordberg case study. MOUSE GM View – Default/Site/Urban.

Urban	Building	Population	Road
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Results

It is possible to open MIKE View (MOUSE result presentation program) from within MOUSE GM. MIKE View opens the same horizontal plan extent as the current View extent in MOUSE GM, and with a background image representing the themes currently displayed in the MOUSE GM View.

If the network in MOUSE GM is the same network that is opened in MIKE View, results can be copied from MIKE View to the clipboard and from the clipboard imported to MOUSE GM.

4.1.2 Process data and model calibration

Due to the complexity of precipitation-runoff transformation in general and the importance of rainfall/snowmelt induced infiltration into sewer systems from pervious surfaces in particular, the conceptual modelling of the hydrological cycle is frequently used in cold climate urban catchments. In the Nordberg case study MOUSE RDII model and long-term rainfall, temperature and evaporation records were used to generate inflow and infiltration to entry

points in the sewer network. Figure A8 shows the long-term records provided by the Blindern hydrometeorological station in Oslo.

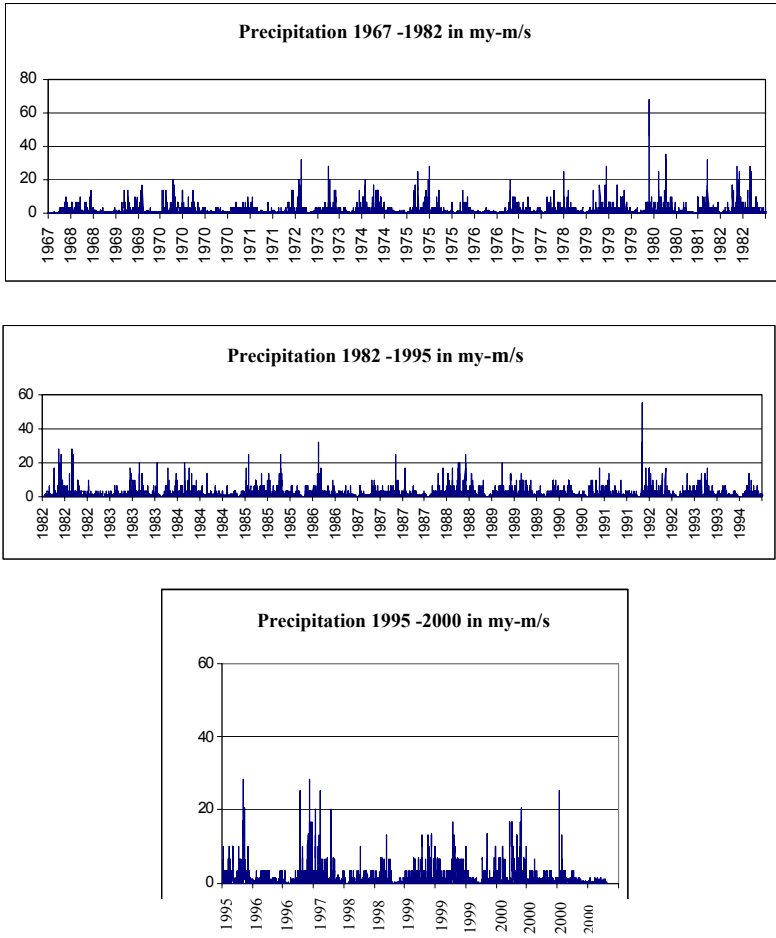


Figure A8 Precipitation data from 1967 to 2000. Rain data is obtained from a tipping bucket gauge which records time to tip with a depth resolution of 0.2 mm per tip. The unit used in MOUSE is my-m/s.

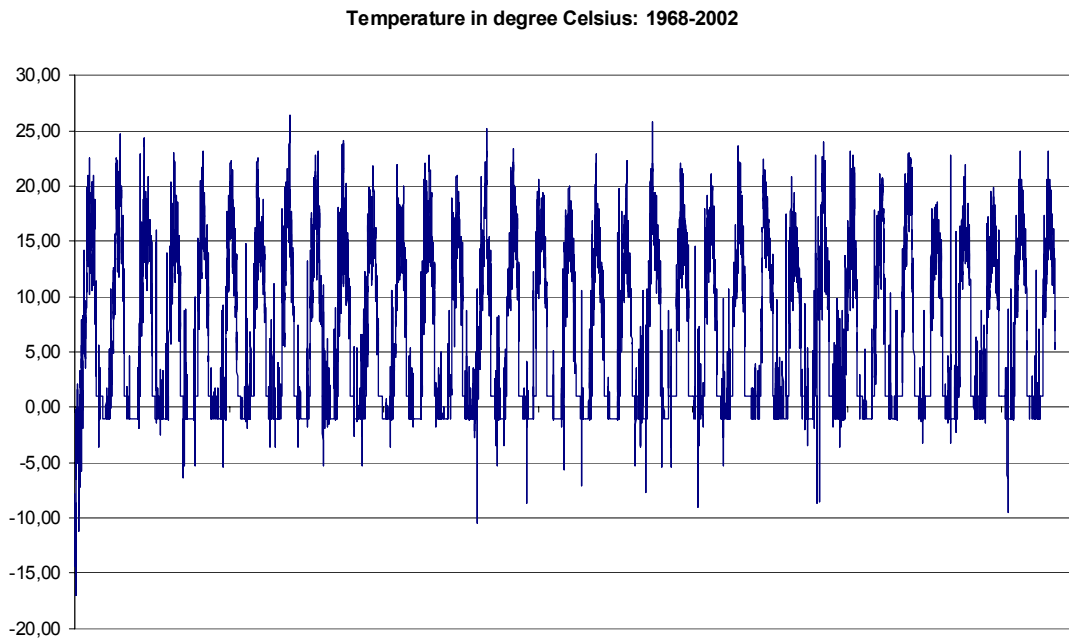


Figure A9 Temperature data from 1968 to 2002.

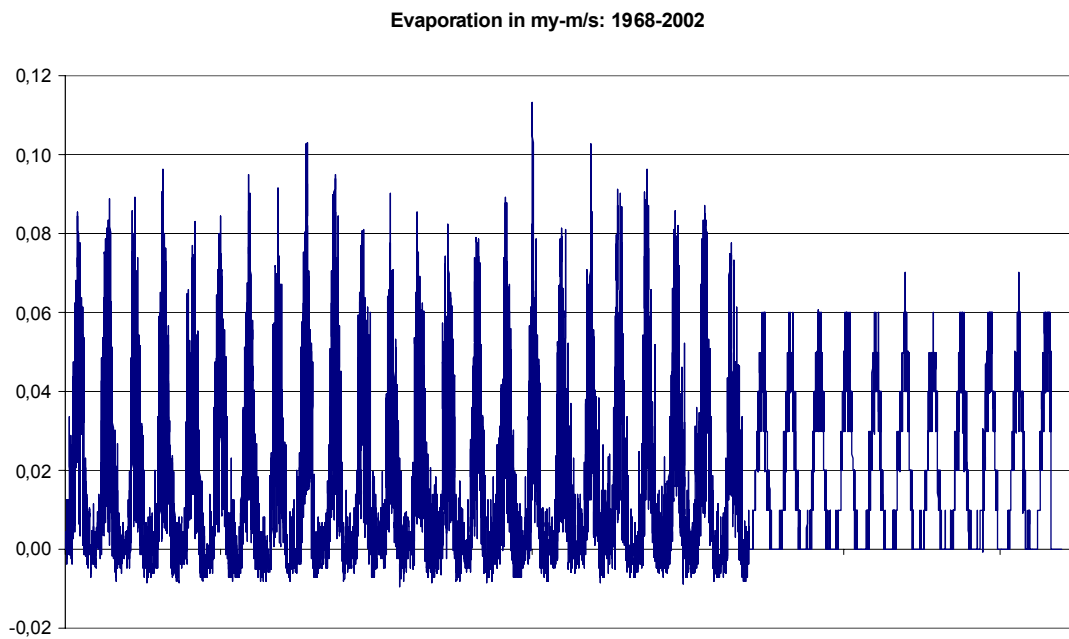


Figure A10 Evaporation data from 1968 to 2002.

The repetitive profile for the foul flow in the Nordberg case study is shown in Figure A11.

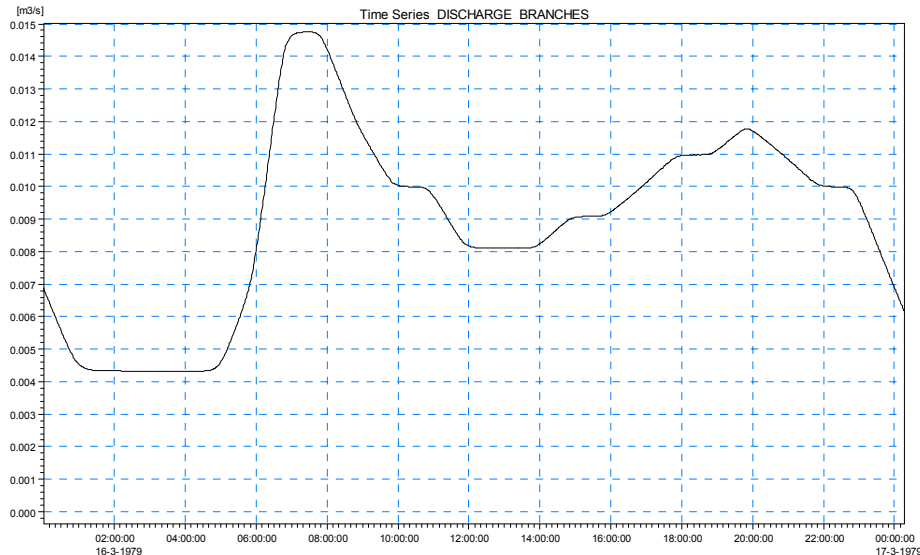


Figure A11 Repetitive profile which reproduces the variations of waste water discharge during the day (quantification of dry weather loading $0.1356 \text{ m}^3/\text{PE}/\text{Day}$).

4.1.3 Model calibration

Oslo municipality uses long term measurements and campaign measurements for the calibration of MOUSE models. The long-term measurements of precipitation, temperature and evaporation have been carried out at 10 stations. In addition, on-site measurements over 2-4 years periods have been carried out in order to refine the RDII parameters. The campaign measurements consist of measurements over a short period of time. These measurements are used for the purpose of MOUSE models calibration.

Surface Runoff Model A and RDII (Rain Dependent Inflow and Infiltration)

During the calibration procedure in the Nordberg case study three sets of Time/Area(A)-RDII parameters were used for three types of sewer system: one for separated (AkrF046DIVk038), one for combined (AkrF047VIFo), and one for storm water sewer system (AkrF048VIOo), Figure A12. The parameter sets for a catchment with combined and a catchment with storm water sewer system are identical.

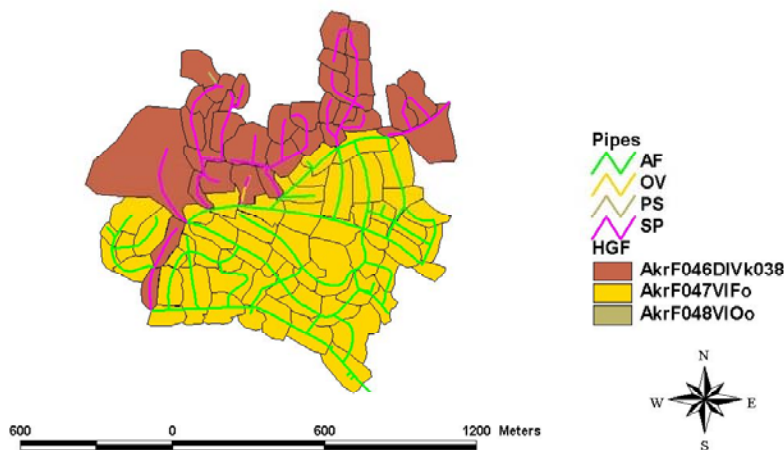


Figure A12 Three Time/Area(A)-RDII parameters set used in Nordberg case study.

The RDI parameters used for the catchments with separated sewer system and combined sewer system are presented below.

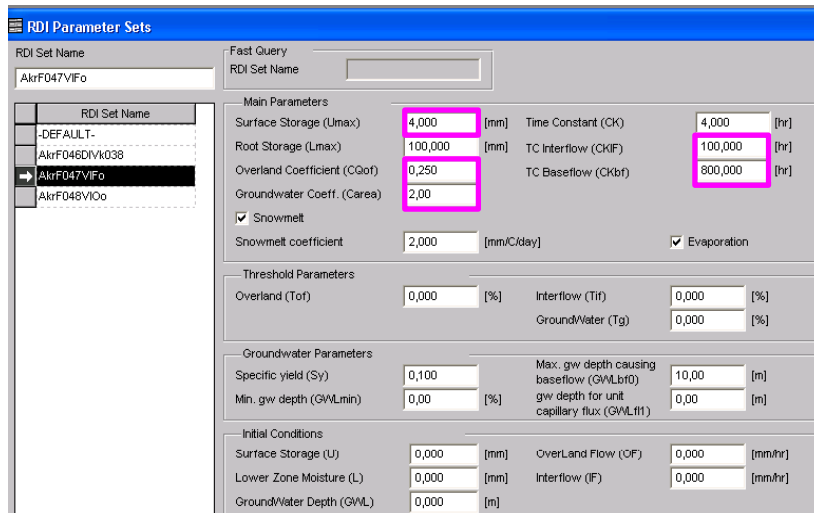


Figure A13 RDI parameters for separated sewer system.

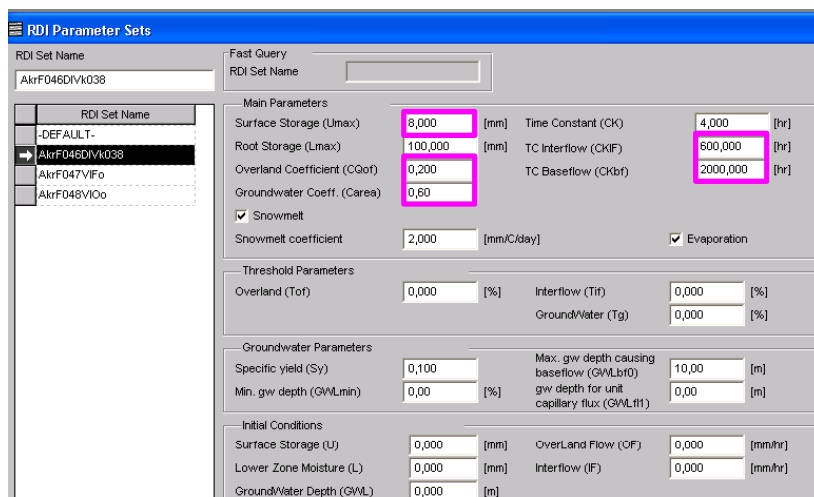


Figure A14 RDI parameters for combined sewer system.

When calculating with MOUSE RDII, time steps are given separately for the Surface Runoff Model and for the rain dependent infiltration part. In the Nordberg case study a time step of 300 seconds was used for calibration of the Fast Response Component (FRC). An RDII time step of 4 hours was chosen for calibration of the Slow Response Component (SRC).

Pipe flow

In the Nordberg case study the dynamic wave method of pipe flow computation was chosen. The computation was performed using runoff hydrographs (CRF file), dry weather flow (DWF file) and repetitive profile (RPF file). In the case of pipe flow computation, 8 seconds were specified for minimum and 30 seconds for maximum_time step. To describe the pipe friction, MOUSE default value for Manning's number of 85 (smooth concrete, $M=1/n$) was used for all pipes.

Calibration results

The MOUSE model of the Nordberg case study was calibrated against discharge measurements at the outflow from the catchment area (manhole 250551). The measurements

with five minutes resolution were taken during the period July – December 2000, Figure A15.

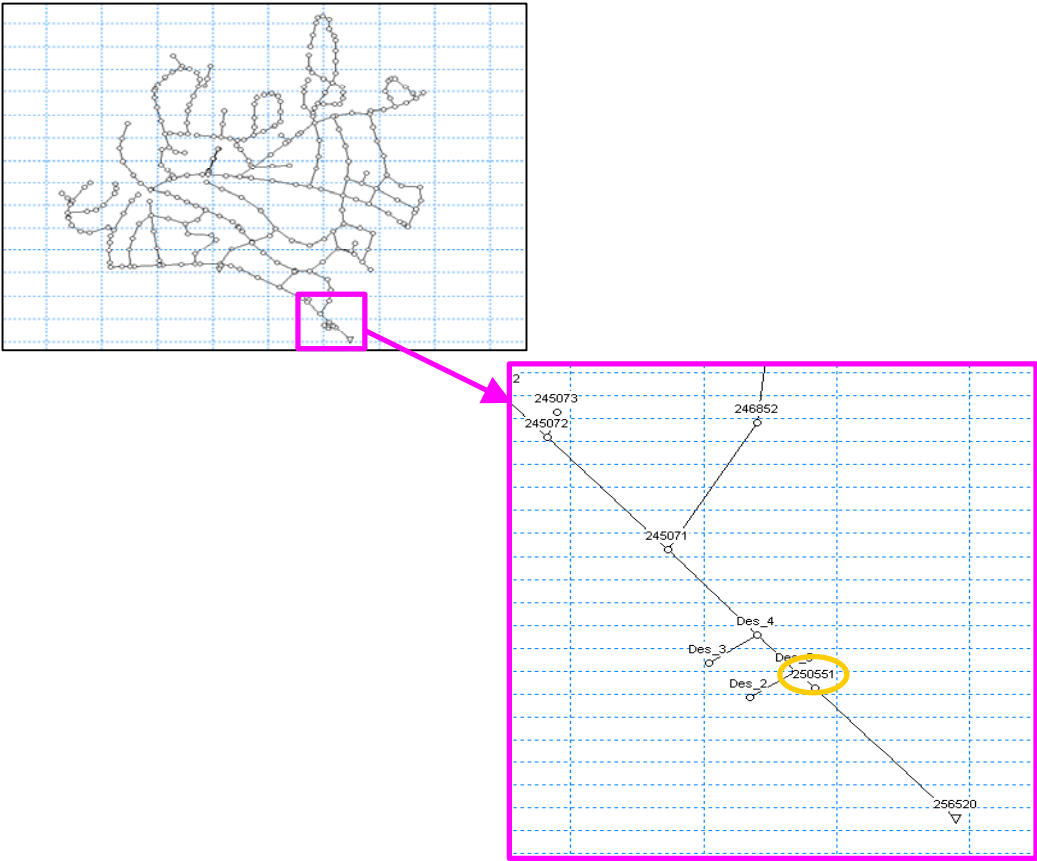
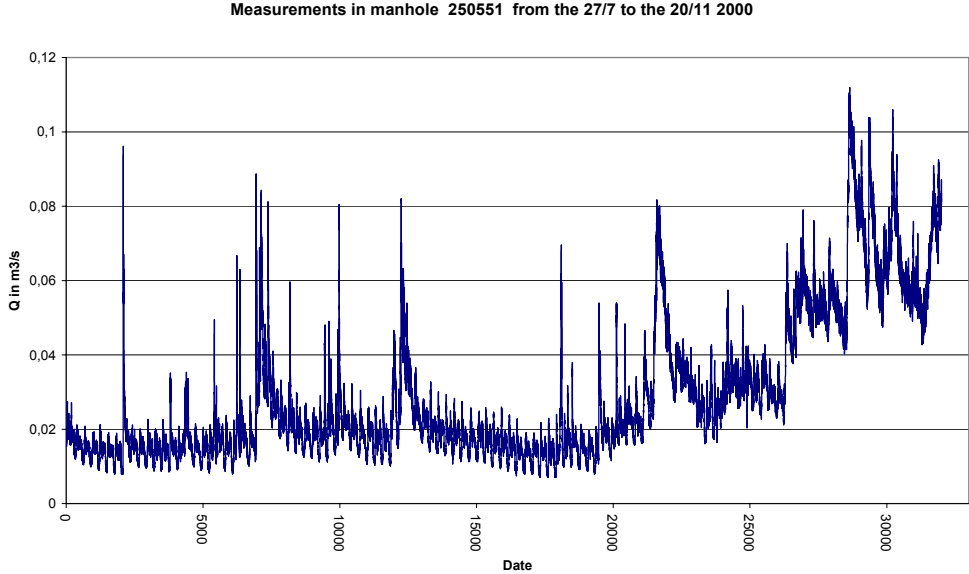
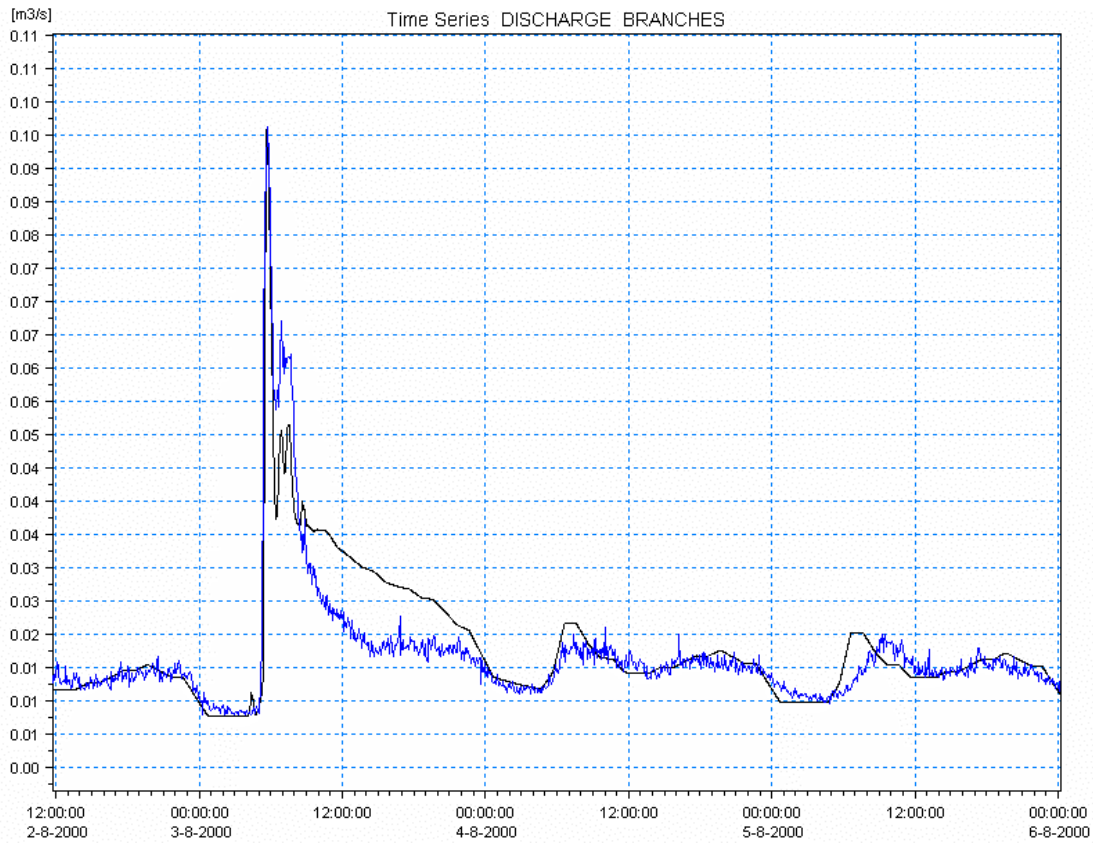
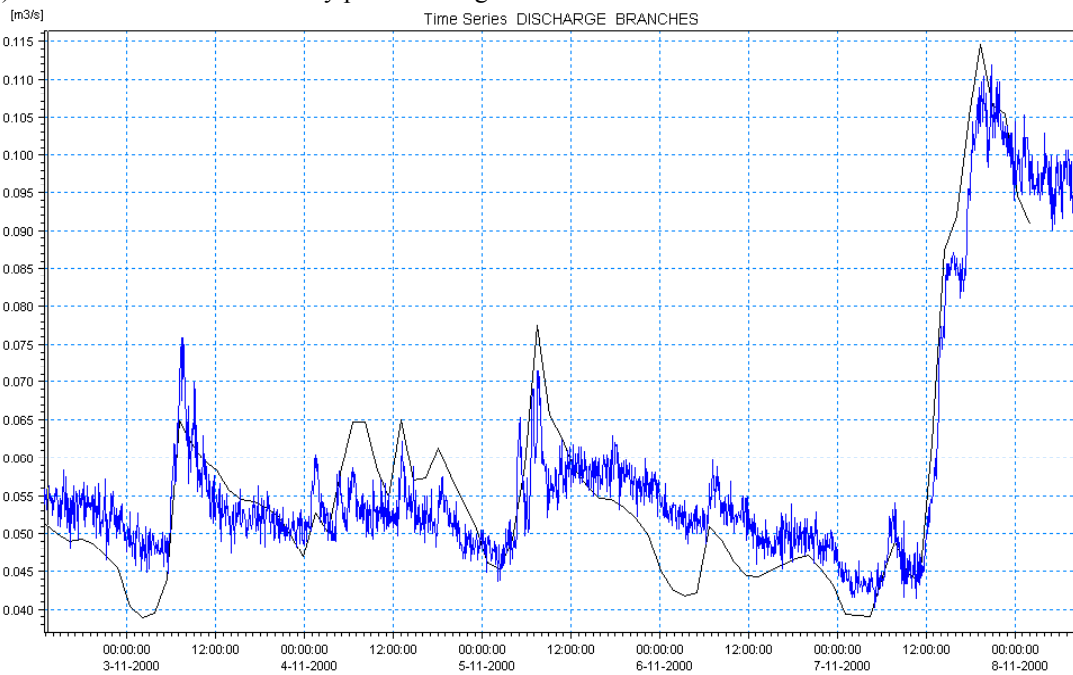


Figure A15 Discharge measurements in manhole 250551 used during the process of model calibration.

The results of model calibration are shown in Figure A16.



a) Calibration results for a 3-day period during the summer.



b) Calibration results for a 5-days period during the winter.

Figure A16 The results of model calibration.

4.1.4 Assessment of the hydraulic performance obtained from the simulation

The calibrated Nordberg model was used to assess the performance of the existing sewer network. In the Nordberg case study a 20-years period was first simulated using Time/Area (A) runoff and RDII infiltration model to assess relevant storm and snowmelt hydrographs. To assess extent and frequency of surcharging and surface flooding in whole sewer network extreme single short-term events from the 20-years period were simulated. The results of surface flooding statistics for all pipes and manholes are shown in Figure A17, A18 and A19.

In the Nordberg catchment no pipelines with 5-year frequency of surface flooding are identified. Surface flooding with 10-year return period is concentrated at the catchment's outlet. The pipelines with 20-year frequency of surface flooding are located in the central part and at the outlet of the catchment area.

The hydraulic performance criteria which relate to the frequency of surface flooding should be set with regard to the European Codes of Practice (EN752) and local design standards and codes of practice.

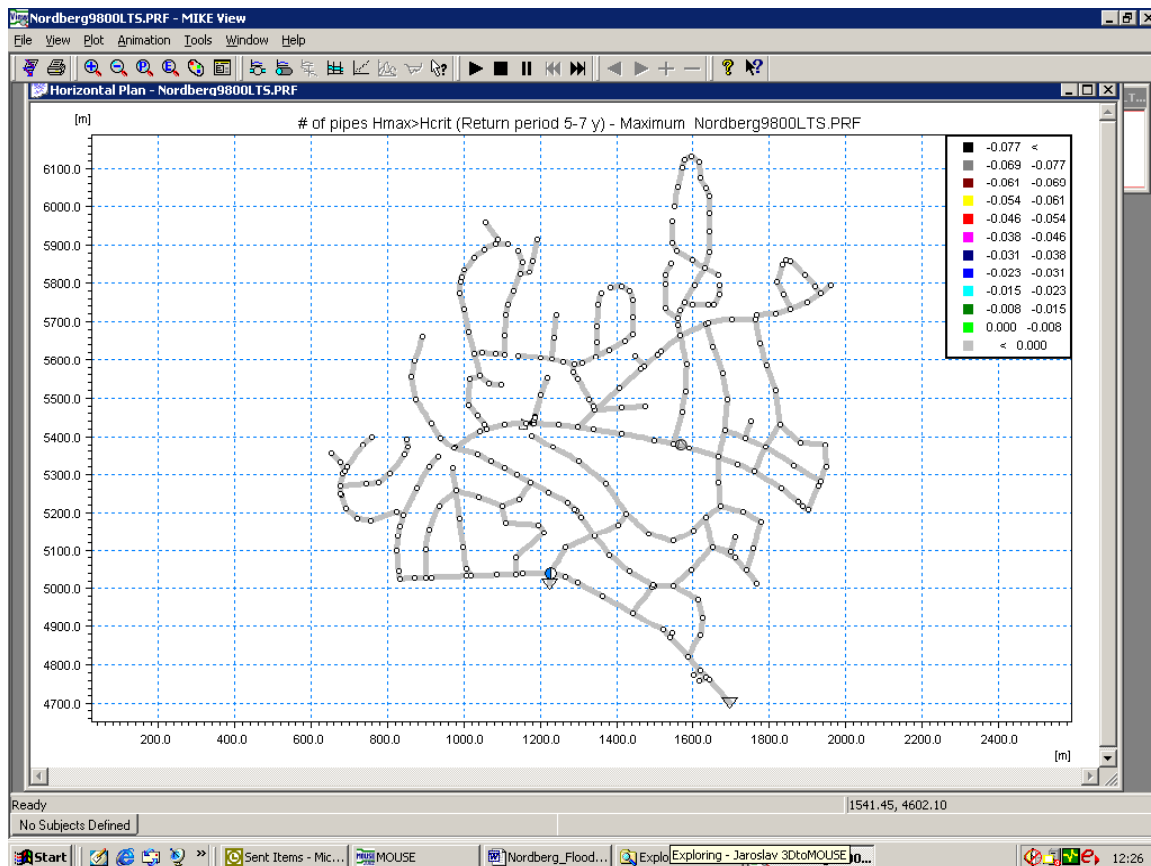


Figure A17 There is no risk of surface flooding for any location in the modelled Nordberg network at a runoff event which has a statistical probability of being equalled or exceeded 20 times in 100 years (**the 5-year surface flood**).

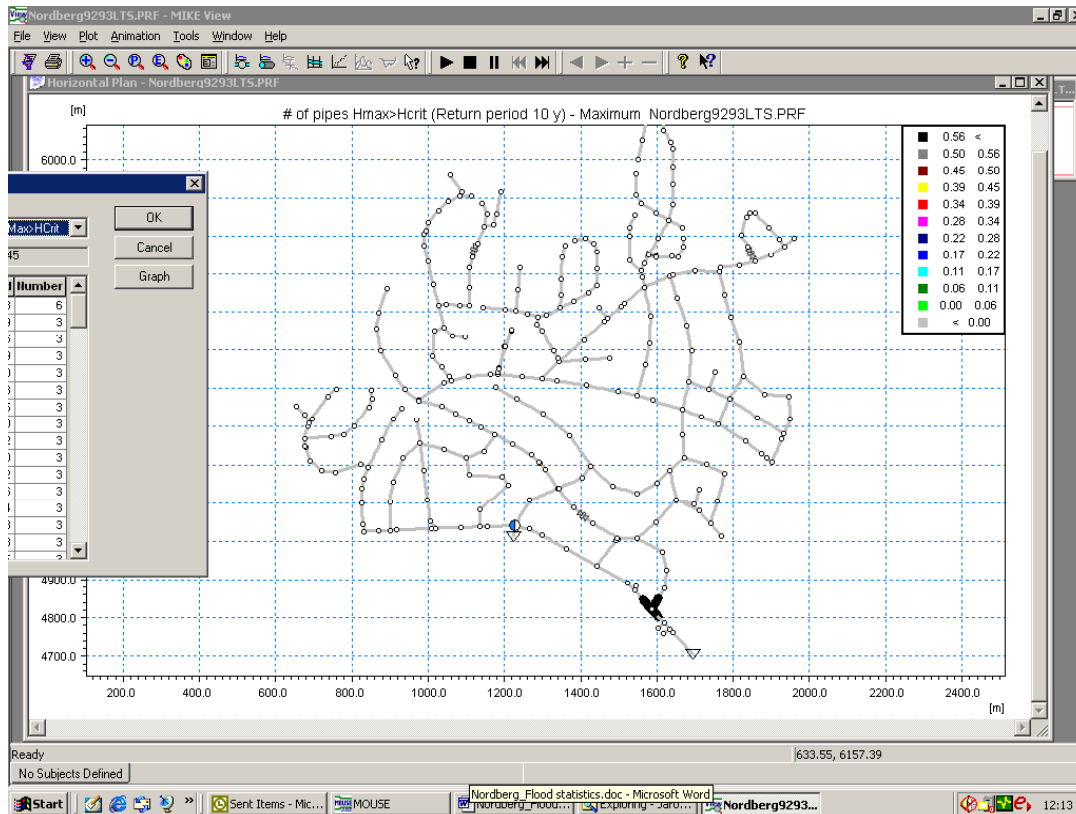


Figure A18 There is surface flooding in location near the catchments outlet at a runoff event which has a statistical probability of being equalled or exceeded 10 times in 100 years (**the 10-year surface flood**).

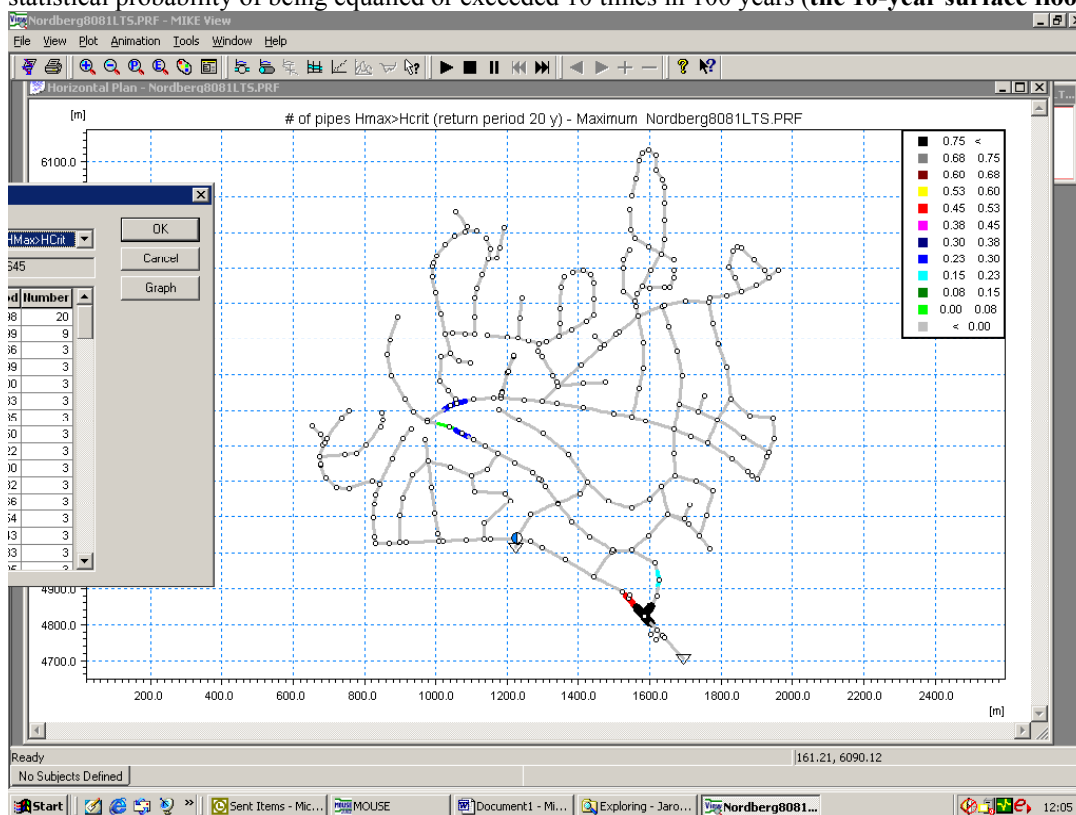


Figure A19 There is surface flooding in several locations in the modelled Nordberg network at a runoff event which has a statistical probability of being equalled or exceeded 5 times in 100 years (**the 20-year surface flood**).

The pipelines with 20-year frequency of high surcharging (and surface flooding) are shown in Figure A20.

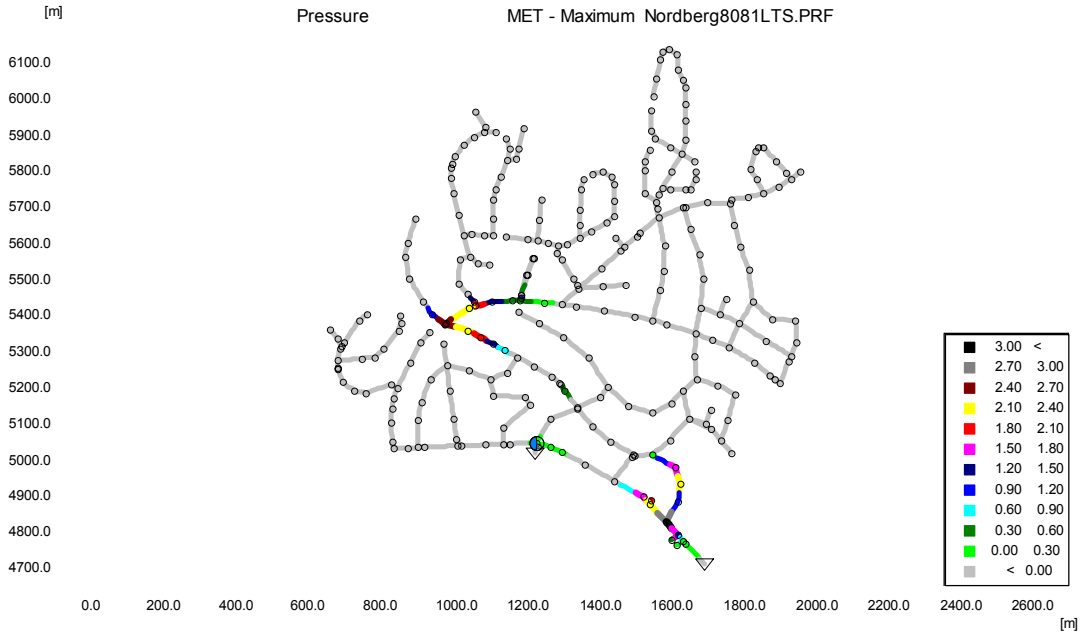


Figure A20 **High surcharging frequency**. Computed as a difference of the computed water level with **20-year return period** and the conduit ceiling

4.1.5 Use of MOUSE LTS module for long-term simulation and statistics in the Nordberg case study

The main principle behind the MOUSE LTS concept is to reduce the computational complexity associated with a long-term simulation. The implementation of the MOUSE LTS module in the Nordberg case study shows that the concept is not efficient in the case where RDII module is used to produce input hydrographs for MOUSE HD. The main restriction for a long-term discontinuous dynamic simulation of for instance 20-year is the size of the *.CRF file containing simulation results from RDII-A model. According to DHI, the problem is related to Windows and compiler for the MOUSE programming language.

The *.CRF file produced in Node 255618 which is located in the northern part of the Nordberg catchment area and characterized by the RDII parameter set for separated sewer system is shown in Figure A21. The file was generated over the one year period. The runoff result file *.CRF which was generated over a 32-year period (from 1968 to 2000) was more than 8 GB large.

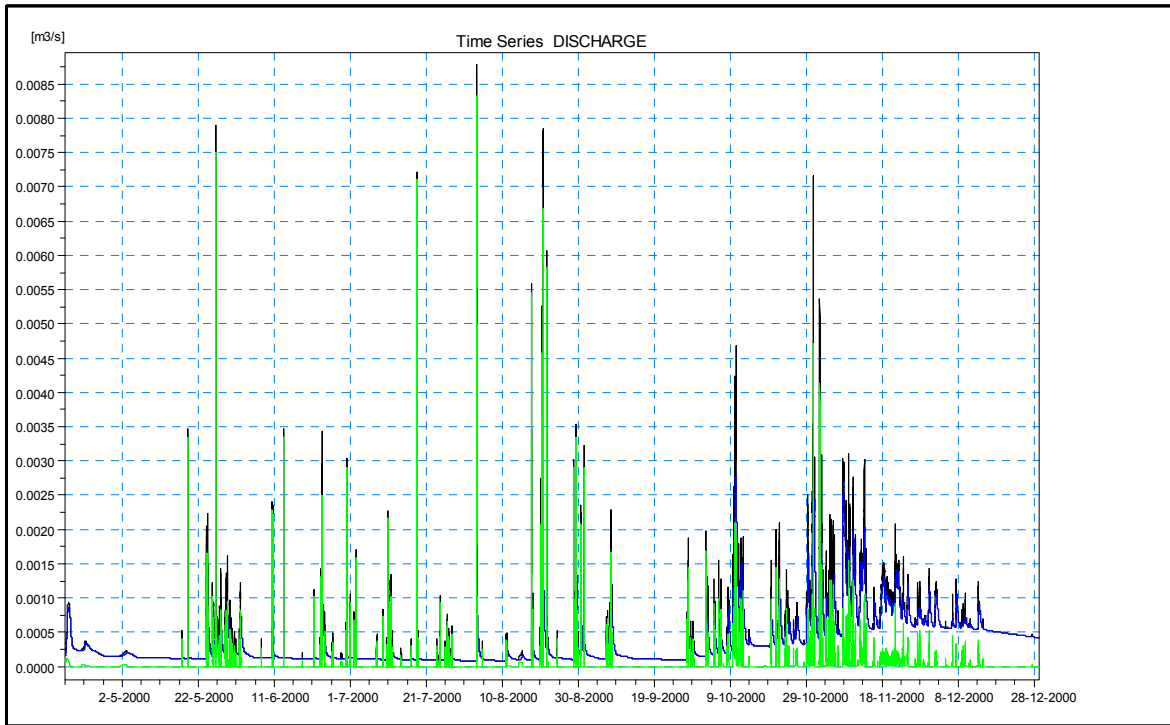


Figure A21 Example of results from runoff computation using RDII module. It can be noticed that in summer, it is the Fast Response Component (FRC) which contributes most to the total hydrograph. On the contrary, during winter, the Slow Response Component (SRC) is the predominant contributor to the inflow to the sewer system.

4.1.6 Overview over mouse files in the Nordberg case study

MOUSE HGF, UND, DWF and RPF file

HYDROLOGY

HGF File (HydroloGical File) Specification. The HGF in-data file for MOUSE contains data for catchments as well as the parameters sets for Runoff Model A (Time/Area routing – T-A curve), B (Non-linear Reservoir Method), C (Linear reservoir method in two sub-variants- C1 Dutch runoff model, C2 French runoff model) and UHM (Unit Hydrograph Method).

HYDRAULIC

UND File (Urban Network Data) Specification. The UND in-data file for MOUSE includes data from former SWF (Catchment and Pipe System Data File), BSF (Time-Series Boundary File), PWF (Hydraulic Data File for Pipe Flow Model), RWF (Selected Rain Data File), PID (Control of weirs, gates and pumps) and ADP (ADditional Parameter file) files. The *.ADP file is an ASCII file which temporarily includes new MOUSE functions prior to their full implementation in the MOUSE GUI, as well as some rarely used functions, which are of interest of only a few MOUSE users.

(For more information about UND and HGF format files see file specification document on BSCW server, new folders).

DRY WEATHER FLOW–QUANTIFICATION OF DRY WEATHER LOADING

Example of the *.DWF file:

AkrF00fore.DWF file:

[DWF_Definition]

SYNTAX_VERSION = 2

UNIT_TYPE = 1

DWF_LineHeader = 'LTYPE', 'DLOCATION', 'PART_ID', 'ITEM_ID', 'GROUP_NAME', 'CMETHOD', 'EFRACTION', 'BVALUE', 'DWF_UNITS'

DWF_Line = 1, ", 1, 1, 'AkrF00fore', 2, 100, 0.1356, 'm3/PE/Day'

[DWF_Parts]

SYNTAX_VERSION = 1

UNIT_TYPE = 1

DWF_PartHeader = 'PART_ID', 'PART_NAME'

DWF_Part = 1, 'Residential'

DWF_Part = 2, 'Commercial'

DWF_Part = 3, 'Base Flow'

DWF_Part = 4, 'Specific'

EndSect // DWF_Parts

[DWF_Items]

SYNTAX_VERSION = 1

UNIT_TYPE = 1

DWF_ItemHeader = 'ITEM_ID', 'ITEM_NAME'

DWF_Item = 1, 'Discharge'

DWF_Item = 2, 'BOD'

DWF_Item = 3, 'COD'

EndSect // DWF_Items

EndSect // DWF_Definition

REPETITIVE PROFILE–DEFINITION OF TEMPORAL DRY WEATHER VARIATION

Example of the *.RPF file:

AkrF00fore.RPF file:

// Created : 2003-03-28 8:21:44

// DLL id : E:\MOUSE2002\Bin\pfs2000.dll

// PFS version : Jul 23 2002 20:47:31

[Daily Profiles]

[hr24_variation]

hr24_ID = 'AkrF00fore'

hr24_Data_Real = 0.39, 0.39, 0.39, 0.39, 0.39, 0.76, 1.77, 1.77, 1.39, 1.14, 1.14, 0.89, 0.89, 0.89, 1.02, 1.02, 1.14, 1.27, 1.27, 1.39, 1.27, 1.14, 1.14, 0.76

EndSect // hr24_variation

[DP_Group]

Group_ID = 'AkrF00fore'

Interpolation = 1

[hr24_Set]

hr24_ID = 'AkrF00fore'

Days_Valid_ID = "

EndSect // hr24_Set

EndSect // DP_Group

EndSect // Daily Profiles

MOUSE Summary *.HTM files

MOUSE Summary *.HTM files contain information on the current simulation and result summary after specification).

Summary_RDII-A:



C:\CARE-S WP3\
Summary_RDII-A_stu

Summary_HD:



C:\CARE-S WP3\
Summary_HD_study_

Overview over MOUSE GM files – The Nordberg case study

Filename	Extension	Contents	ArcView file
combined	dbf, sbn, sbx, shp, shx	combined areas	shape-files Polygon (283)
export_omr	dbf, sbn, sbx, shp, shx	defines the area	shape-files Polygon (1)
gully_trap	aih, ain, dbf, sbn, sbx, shp, shx	gully trap	shape-files Point (425)
hgf	dbf, sbn, sbx, shp, shx	catchment data	shape-files Polygon (426)
rdii_param	dbf	parameters for RDII	
separated	dbf, sbn, sbx, shp, shx	separated system areas	shape-files Point (89)
storm	dbf, sbn, sbx, shp, shx	storm water areas	shape-files Point (54)
node	aih, ain, dbf, sbn, sbx, shp, shx	nodes from GVA	shape-files Point (1565)
pipe	aih, ain, dbf, sbn, sbx, shp, shx	pipes from GVA	shape-files PolyLine (1636)
basin	aih, ain, dbf, shp, shx	none	shape-files Point (0)
outlet	aih, ain, dbf, sbn, sbx, shp, shx	outlets with water level	shape-files Point (14)
pump	dbf, shp, shx	none	shape-files Point (0)
weir	dbf, sbn, sbx, shp, shx	weirs	shape-files Point (5)
link	dbf		
building	dbf, shp, shx	buildings	shape-files Polygon (6348)
population	dbf, shp, shx	population data related to buildings	shape-files Point (3502)
road	dbf, shp, shx	roads	shape-files Polygon (146)
area_selection	avl, dbf, sbn, sbx, shp, shx	the whole area	shape-files Polygon (2)
felles	avl		legend definition
felles	dbf, shp, shx	none	shape-files Point (0)
level_3	avl		legend definition
level_3	dbf, shp, shx	pipes in upper left zone	shape-files PolyLine (635)
malesone	avl		legend definition
malesone	dbf, shp, shx	defines measures zones (5)	shape-files PolyLine (21)
pipes	avl		legend definition
pipes	dbf, shp, shx	pipes	shape-files PolyLine (1632)
rdiisetaf	avl		legend definition
rdiisetaf	dbf, shp, shx	catchment with combined system	shape-files Polygon (219)
rdiisetov	avl		legend definition
rdiisetov	dbf, shp, shx	catchment with storm system	shape-files Polygon (43)
rdiisetasp	avl		legend definition
rdiisetasp	dbf, shp, shx	catchment with separated system	shape-files Polygon (52)
respipe	dbf, shp, shx	pipes in upper left zone	shape-files PolyLine (645)
typeomrfelles	avl, dbf, shp, shx	catchment with combined system	shape-files Polygon (219)
typeomroverv	avl, dbf, shp, shx	catchment with storm system	shape-files Polygon (43)
typesep	avl, dbf, shp, shx	catchment with separated system	shape-files Polygon (52)
basin	avl	basin	legend definition
building	avl	building	legend definition
combined	avl	combined area	legend definition
cross section	avl		legend definition
gully trap	avl		legend definition
hgf	avl		legend definition

Filename	Extension	Contents	ArcView file
manhole	avl	manhole/nodes	legend definition
outlet	avl	outlet	legend definition
pipes	avl	pipes	legend definition
population	avl	population	legend definition
pump	avl	pump	legend definition
road	avl	road	legend definition
separat	avl	separate area	legend definition
slope	avl		legend definition
status	avl		legend definition
storm	avl	storm area	legend definition
top_level	avl		legend definition
weir	avl	weir	legend definition

A1.5.4 MOUSE GM files. Site themes: Buildings, Population and Road dbf files – The Nordberg case study

Urban – Building

AREA	PERIMETER	BYGG01_	BYGG01_ID	OPPR	KOORDH	PTEMA	AJOURDATO	MAALEMETOD	NOEYAKTIGH	SYNBARHET	H_MAALEMET	H_NOEYAKTI	MAX_AVVIK	BYGGTYP	KOMM	DATO
66,414	33,251	4140	4139	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
3,936	9,160	4156	4155	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
12,512	14,863	4175	4174	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
110,029	44,159	4185	4184	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
6,777	10,419	4238	4237	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
20,764	18,251	4246	4245	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
1068,940	184,817	4272	4271	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
246,489	72,352	4287	4286	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
13,562	14,852	4291	4290	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
5204,549	398,518	4300	4299	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
65,211	32,381	4324	4323	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
19,021	18,569	4330	4329	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
34,291	24,654	4339	4338	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000
49,729	28,462	4340	4339	0	0,00	0	0	0	0	0	0	0	0	0	0	0 00000000

Urban – Population

GNR	HNR	LITRA	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	ADR	TREFF	NYX	NY	ADR2
10063	1		1	1	0	0	0	0	0	0	0	0	0	0	0	1	0	10063 1	0	2916,00	784,00	10063 1
10063	2		3	2	1	0	0	0	0	0	0	0	0	2	1	0	0	10063 2	0	2962,00	805,00	10063 2
10063	4		5	2	3	0	0	0	0	0	0	0	1	2	2	0	0	10063 4	0	2970,00	808,00	10063 4
10063	5		4	3	1	0	0	0	0	1	0	0	0	2	1	0	0	10063 5	0	2926,00	790,00	10063 5
10063	6		2	1	1	0	0	0	0	0	0	0	0	0	1	1	0	10063 6	0	2976,00	812,00	10063 6
10063	7		4	3	1	2	0	0	0	0	0	0	0	1	1	0	0	10063 7	0	2928,00	792,00	10063 7
10063	8		3	3	0	0	0	0	0	1	0	1	0	1	0	0	0	10063 8	0	2979,00	814,00	10063 8
10063	9		2	1	1	0	0	0	0	0	0	0	0	1	1	0	0	10063 9	0	2936,00	797,00	10063 9
10063	10		2	1	1	0	0	0	1	0	0	0	0	1	0	0	0	10063 10	0	2986,00	819,00	10063 10
10063	11		4	2	2	0	0	0	0	0	1	0	0	2	1	0	0	10063 11	0	2940,00	801,00	10063 11
10063	12		4	1	3	0	1	0	1	0	0	0	0	1	1	0	0	10063 12	0	2988,00	821,00	10063 12
10063	13		1	1	0	0	0	0	0	0	0	0	0	0	0	1	0	10063 13	0	2944,00	806,00	10063 13
10063	14		3	2	1	0	0	0	0	0	0	0	0	1	1	1	0	10063 14	0	2995,00	825,00	10063 14
10063	15		2	1	1	0	0	0	0	0	0	0	0	1	1	0	0	10063 15	0	2952,00	813,00	10063 15

Urban – Road

AREA	PERIMETER	HVEI05_	HVEI05_ID	OPPR	KOORDH	PTEMA	MAALEMETOD	NOEYAKTIGH	SYNBARHET	H_MAALEMET	H_NOEYAKTI	MAX_AVVIK	MEDIUM	STATUS	KOMM	DATO	VEGTYPE	VEGSTATUS	VEGNUMMER	HOVEDPARSE	METER_FRA	METER_TIL	VKJØRFLT	VFRADATO
6018,997	1859,695	228	2613	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
2036,026	735,823	231	2612	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
5078,853	1518,911	255	1867	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
4265,022	1026,046	261	2611	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	F	V		0	0	0	0	00000000	
4027,620	1390,606	263	1839	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	P	V		0	0	0	0	00000000	
5707,626	1565,256	266	1837	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	F	V		0	0	0	0	00000000	
1447,142	441,685	275	2610	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
763,766	524,579	288	2607	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	P	V		0	0	0	0	00000000	
3101,875	1071,238	289	1838	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
9479,216	2533,009	291	1904	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	F	V		0	0	0	0	00000000	
7534,457	2065,007	293	1909	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
397,694	124,496	306	2615	3	0,00	7002	82	300	0	61	100	0	L	301	01.07.1999	F	V		0	0	0	0	00000000	
915,388	299,182	312	1844	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	P	V		0	0	0	0	00000000	
2368,652	819,348	317	2603	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	
2090,490	685,379	318	1843	3	0,00	7002	82	300	0	61	100	0		301	01.07.1999	K	V		0	0	0	0	00000000	

4.2 A2 The Correggio catchment case study (Reggio Emilia – Italy)

The Correggio municipality is located at 12 km in North – East direction from Reggio Emilia in Emilia Romagna region in Italy.

The catchment can be considered as a residential area. Actually the 487 ha are occupied by 20769 inhabitants (31.12.2000).

In the last decades the evolution of economic activities inside the area has increased the extension of impervious surfaces with the consequence of much more water to drain. The municipality was not able to improve the system performance, so many areas of the network are not capable to produce the drainage required: lots of flooding events, even with a 2-5 years return period rain, are the most evident result of this problem.

Water flows inside the pipes from the north to the east. Thus the main outlet of the system is located in the final node of the network in the east side of the area. This is a gravity system with a impermeability coefficient of 27%. Pipes depth is from 1 to 3 m below the road.

The sewer system has a total length of 92,7 km: 63% of which are occupied by combined sewer pipes, 32% it is a separate system, 2% are open channels and the other 2% it is located in the old town center of Correggio and it is not known how the system is defined (Fig.A2-1).

The 78% of the pipes are in concrete material, while the other 20% is, more or less, in PVC.

Pipes sections can be classified in:

- 91% of the total length circular section;
- 7,4% of the total length rectangular section;
- 1,6% of the total length, other.

Diameters of pipes are minor than 600 mm for the 80%:

- 43% of the total length ≤ 300 mm
- 35% of the total length $350 < DN < 600$ mm
- 13% of the total length $700 < DN < 800$ mm
- 10% of the total length > 800 mm.

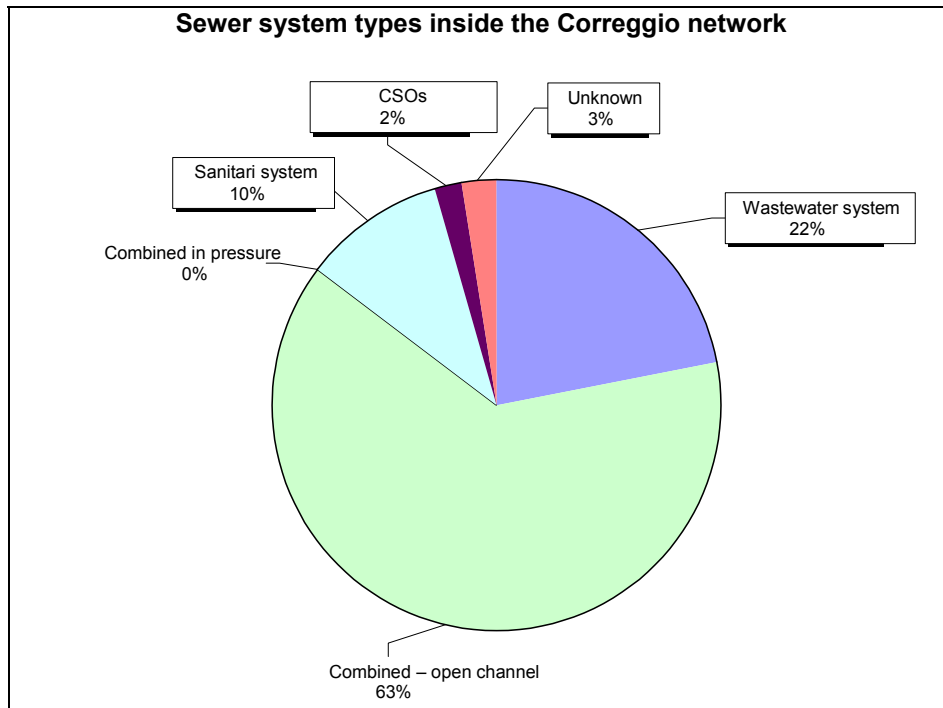


Fig.A2 1: Sewer types on the Correggio system

The studied area can be divided in 7 zones (Fig. A2- 2)

- Zona A: it is the old side of the town located in the centre of Correggio;
- Zona B: located in the South part of the system;
- Zona C: located in the centre – East side of the system;
- Zona D: located in the West side of the network;
- Zona E: it is the portion of the system on the North of the old town centre;
- Zona F: the South – East side;
- Zona G: it is the industrial area.

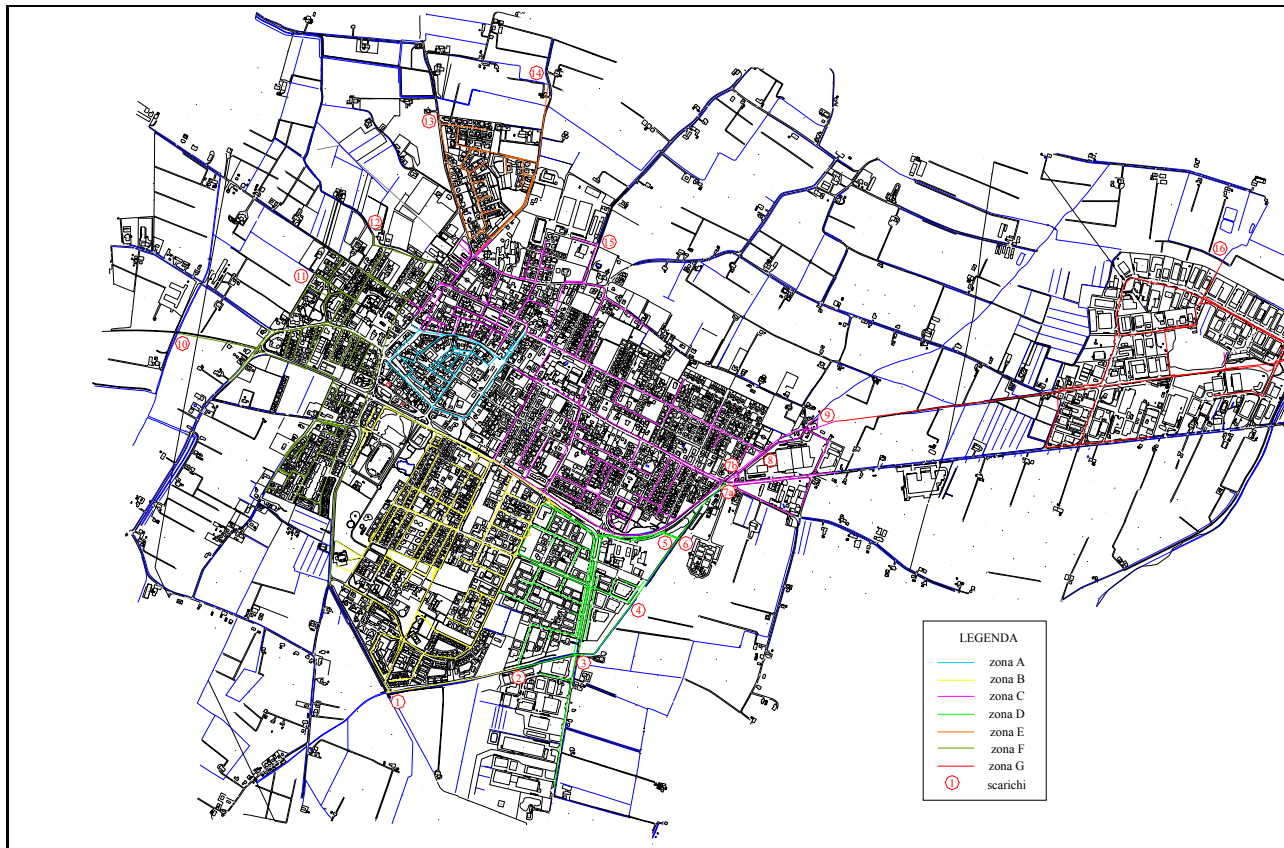


Fig.A2 2: Distribution of the seven areas on the Correggio system.

The system include 5 receiving water bodies:

- Cavo Argine: it is the most important of the water bodies which influences the drainage system performance. Its water level interests zone B, C and F.
- Cavo Rio;
- Fossetta Seconda;
- Scolo Mandrio;
- Fossetta delle Monache.

Using data provided by the Correggio municipality it was possible to define the areas subject to frequent flooding events and draw a map where those areas are highlighted (Fig. A2 - 3) The map is a useful tool to compare real flooding information with the model results.

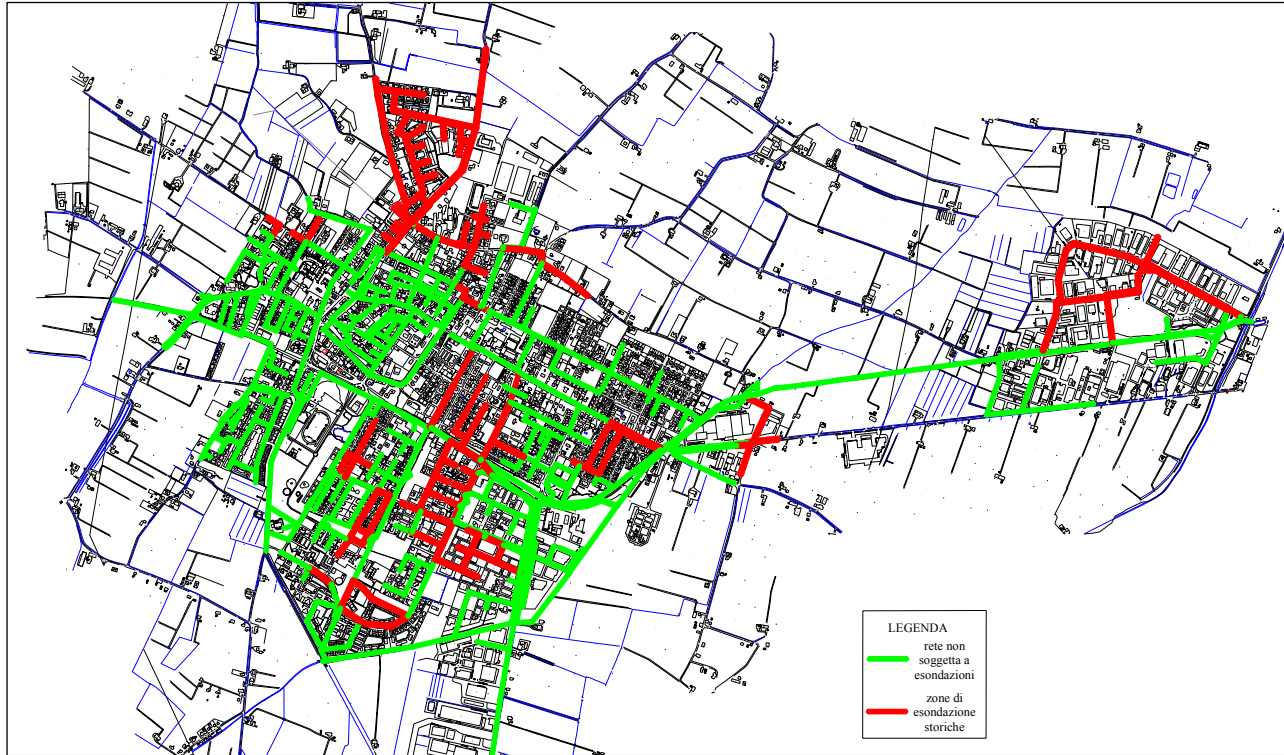


Fig.A2 3.: Flooding Map

4.2.1 The hydraulic model structure

The following picture shown a synthetic scheme of the Hydraulic and quality model structure used by InfoWorks.

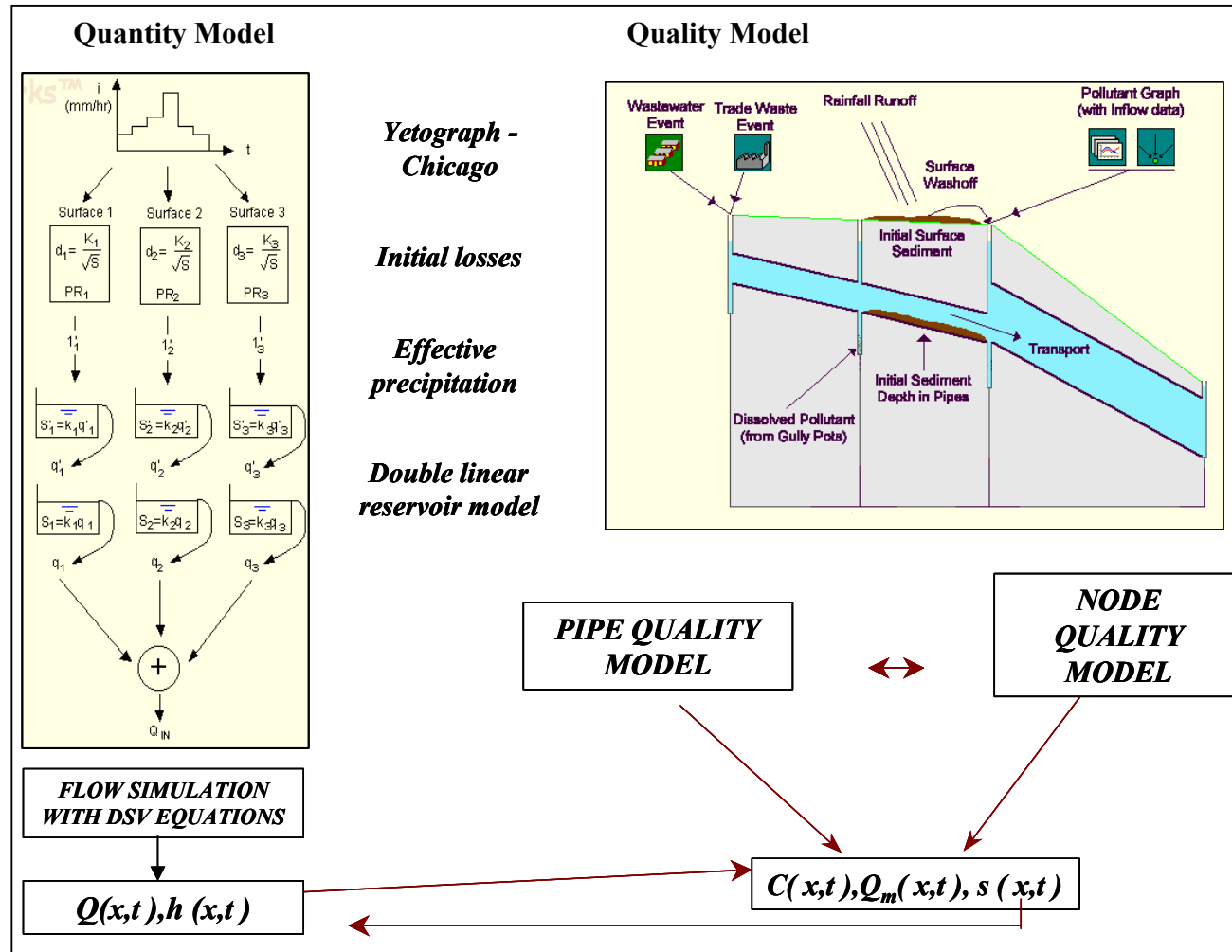


Fig.A2-4: InfoWorks Hydraulic and quality model structure

The catchment of 470 ha was divided in 540 sub - catchments. Sub – catchments boundaries was defined considering the contribution area for each manhole.

The drainage system include 888 pipes and 816 nodes, classified in 769 manholes, 6 ponds and 14 outfalls.

There are 2 manholes with pumps stations.

4.2.1.1 Sensitivity analysis

The parameters sensitivity analysis permits the evaluation of importance of each parameter variation on the system simulations.

Simulations have been done with different values of:

- initial total loss for impervious areas;
- k coefficient, used to calculate initial losses as: $D = k/\sqrt{s}$ (where D is the initial loss in m; s is the surface slope in m/m; k explains the topography of the surface in m);
- Soil type: coefficient that explains the soil infiltration capacity; it ranges between 0.15 and 0.50, where the highest values means low infiltration;
- UCWI : (Urban Catchment Wetness Index) for verification storms is individually calculated using data from the Met Office and the precipitation over the previous 5 days;
- pipes roughness;
- K coefficient for calculating local head losses ($\Delta H = KV^2/2g$).

INITIAL TOTAL ABSOLUTE LOSS FOR IMPERVIOUS AREAS

The Water Cycle requires no explanation here. It can be considered to start with evaporation from the sea creating clouds of water vapour which then move over land and deposit their water load as rain. The rainfall lands on the ground where some of it is immediately lost (the “Initial Losses”) due to evaporation (especially off warm surfaces), wetting of surface layers or dust, absorption into shallow surface layers and the filling of shallow depression storage (from which it later evaporates). If the rainfall exceeds the initial losses a proportion of the rainfall creates runoff whilst the remainder soaks into the ground and contributes to the soil storage. The proportion that is directed to runoff depends upon a number of factors but the chief one is the nature of the surface. An impermeable surface such as roads or roofs cause a high proportion of the rainfall to contribute to the runoff whilst a recently ploughed field may create no runoff with all of the rain

soaking into the soil. The degree of wetness of the surfaces varies throughout the period of the storm. Impermeable surfaces tend to become fully saturated very quickly and thereafter all of the rainfall is turned into runoff though some of that runoff may go to permeable areas and may not enter any drainage network. Once these impermeable surfaces have been wetted the percentage runoff does not vary greatly. Permeable surfaces react differently. As the storm progresses the upper layers of the soil become wetter and wetter and when the rainfall exceeds the rate at which it can soak into the ground the rainfall is turned into runoff and when the rainfall intensity drops below the soakage rate the runoff ceases even though rainfall may still continue. Therefore the percentage runoff varies throughout the duration of the storm. Runoff from impermeable surfaces is routed via guttering, rainwater pipes and gullies into the sewerage system. The extent of impermeable surfaces that drain to sewerage networks varies considerably depending on a number of factors. Areas that have good soakage characteristics frequently have roofs and paved surfaces connected to soakaways. In many areas the roads and roofs drain directly into the sewerage systems. This is not always the case with footpaths, especially where there is a grass verge inbetween the footway and the road – in these cases the impermeable surfaces may drain to permeable areas. Therefore most sewers with any impermeable surfaces connected, exhibit a quick response to rainfall with a peak flow occurring a short time after the peak rainfall. Rainfall, which soaks into the ground, is directed into the ‘soil storage reservoir’ but when the soil reaches a certain saturation threshold (the percolation threshold) water starts to percolate downwards. A proportion of this percolation flow might infiltrate directly into the sewer network whilst the remainder penetrates deeper into the groundwater storage reservoir. The infiltration flows into the sewers are therefore not dependant upon the groundwater table rising up to above the invert level of the pipelines.

With all the others parameter constant at the following values:

- k coefficient (permeable surface): 0.00028 m;
- Roughness: $65 \text{ m}^{1/3}/\text{s}$
- UCWI: 80
- Soil type: 5
- K coefficient: 0.5

Simulations were done with different values of initial losses for impermeable surfaces.

Next tables and fig. explain how the “initial absolute loss” weigh upon the total inflow and on the flow throughout the pipe section immediately before the manhole 300645:

Absolute initial loss (m)	Total runoff (mc)	Total inflow (mc)	Total Inflow Variation(%)	Volume to WWTP (mc)	Outfalls (mc)
0.0002	19334.8	20218.1	0.00	6842	13292.9
0.002	17417	18300.4	-9.5	6576.4	11809.6
0.008	11024.5	11907.9	-41.1	5488.7	6460.1
0.02	1968.4	2851.8	-85.9	2681.7	354

Tabl. A2-1: Total water volumes in the system

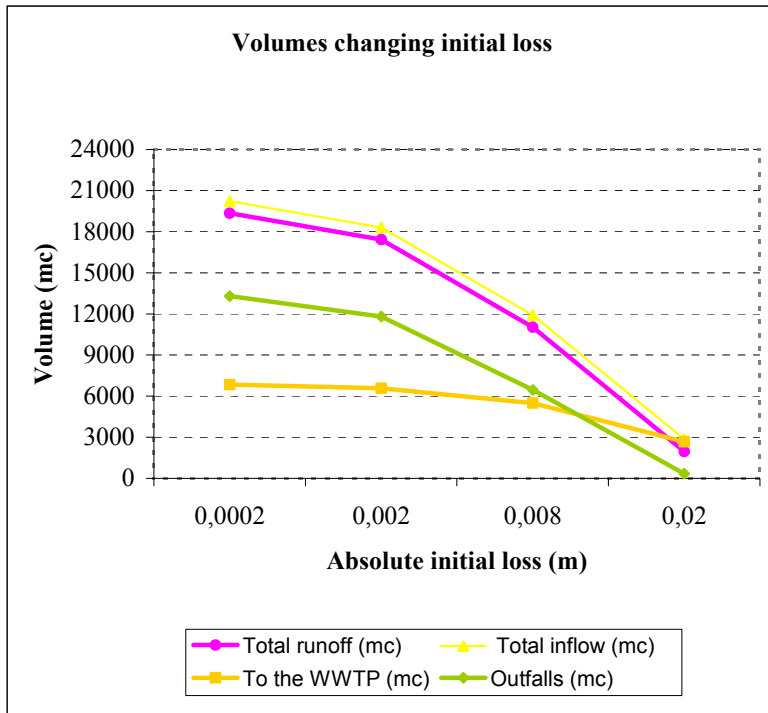


Fig. A2-5: Volumes comparison for different initial loss in the system

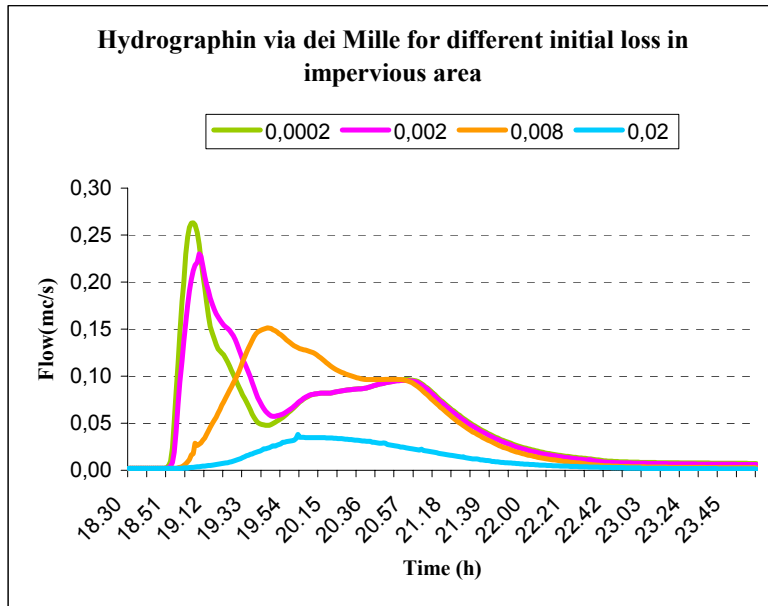


Fig. A2-6: Hydrograph in Via dei Mille for different initial loss in impervious area.

It is evident that increasing the initial loss of impervious surface the flow peak decreases and it occurs in a postponed time.

INITIAL TOTAL RELATIVE LOSS FOR IMPERVIOUS

With the others parameters constant at the following values:

- Coeff. K (permeable surf.): 0.00028
- roughness coefficient: 65 m^{1/3}/s
- UCWI : 80
- K = 0.5
- Soil type: 5

simulations with different values of K coefficient for impervious areas were done:

Coeff. K (m)	Total runoff (mc)	Total inflow (mc)	Total Inflow Variation(%)	Volumes to WWTP (mc)	Outfalls (mc)
0.000028	18880.8	19764.2	0.0	6802.9	12938.6
0.000071	17856.4	18739.8	-5.2	6653.1	12101.5
0.00028	12877.3	13760.6	-30.4	5840.6	7977.9
0.00071	2633.2	3516.5	-82.2	3084.7	576.9
0.0028	1968.4	2851.8	-85.6	2681.7	354

Tabl. A2-2: Total water volumes in the system

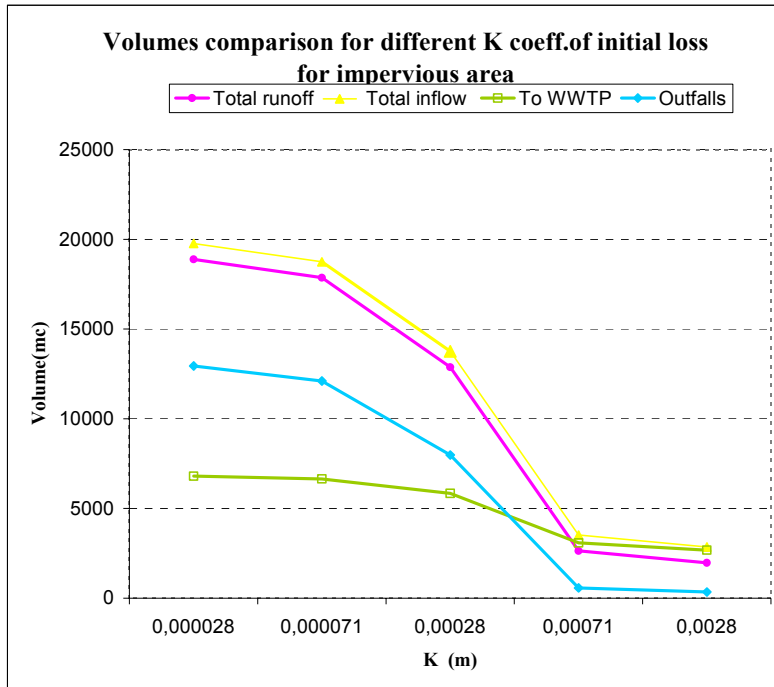


Fig. A2-7: Volumes comparison for different Values of K coefficient for impervious area

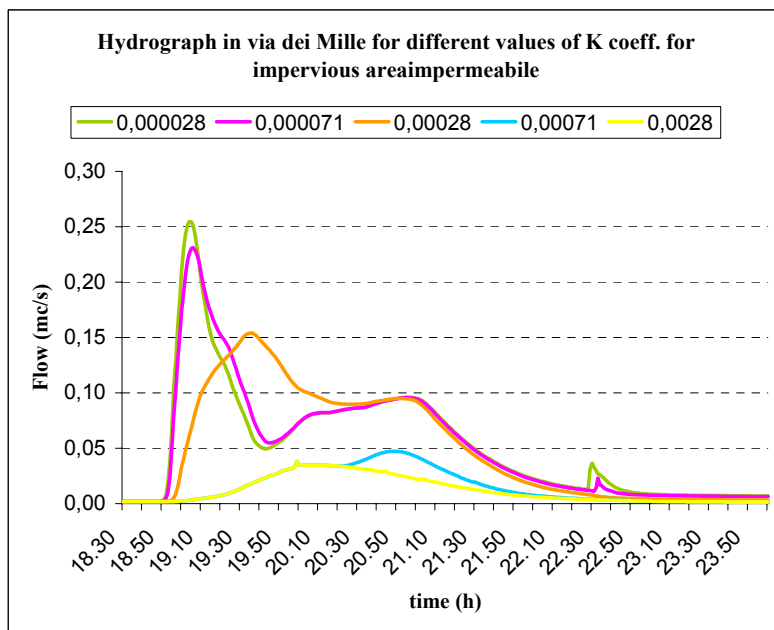


Fig. A2-8: Hydrograph in Via dei Mille for different values of K coefficient for impervious area.

INITIAL TOTAL RELATIVE LOSS FOR PERMEABLE AREAS

Constant parameters:

- Coeff. K (impervious surf.):0.000071
- roughness coefficient: 65 m^{1/3}/s
- UCWI : 80
- K = 0.5
- Soil type: 5

simulations with different values of K coefficient for permeable areas were done:

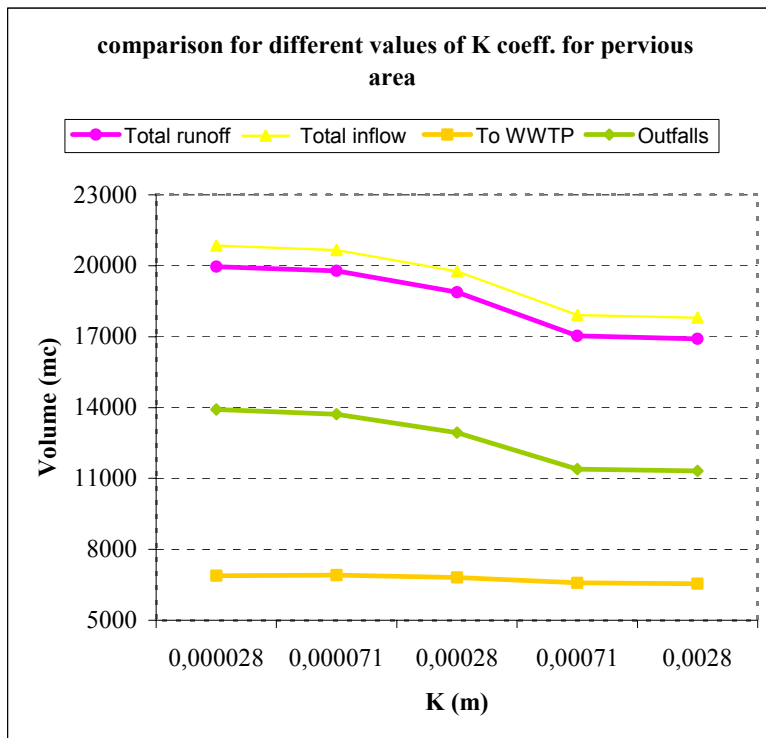


Fig. A2-9: Volumes comparison for different Values of K coefficient for pervious area

Coeff. K (m)	Total runoff (mc)	Total inflow (mc)	Total Inflow Variation(%)	Volumes to WWTP (mc)	Outfalls (mc)
0.000028	19964.1	20847.5	0.0	6884	13915.8
0.000071	19779.3	20662.6	-0.9	6911.1	13712.6
0.00028	18880.8	19764.2	-5.2	6802.9	12938.6
0.00071	17032.3	17915.7	-14.1	6587.2	11392.6
0.0028	16912.4	17795.7	-14.6	6546.2	11316.1

Tabl. A2-3: Total water volumes in the system

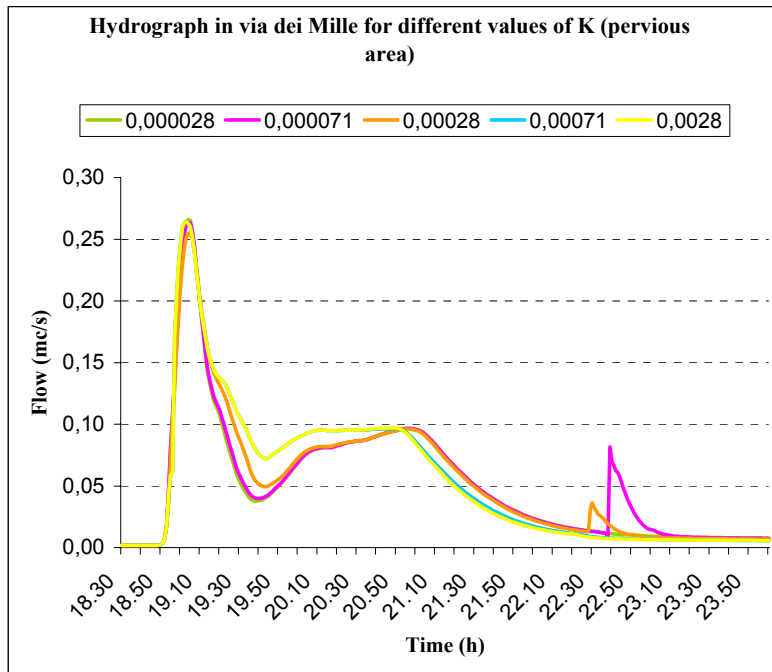


Fig. A2-10: Hydrograph in Via dei Mille for different values of K coefficient in pervious area.

Increasing the total losses on the permeable surfaces:

- peak instant and peak flow don't change;
- total volume throw the section considered increases during the rain event studied.

From this first analysis we can conclude that between permeable and impermeable parameters the most significant for the calibration are the impermeable parameters.

ROUGHNESS

Standing all the others parameters:

- Coeff. K (impervious surf.):0.000071
- Coeff. K (permeable surf.):0.00028
- UCWI : 80
- $K = 0.5$
- Soil type: 5

simulations with different values of K_s (Gauckler – Strickler coefficient)($m^{1/3}/s$) coefficient were done:

Roughness Ks (m ^{1/3} s ⁻¹)	Total runoff (mc)	Total inflow (mc)	Volumes to the WWTP (mc)	Volume Var. % to WWTP	Outfalls (mc)
40	17856.4	18739.8	5905.5	-15.8	12588.6
50	17856.4	18739.8	6235.3	-11.1	12380.3
55	17856.4	18739.8	6374.2	-9.1	12301.7
60	17856.4	18739.8	6513.4	-7.2	12211.6
65	17856.4	18739.8	6653.1	-5.2	12101.5
70	17856.4	18739.8	6752.1	-3.8	12003.4
75.2	17856.4	18739.8	6861.4	-2.2	12082.4
80	17856.4	18739.8	6883.5	-1.9	11849.9
84.7	17856.4	18739.8	7015.9	0.0	11693.9

Tabl. A2-4: Total water volumes in the system

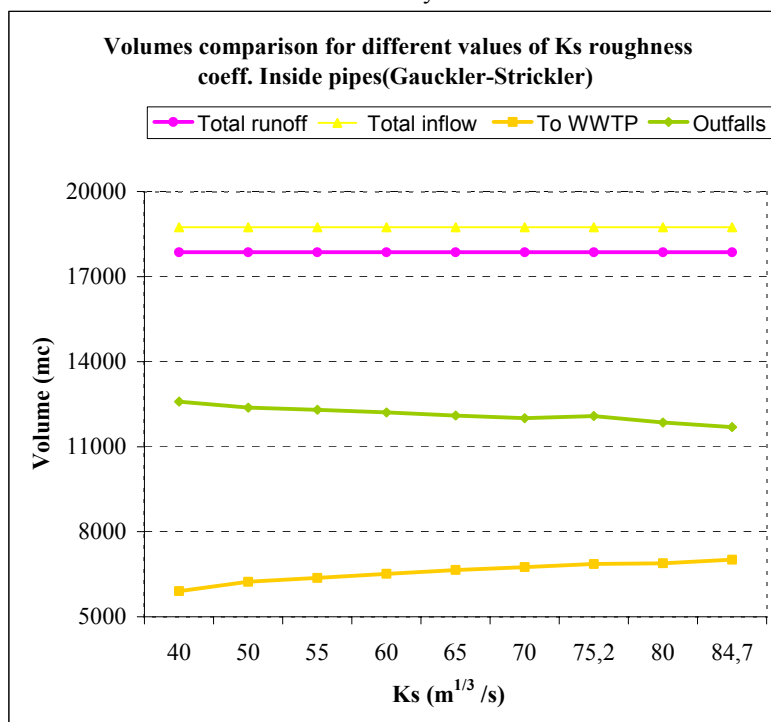


Fig. A2-11: Volumes comparison for different values of Ks roughness coefficient inside pipes

As we can see, roughness doesn't influence too much the volumes going to the WWTP or the total outfalls. But it is more important for its effects on flow throw the section studied; as explained in the following graphs:

Increasing Roughness:

- the maximum flow occurs before;
- increase the peak flow;
- for $40 < Ks < 65 \text{ m}^{1/3} / \text{s}$ the second part of the curve is more detailed.

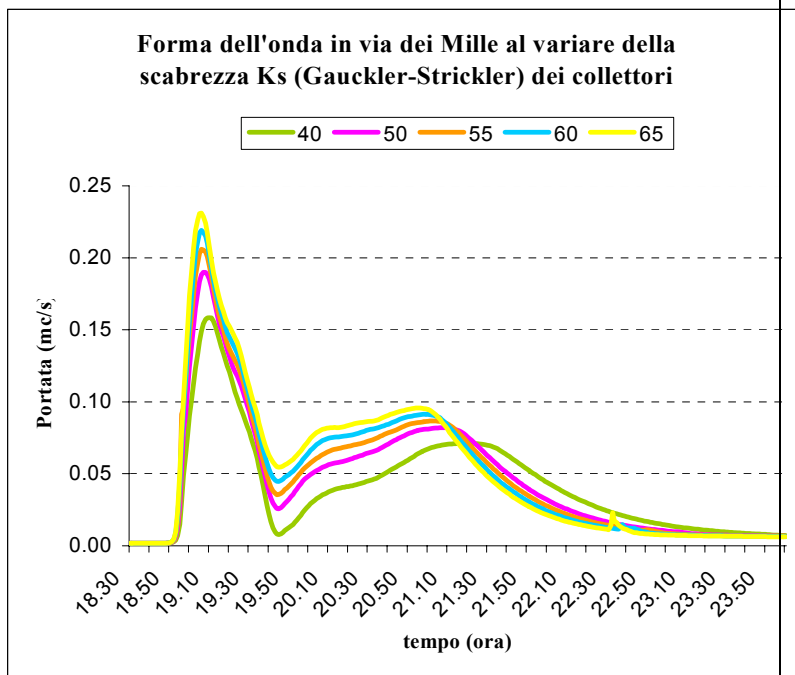


Fig. A2-12: Hydrograph in Via dei Mille for different values of K_s roughness coefficient inside pipes.

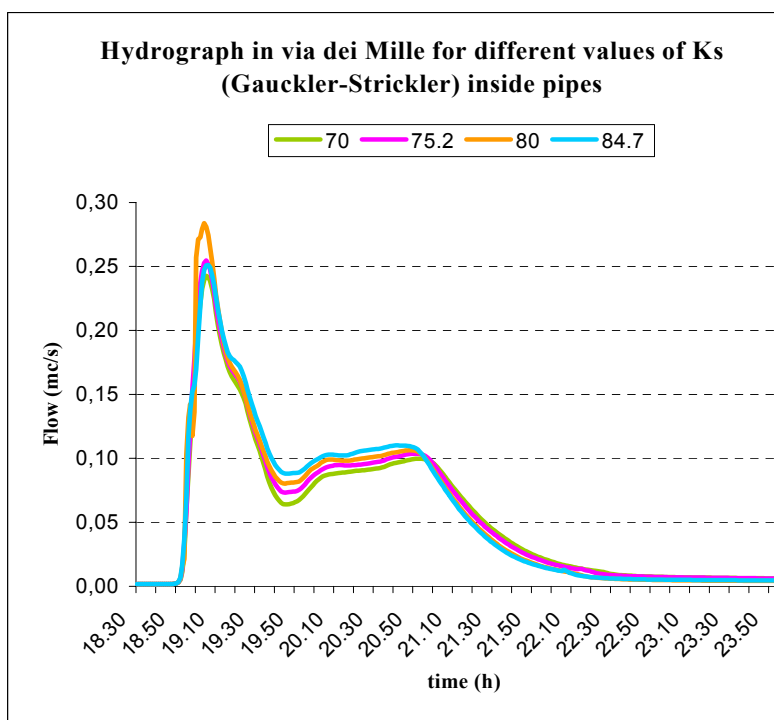


Fig. A2-13: Hydrograph in Via dei Mille for different values of K_s roughness coefficient inside pipes.

UCWI

Standing all the others parameters:

- Coeff. K (impervious surf.):0.000071
- Coeff. K (permeable surf.):0.00028
- Ks: 65 ml/3 /s
- K = 0.5
- Soil type: 5

simulations with different values of UCWI coefficient was done:

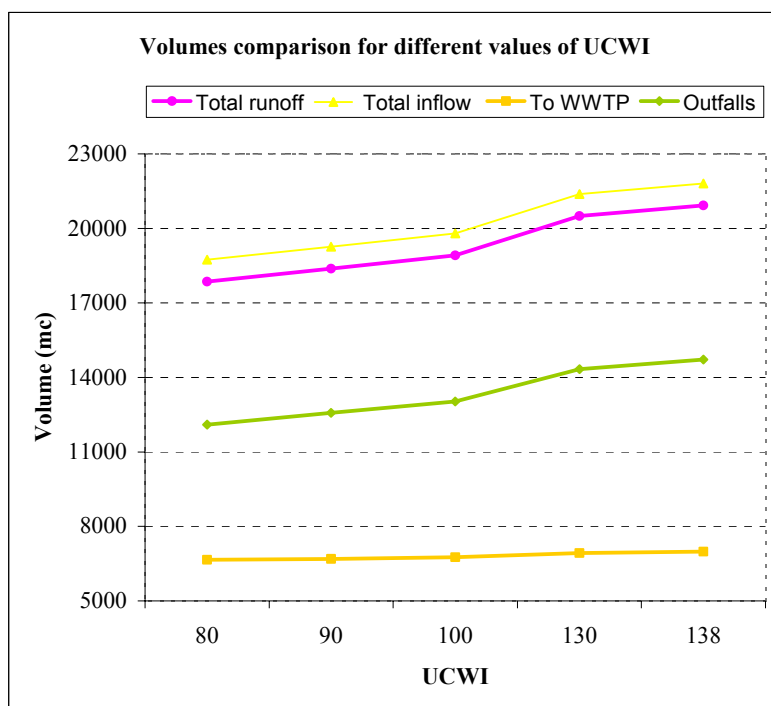


Fig. A2-14: Volumes comparison for different values of UCWI

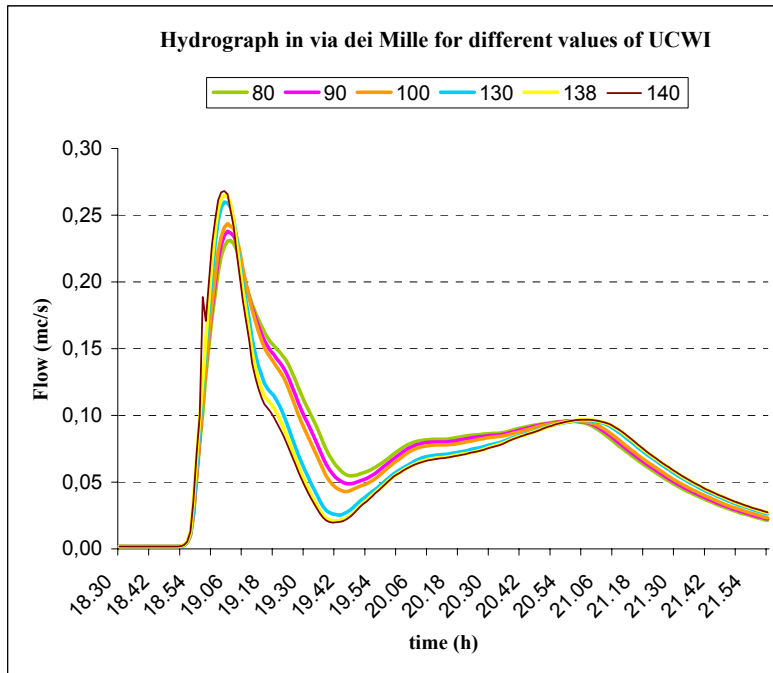


Fig A2-15: Hydrograph in Via dei Mille for different values of UCWI.

Tabl. A2-5: Total water volumes in the system

Ucwi	Total runoff (mc)	Total inflow (mc)	Total Inflow Variation (%)	Volumes to WWTP (mc)	Outfalls (mc)
80	17856.4	18739.8	-14.1	6653.1	12101.5
90	18385.7	19269.1	-11.6	6694.4	12574.6
100	18915	19798.4	-9.2	6759.8	13029.4
130	20502.9	21386.3	-1.9	6932.8	14329
138	20926.4	21809.7	0.0	6982.3	14719.4

Increasing the Urban Catchment Wetness Index:

- increases the flow peak;
- the maximum flow occurs before;
- the relative maximum of the second part of the curve doesn't change.

LOCAL HEAD LOSSES

The types of local head losses analyzed for the sensitivity analysis are:

- k constant depending on the surcharge ratio = 0.25
- k constant depending on the surcharge ratio = 0.50

- Normal headloss
- High headloss

Standing all the others parameters:

- Coeff. K (impervious surf.):0.000071
- Coeff. K (permeable surf.):0.00028
- Ks: 65 m^{1/3}/s
- UCWI: 80
- Soil type: 5

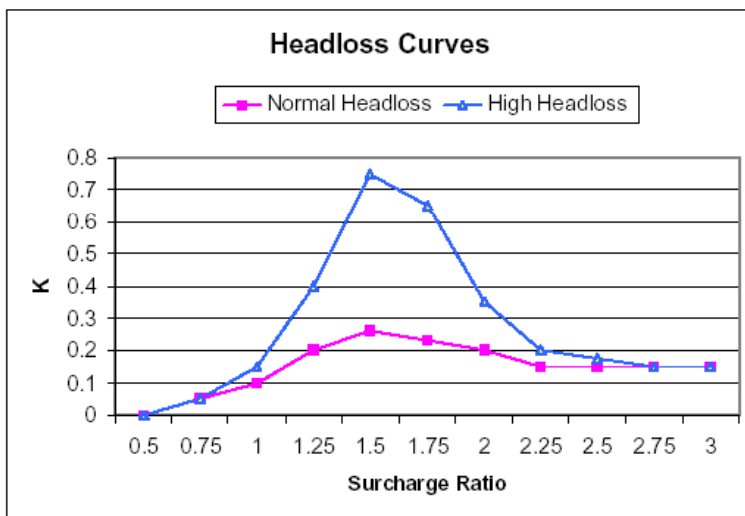


Fig. A2-16: Headloss Curves: Normal headloss and High headloss

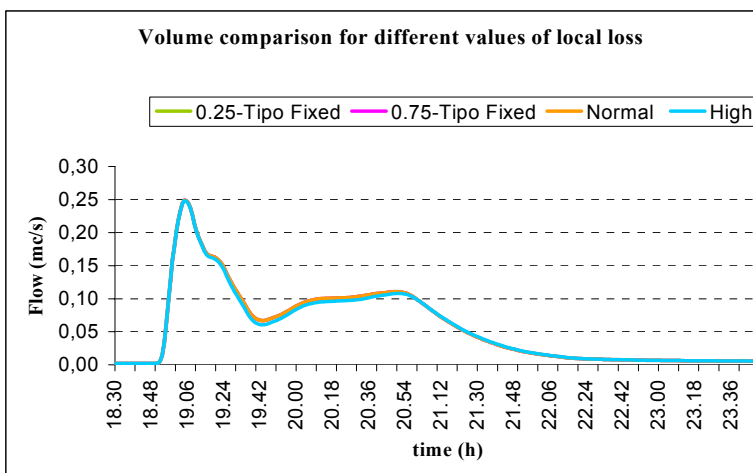


Fig. A2-17: Volume comparison for different values of local loss.

K coefficient doesn't influence at all hydraulic performances.

SOIL TYPE

Standing all the others parameters:

- Coeff. K (impervious surf.):0.000071
- Coeff. K (permeable surf.):0.00028
- Ks: 65 ml/3 /s
- K = 0.5
- UCWI = 80

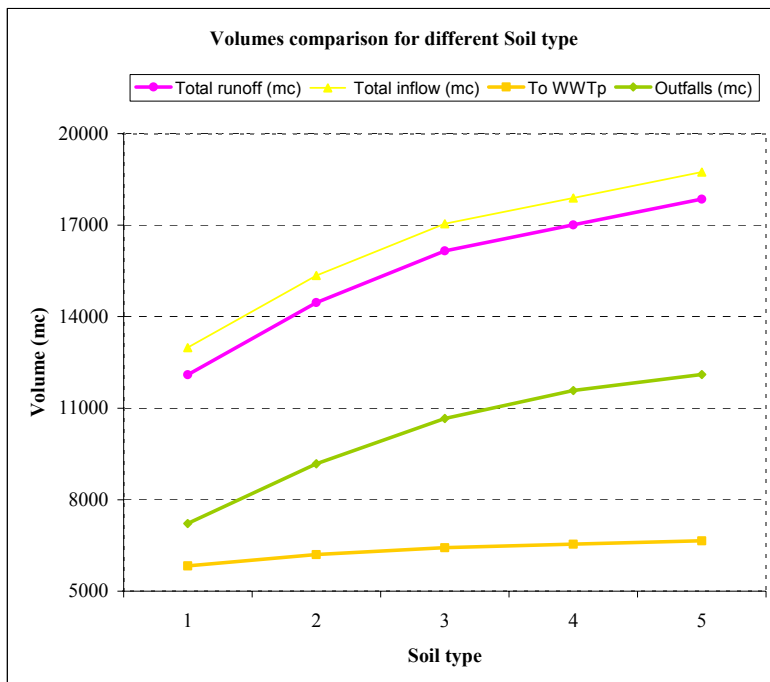


Fig. A2-18: Volume comparison for different Soil type

Soil Type	Total runoff (mc)	Total inflow (mc)	Total Inflow Variation(%)	To WWTP (mc)	Outfalls (mc)
1	12098.5	12981.8	-30.7	5829.8	7221.7
2	14463.4	15346.8	-18.1	6200.2	9181.2
3	16159.9	17043.3	-9.1	6425.6	10662.6
4	17008.2	17891.5	-4.5	6545.1	11576.7
5	17856.4	18739.8	0.0	6653.1	12101.5

Table A2-6: Total water volumes in the system.

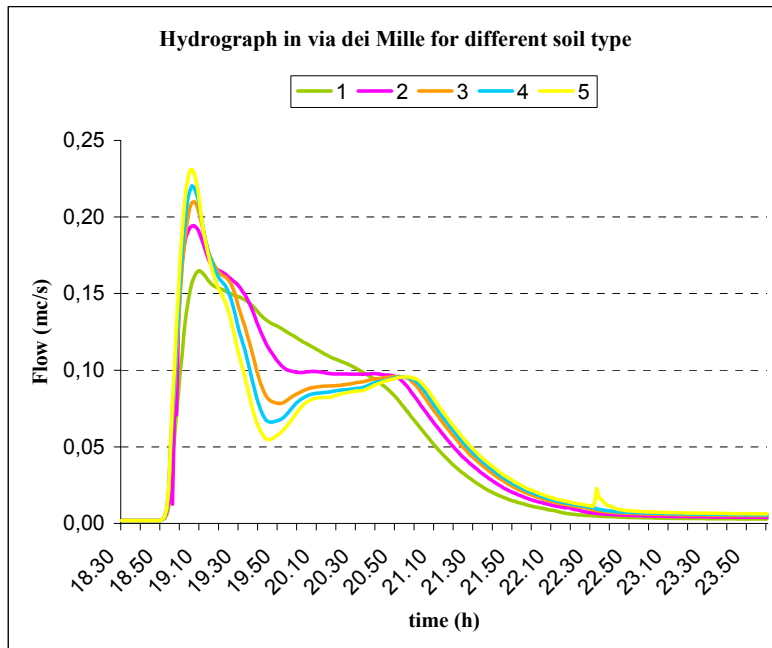


Fig. A2-19: Hydrograph in via dei Mille for different soil type.

4.2.1.2 Calibration

Parameters used for the calibration of the model for the Correggio sewer network are:

initial water losses (relative and absolute);
the weight coefficient f_i of the PR equation.

$$PR = (0.829 * PIMP) + (25 * SOIL) + (0.078 * UCWI) - 20.7$$

This equation was the result of regression analyses carried out on data from 510 storm events from 17 different catchments. This was based on a statistical approach and the equation explained 58% of the variation in the data with a standard error of 10.3%. In WaPUG User Note No 9(5) Ron Chapman explains the PR Equation and concludes that the PR Equation has many limitations and users must be aware of these if they wish to obtain realistic results. He goes on to state that users should make the effort to understand the significance of the parameters used. With low values of PIMP, SOIL or UCWI unreasonably low or even negative values of PR can be created. To overcome this the software sets a lower limit of 20% and an upper limit of 100%. With typical summer UCWI values it is necessary for the PIMP value to be generally be in excess of 40% for the PR value from the equation to be above the minimum 20%.

The overall runoff from a contributing area (to a single node) is quite complex with individual runoffs from the different surface types depending on the individual areas of each surface type and weighting factors with pervious areas only having 10% of the weighting of impermeable surfaces. This means that all surfaces (even permeable ones) produce runoff provided that the rainfall is in excess of the initial losses.

Rain event used for the calibration were registered in Via dei Mille the 28th of June 2000 and 7th July 2000. Between the 2 registration the system was changed: a new CSO was put in “Cavo

D'Argine". This change will influenced accordance between measured data and simulated data as presented in the following pages.

The rain event of 28th June 2000 was characterized by a duration of 2 hours and a return period of 2 years more or less.

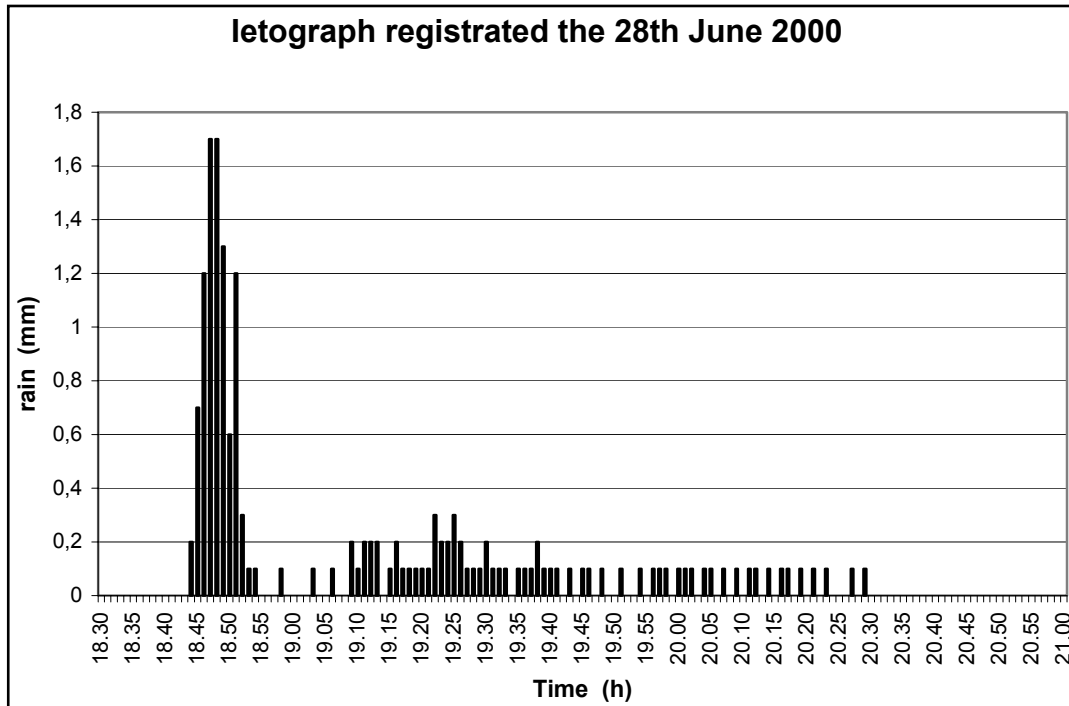


Fig. A2-20: Ietograph registered the 28th June 2000

Calibration Results

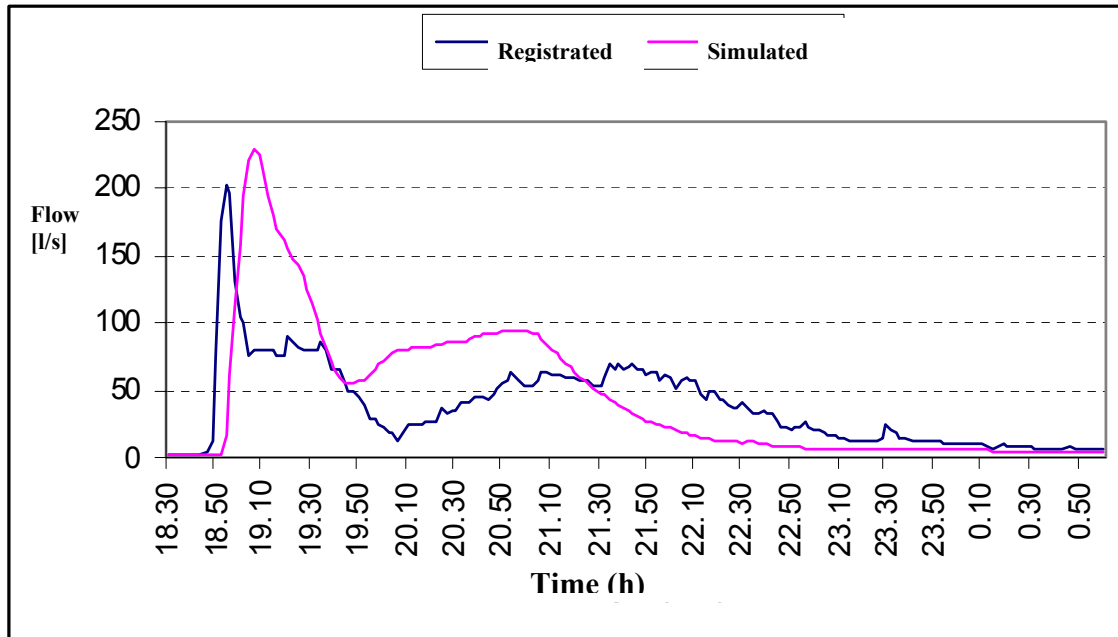


Fig. A2-21: Comparison between the Hydrograph simulated and registrated in Via dei Mille

It can be noticed that the results from InfoWorks are not so close to the results from measurements. Only the first part of the Hydrogram is satisfactory. Differences can be attributable to:

- in the simulation scheme there is a wastepipe more after the manhole of reference than in the situation in which measurements were taken, so in that section there's a bigger flow. That is the reason why the peak flow is over – estimated;
- the bigger flow affect the exhaustion time of the volumes passing: so the simulated wave has a less duration than the measured one.

Even if it was not possible to build the second part of hydrograph close to the measured one, we tried to guarantee the same volumes passing. Integrating the curves in time we obtain that the total volume passing during the measured event is 880 m^3 , instead for the one simulated is 1000 m^3 : so the result is acceptable.

4.2.1.3 Results

The simulations done can be useful to evaluate critical areas in the Correggio sewer network. During the simulations in addition to rain events also different flood waves were be taken in account with the same return time of the rain events. As flood wave we have considered a rectangular wave with a constant value on the peak with duration of 3 hours.

Results from simulations with different return period (2, 5 and 10 years) have been compared. To see immediately the results, for each simulation we prepared some Map/tables:

- volumes balance table;
 - FLOOD Map: the colour explain the difference between the “piezometric” on the manhole and the ground level. Circle diameter is proportional to the water flooding;
 - PIPE – FILLING Map: explain the maximum pipes filling level which help in understanding if the pipe is pressurized;
 - Q – Qmanning ratio Map: is the ratio between the maximum flow that flowed in the pipe and the flow that can flow in the pipe section completely full, in uniform flow conditions. That is important to know if the pressurized condition in caused by a pipe insufficient dimension or by a backwater condition coming from downstream.
- **SIMULATIONS WITH 2 YEARS RETURN PERIOD** and with a wave in the open channel with a 2 years return period.

We used:

- the Chicago hyetograph with a 2 years return period as project rainfall;
- rectangular hydrograph with constant peak value of $Q_2 = 1,85 \text{ m}^3/\text{s}$ ($Q_2 =$ flow with 2 years return period).

Volumes balance:

Total rainfall (mc)	168259.3
Total runoff (mc)	43337.5
Total inflow (mc)	64195.7
Total outflow (mc)	60596.5

Tab. A2-7: Volume balance

Also for a 2 years return period event, there are flooding areas in the network (area B,F,E and especially G).

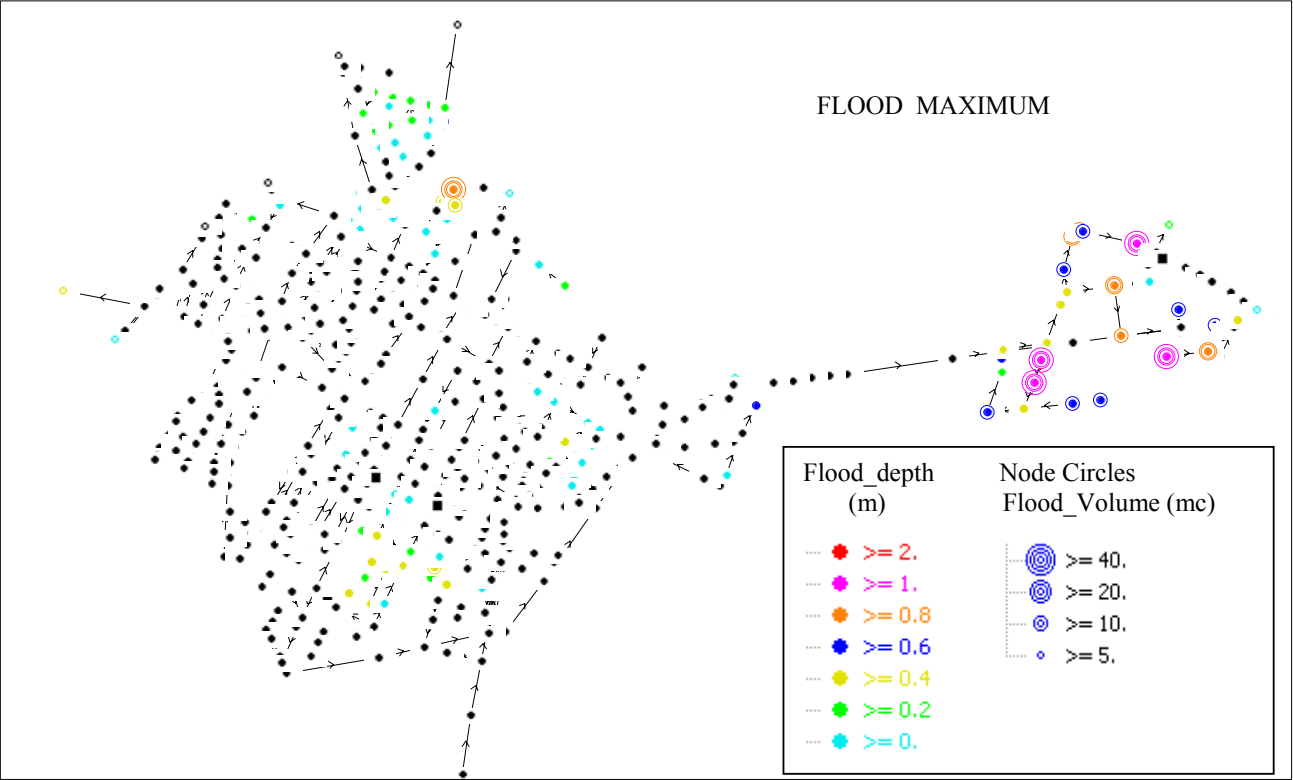


Fig. A2-21: Flood Maximum: 2 years return period and with a wave in the open channel with a 2 years return period.

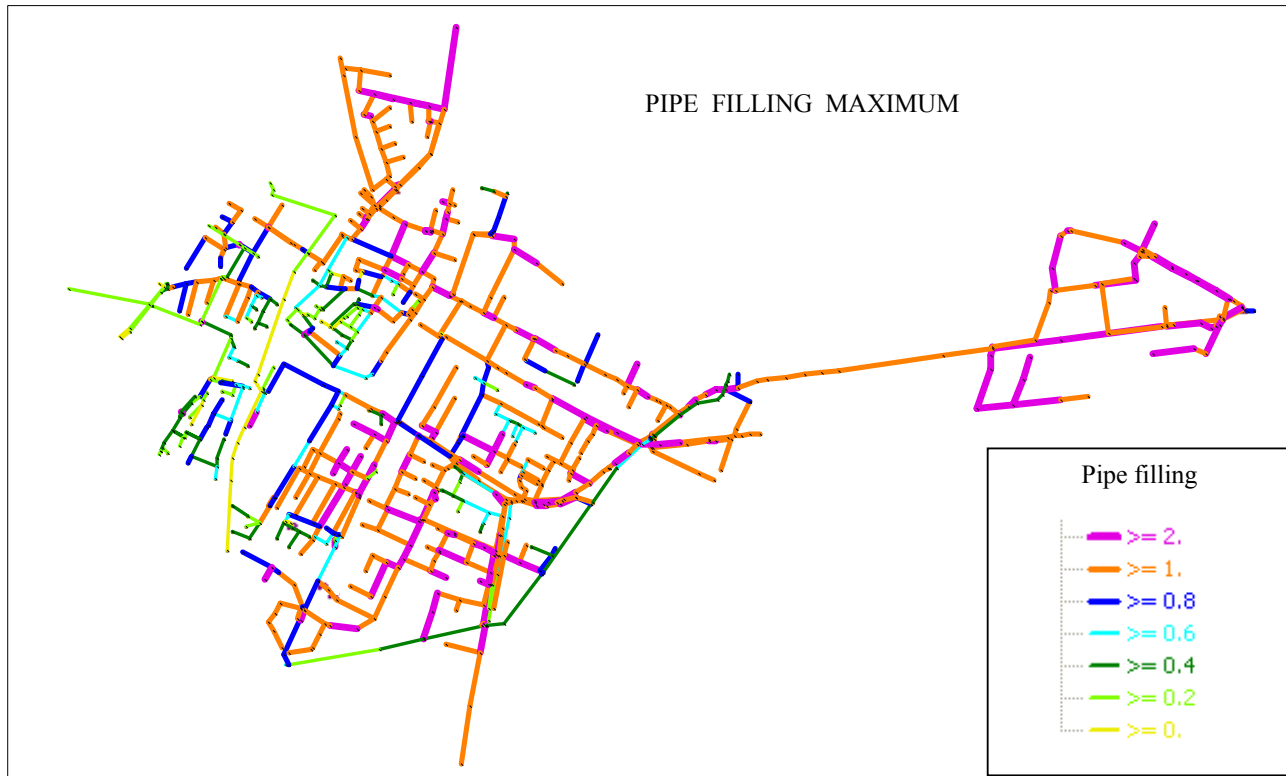


Fig. A2-22: Pipe Filling Maximum: 2 years return period and with a wave in the open channel with a 2 years return period.

The most part of pipes are pressurized.

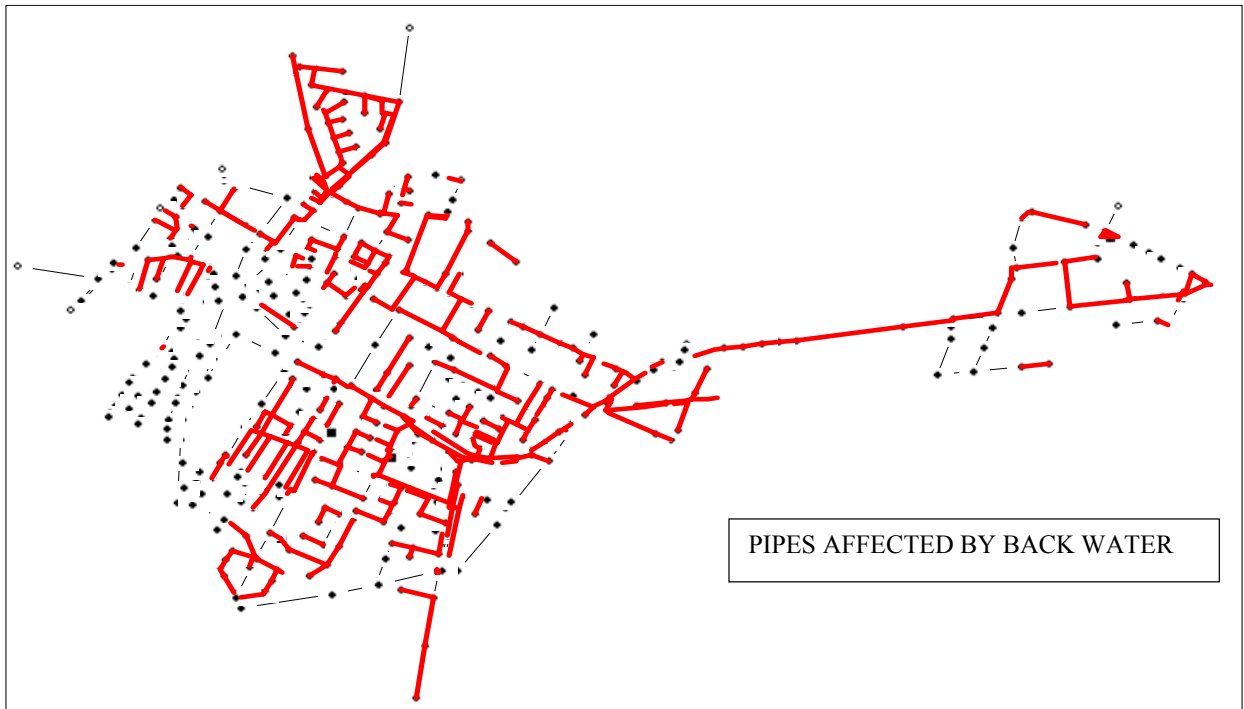
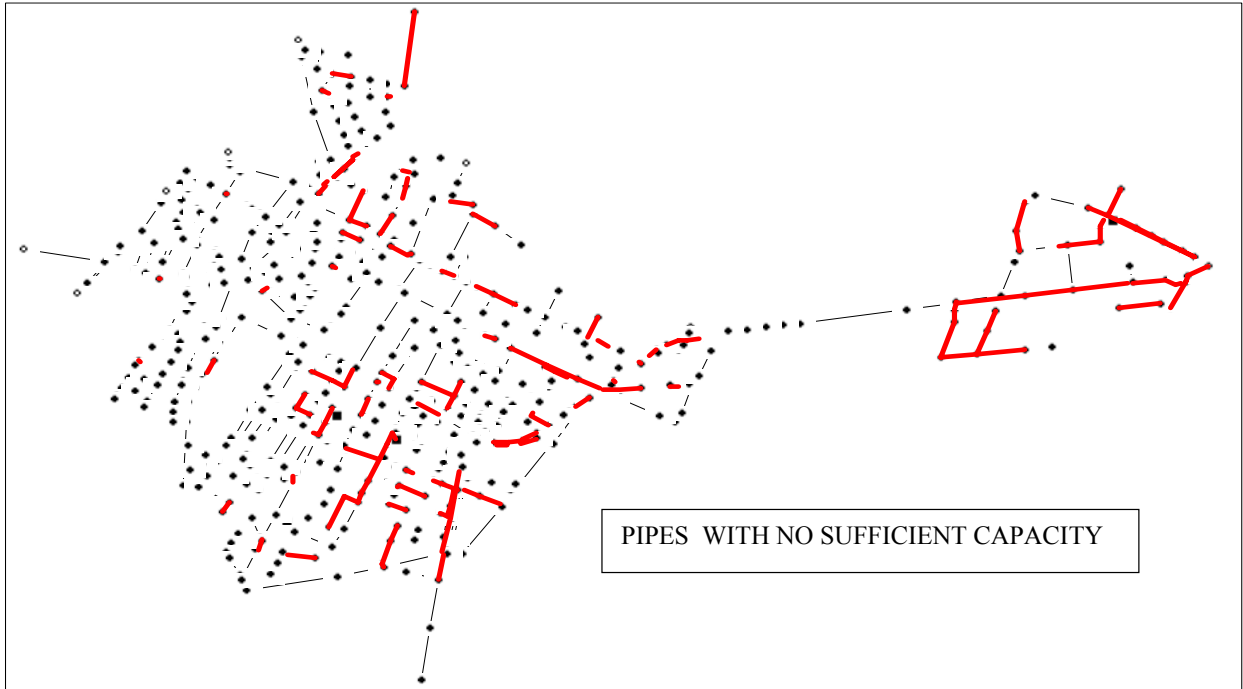
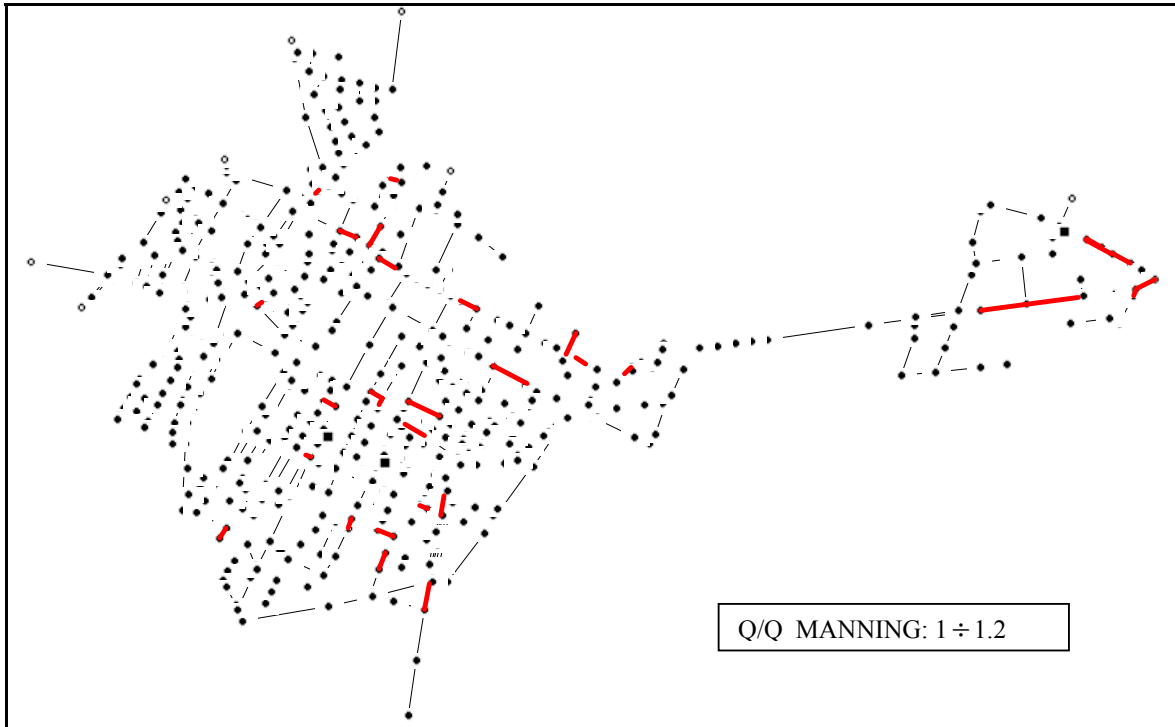
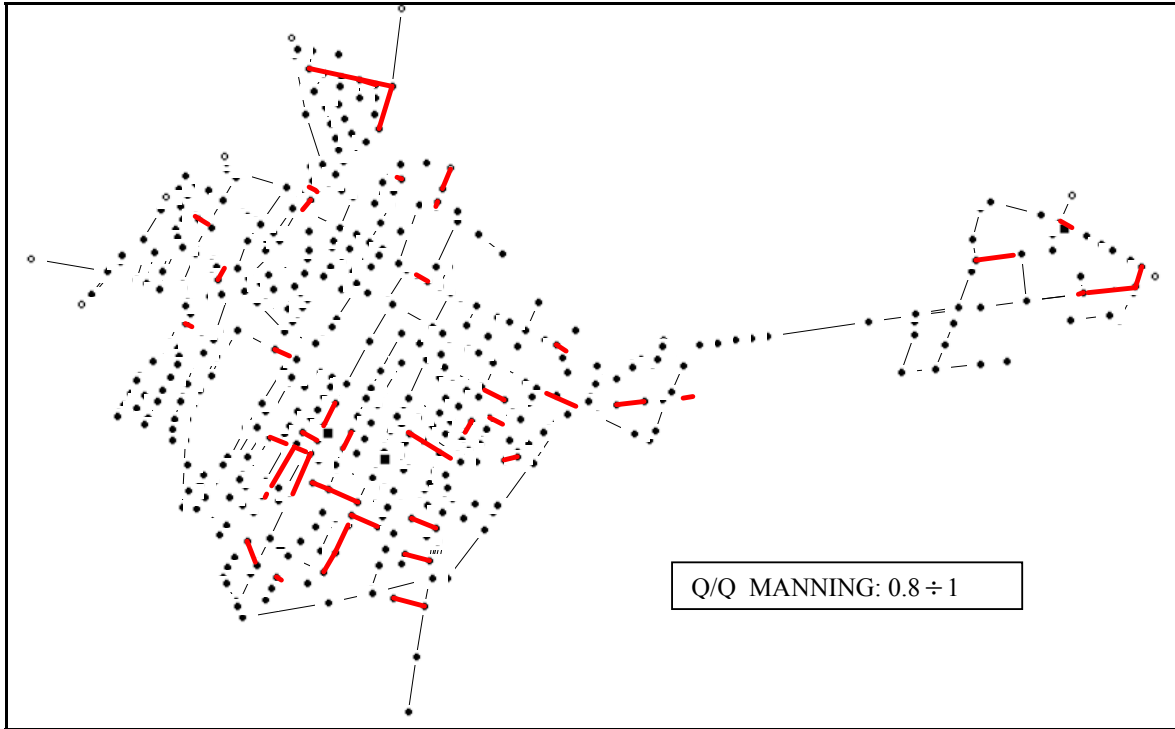
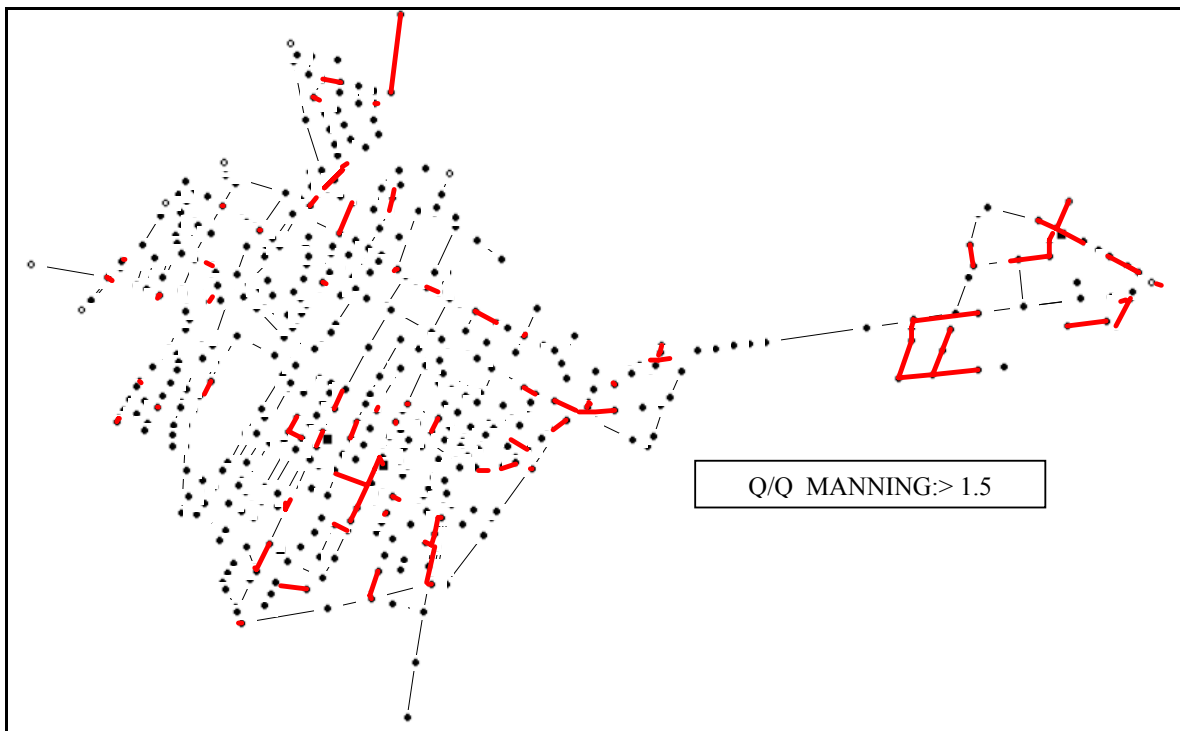
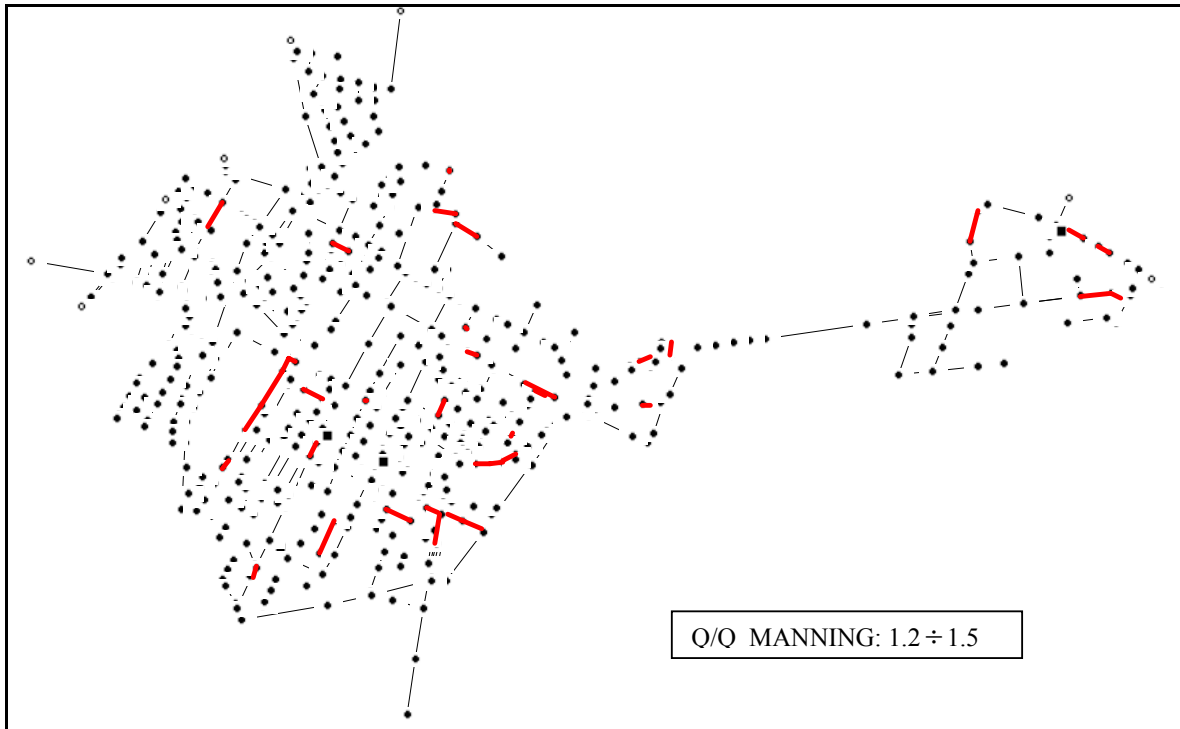


Fig. A2-23: Pipes with no sufficient capacity or affected by backwater: 2 years return period and with a wave in the open channel with a 2 years return period.





The majority of the pipes can transport the flow generated from this rainfall event, but the backwater from downstream produces pressurized conditions and overflows.

- **SIMULATIONS WITH 5 YEARS RETURN PERIOD** and with a wave in the open channel with a 5 years return period.

We used:

the Chicago hyetograph with a 5 years return period as project rainfall;

Rectangular hydrograph with constant peak value of $Q_5 = 2,90 \text{ m}^3/\text{s}$ ($Q_5 =$ flow with 5 years return period).

Volumes balance:

Total rainfall (mc)	228679.6
Total runoff (mc)	59507.8
Total inflow (mc)	91864.8
Total outflow (mc)	86050.9

Tab. A2-8: Volume balance

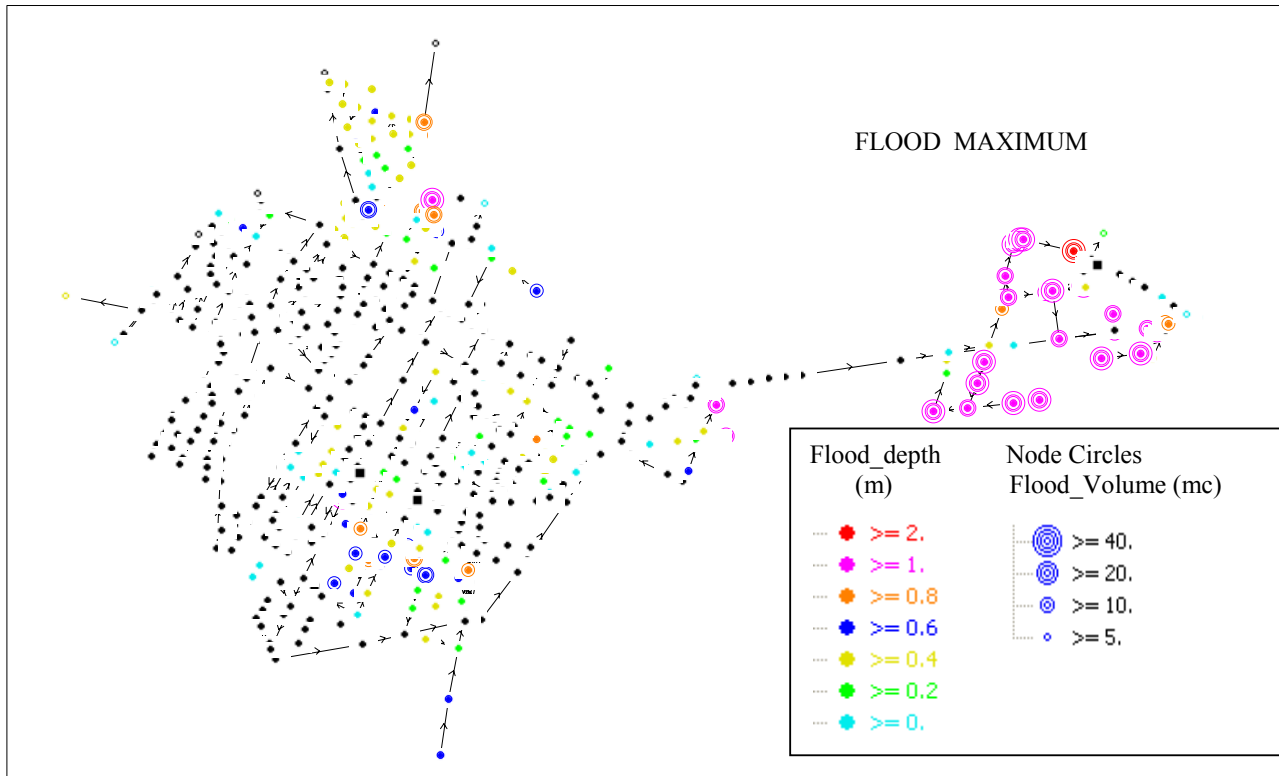


Fig. A2-24: Flood Maximum: 5 years return period and with a wave in the open channel with a 5 years return period.

From FLOOD results it can be noticed that the parts subject to overflows increase in considerable way: besides areas B,F,G and E also areas C and D start having overflow phenomena.

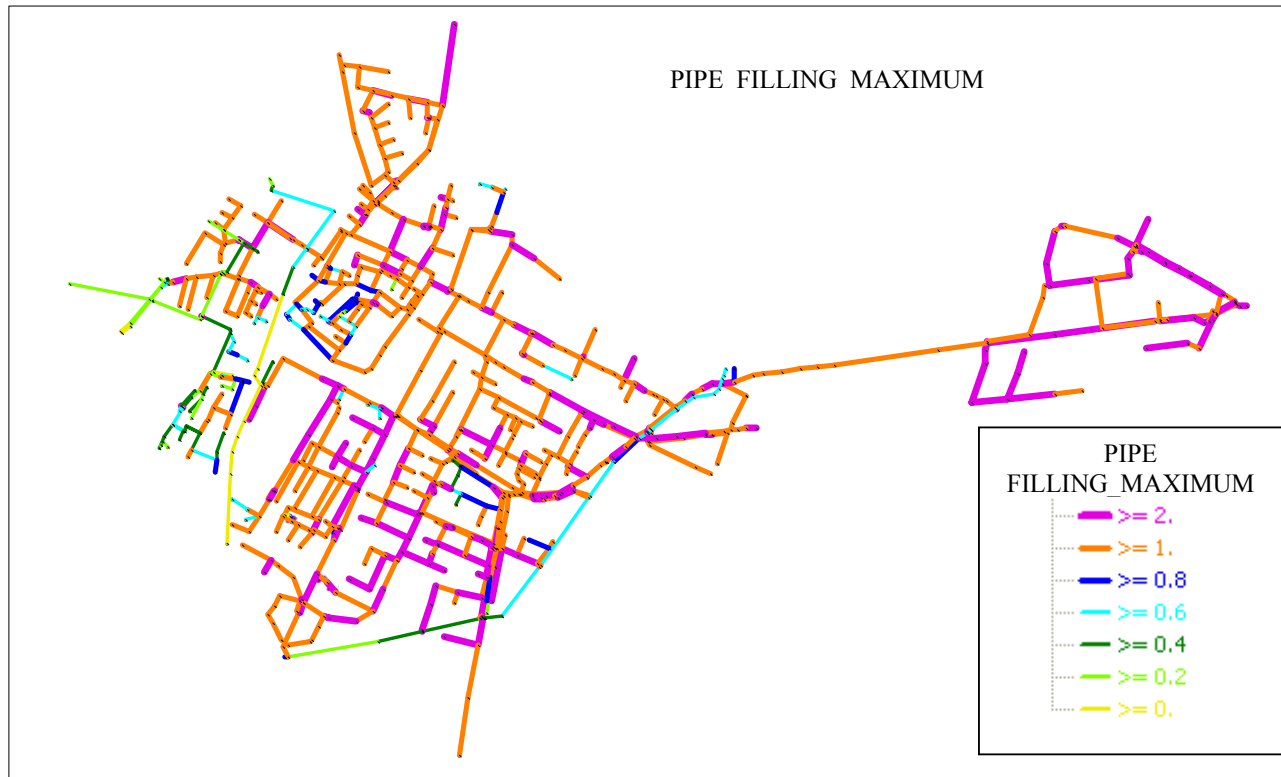


Fig. A2-25: Pipe Filling Maximum: 5 years return period and with a wave in the open channel with a 5 years return period.

The most parte of the pipes is pressurized.

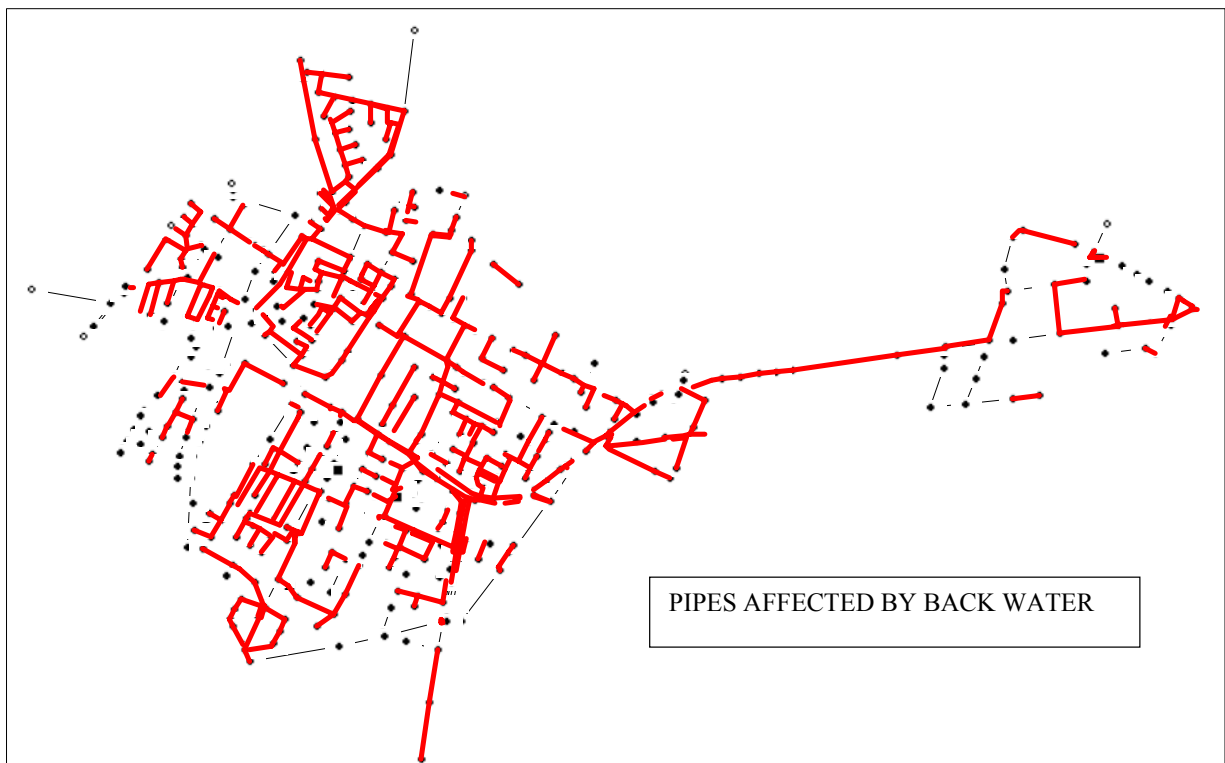
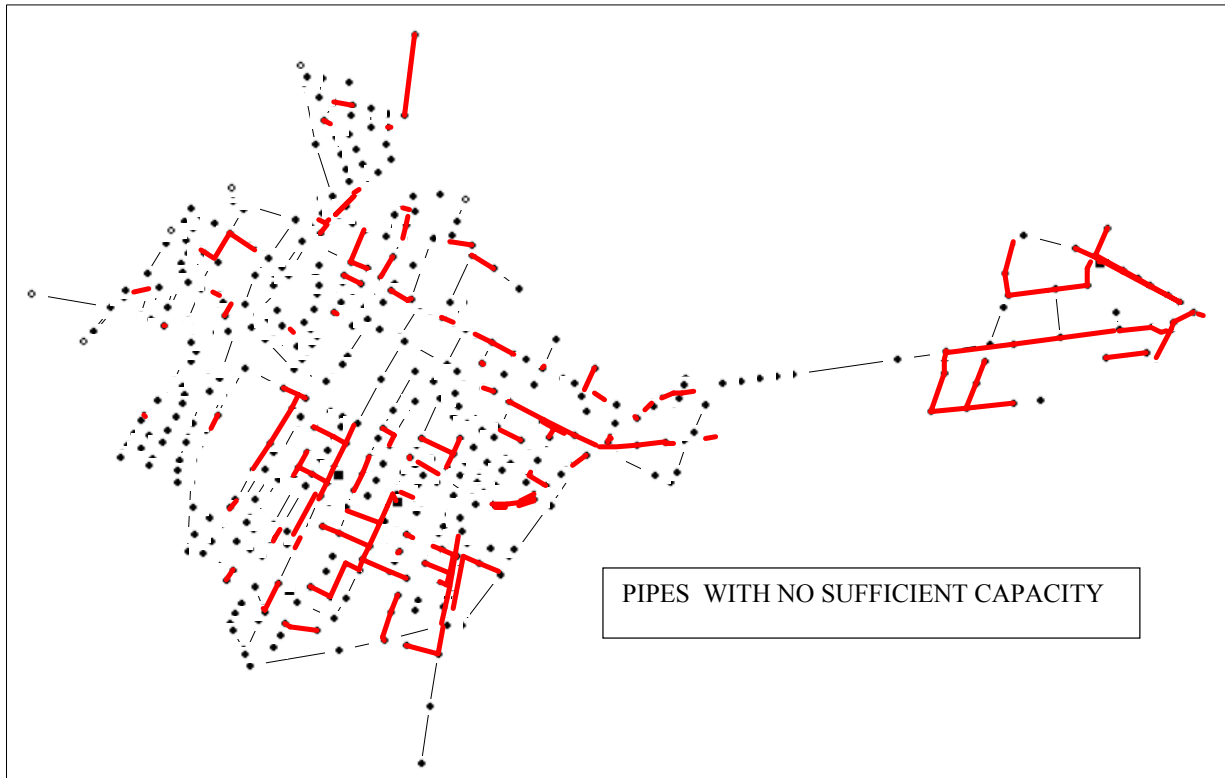
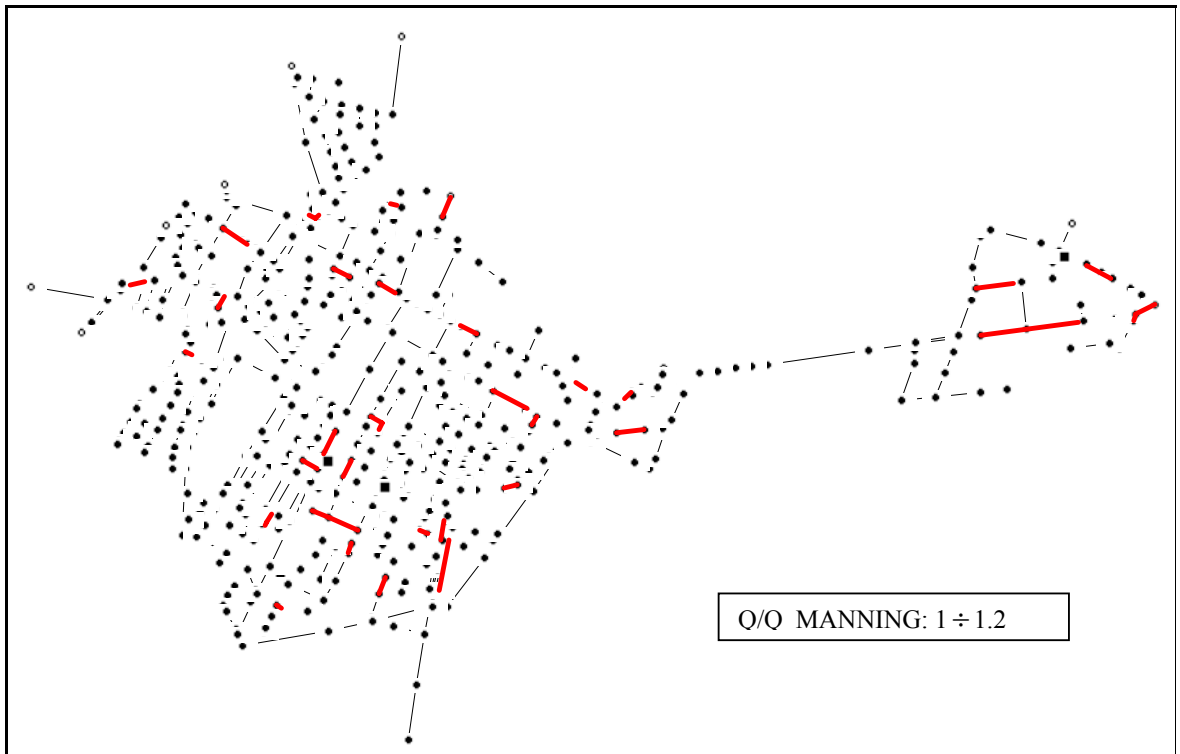
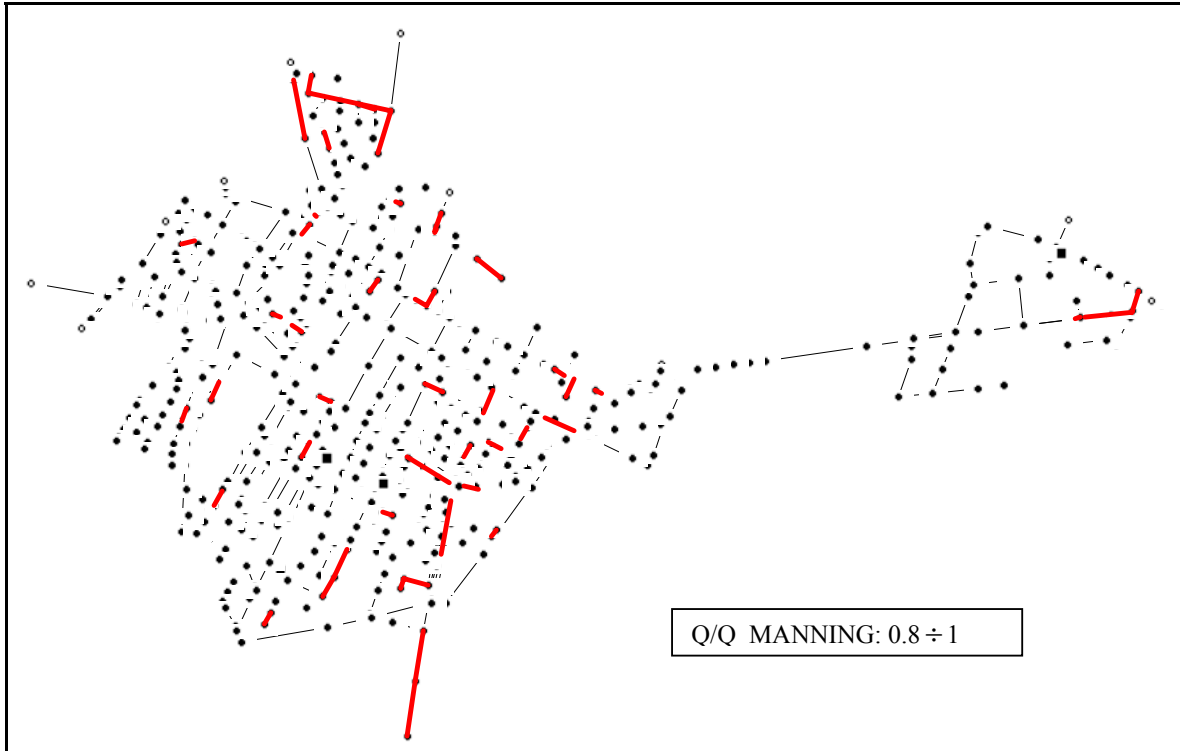
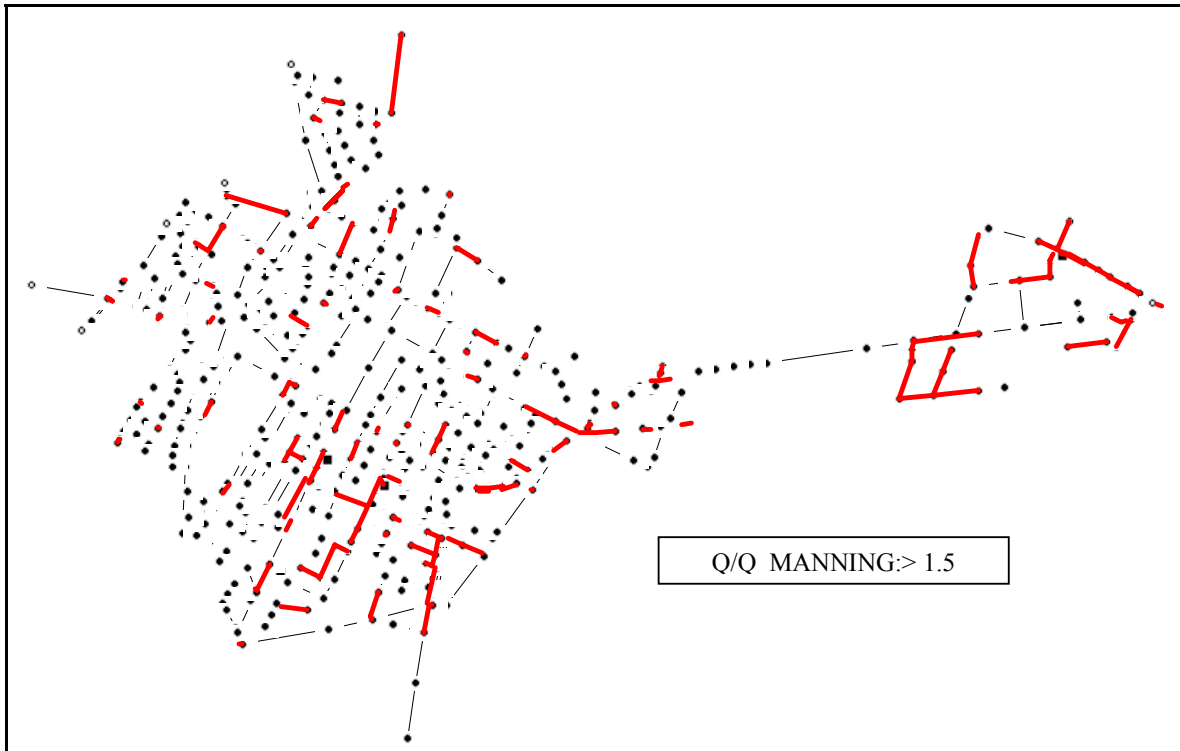
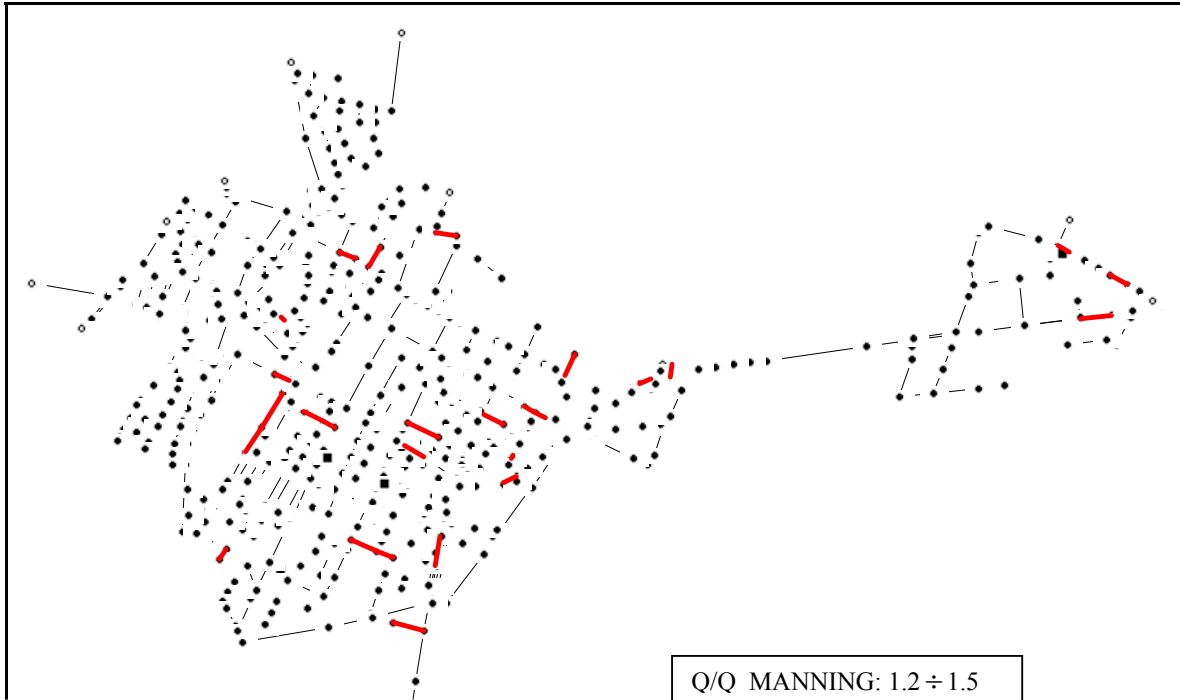


Fig. A2-26: Pipes with no sufficient capacity or affected by backwater: 5 years return period and with a wave in the open channel with a 5 years return period.





The number of pipes subject to backwater phenomenon are increasing.

- **SIMULATIONS WITH 10 YEARS RETURN PERIOD** and without a wave in the open channel.

We used:

the Chicago hyetograph with a 10 years return period as project rainfall;

Volumes balance:

Total rainfall (mc)	274261.7
Total runoff (mc)	71706.8
Total inflow (mc)	72279.7
Total outflow (mc)	66248.9

Tab. A2-9: Volume balance

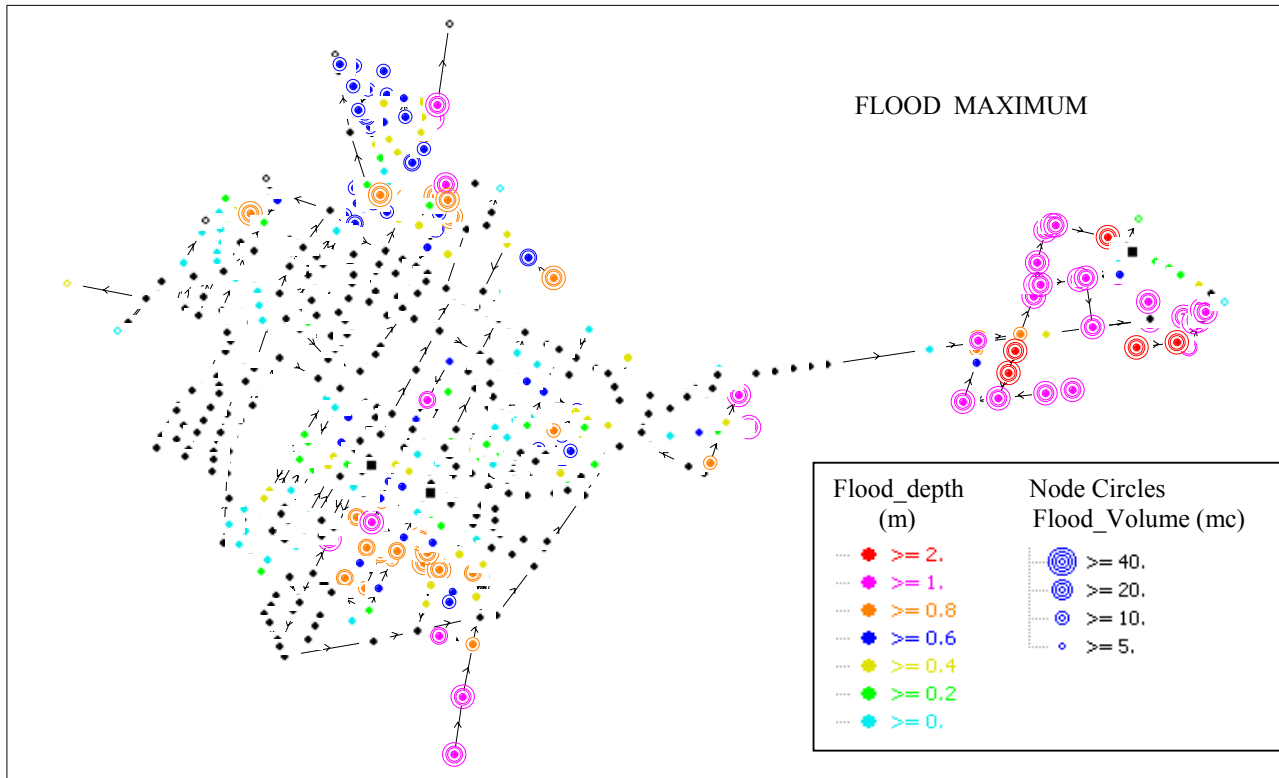


Fig. A2-27: Flood Maximum: 10 years return period and without wave in the open channel.

Comparing FLOOD results with the map of historical overflows of the network, we can notice that the model reflects in a satisfying way the real situation.

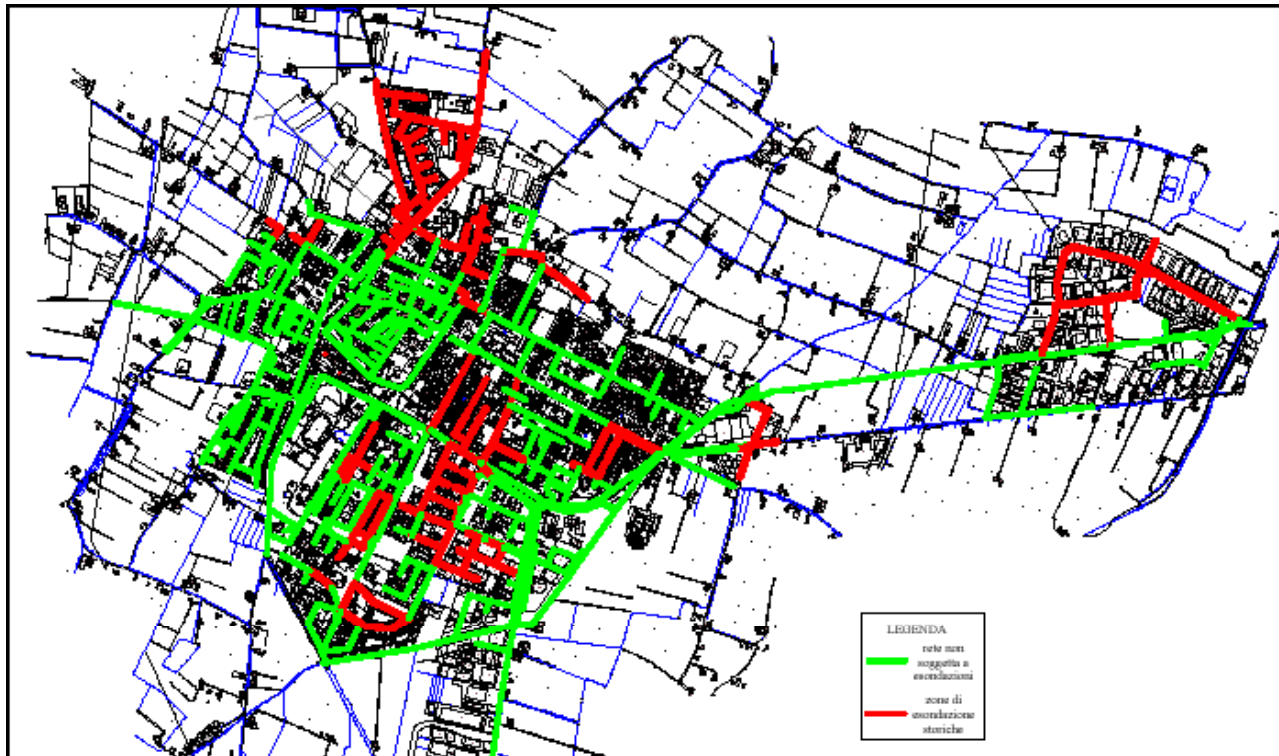


Fig. A2-28: map of historical overflows of the network

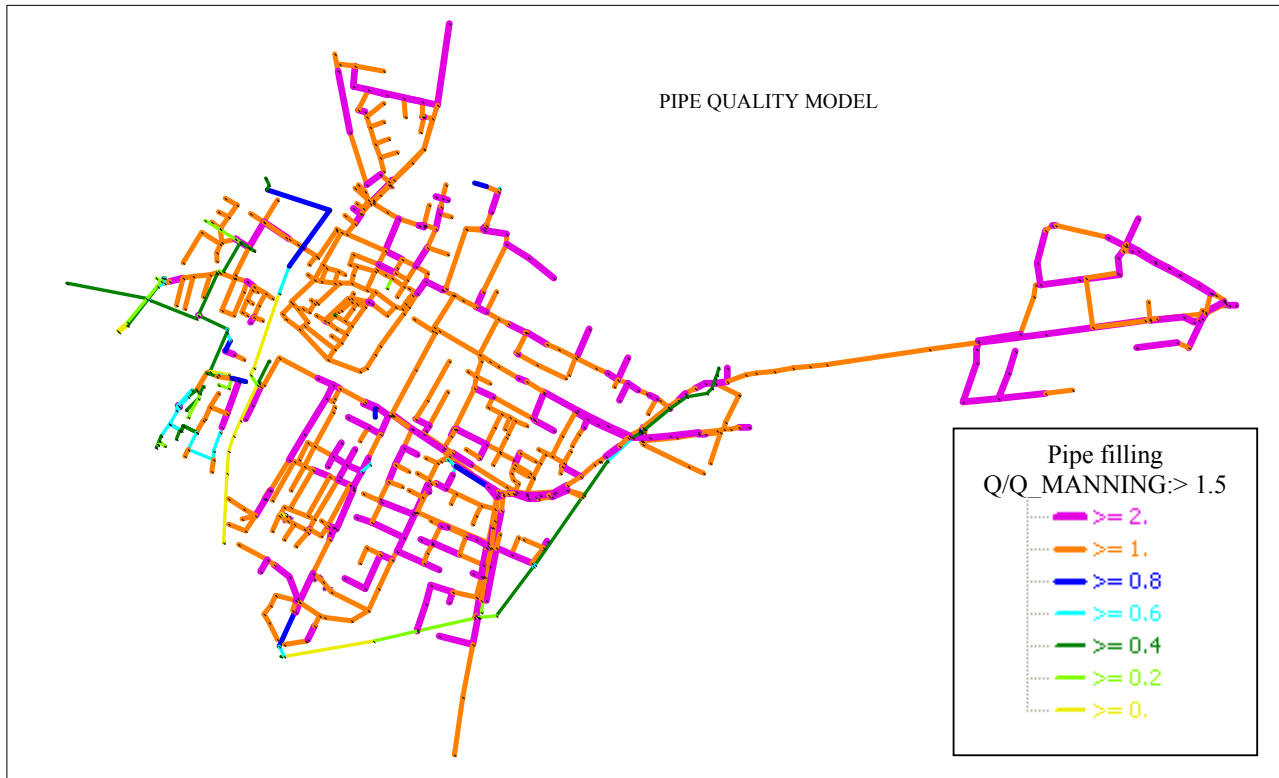


Fig.A2-29: Pipe filling Maximum: 10 years return period and without wave in the open channel.

PIPE – FILLING results indicate that only the open channels are not pressurized.

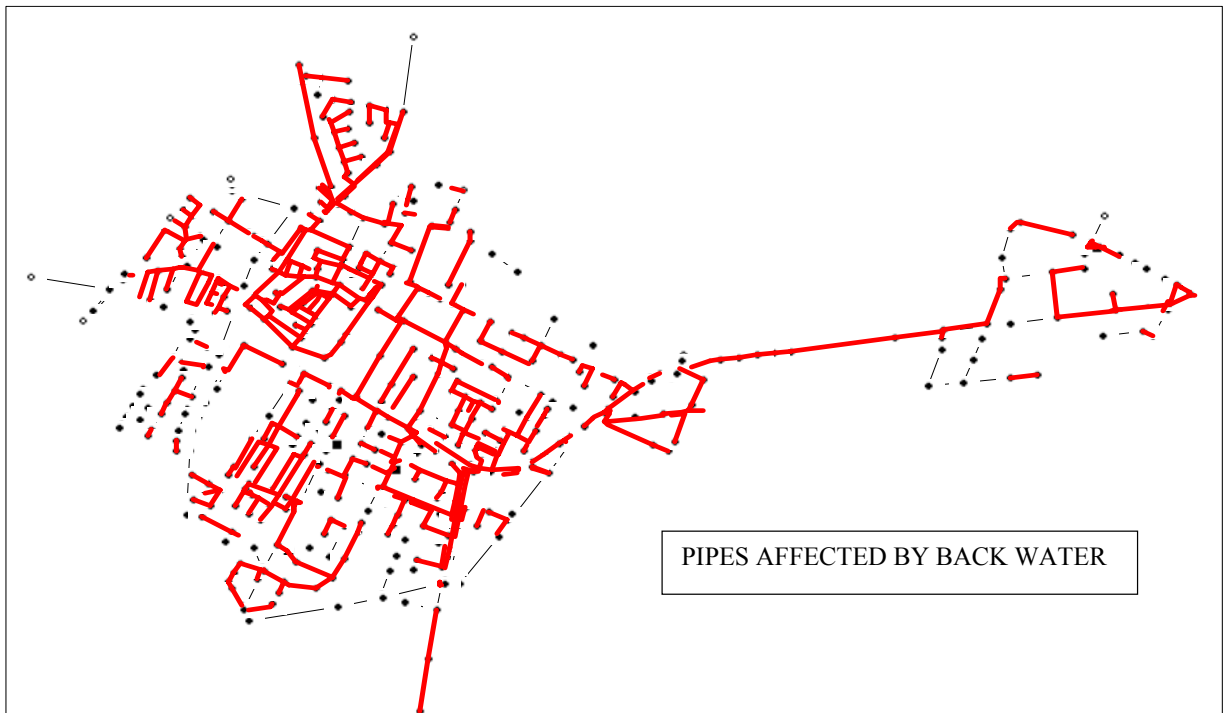
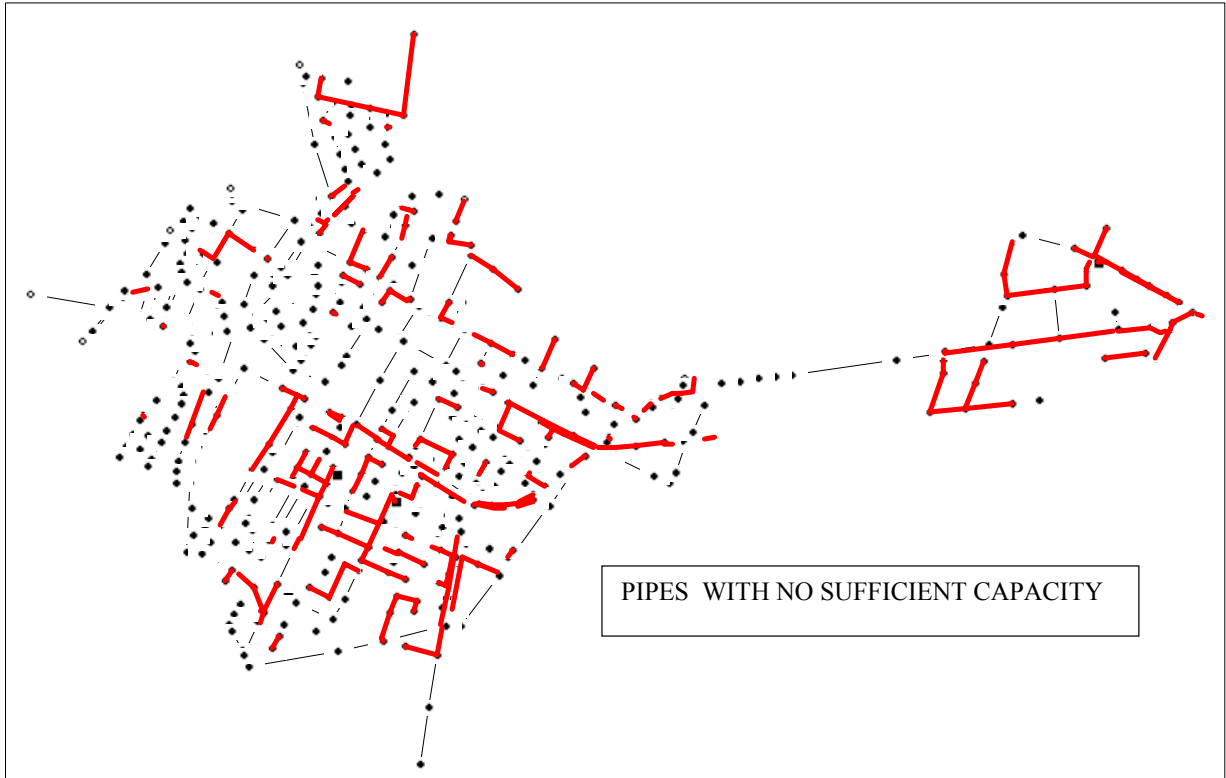
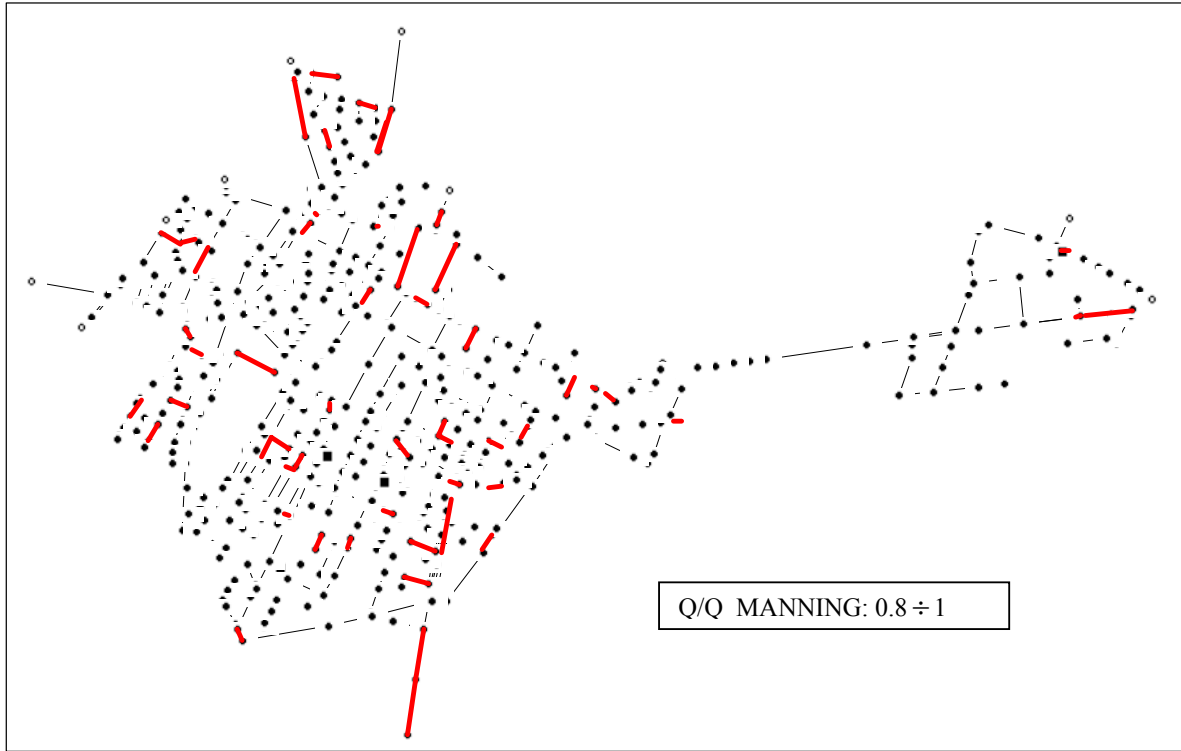
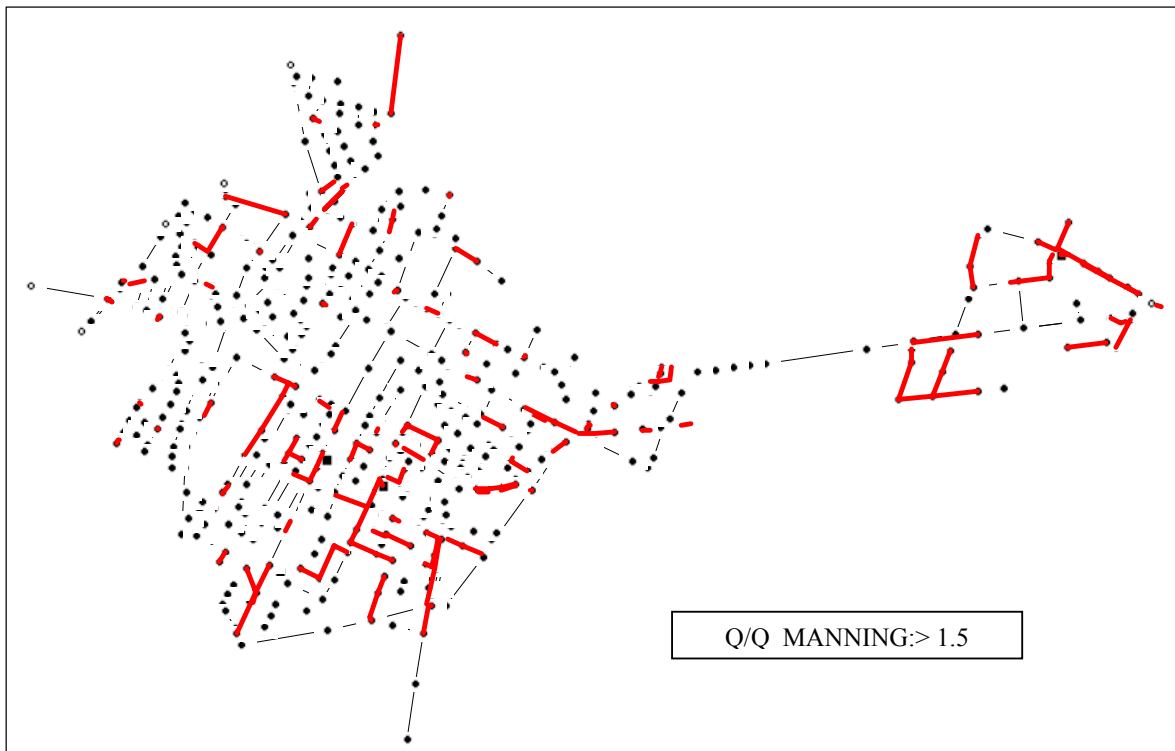
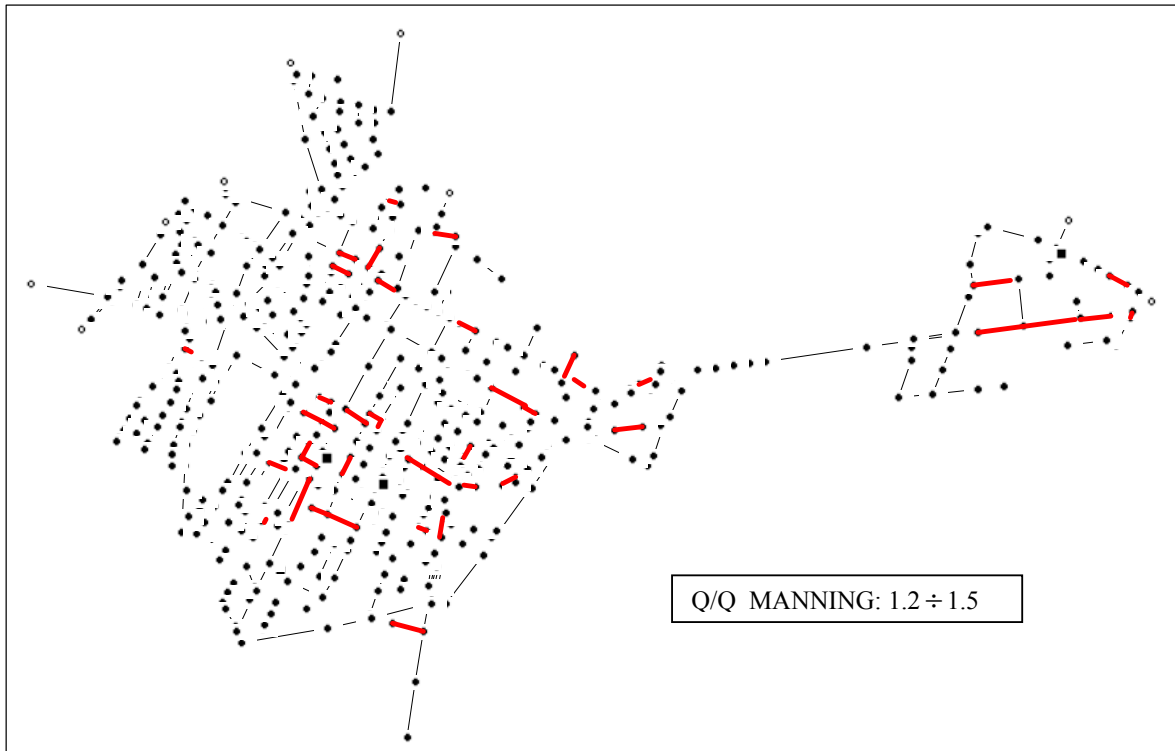


Fig. A2-30: Pipes with no sufficient capacity or affected by backwater: 10 years return period and without wave in the open channel.





The most part of the network presents backwater conditions.

- **SIMULATIONS WITH 10 YEARS RETURN PERIOD** and with a wave in the open channel with a 10 years return period.

We used:

the Chicago hyetograph with a 10 years return period as project rainfall;
 rectangular hydrograph with constant peak value of $Q_{10} = 3,70 \text{ m}^3/\text{s}$ (Q_{10} = flow with 10 years return period).

Volumes balance:

Total rainfall (mc)	274261.7
Total runoff (mc)	71706.8
Total inflow (mc)	112831.7
Total outflow (mc)	105530.5
Total lost (mc)	683.9

Tab. A2-10: Volume balance

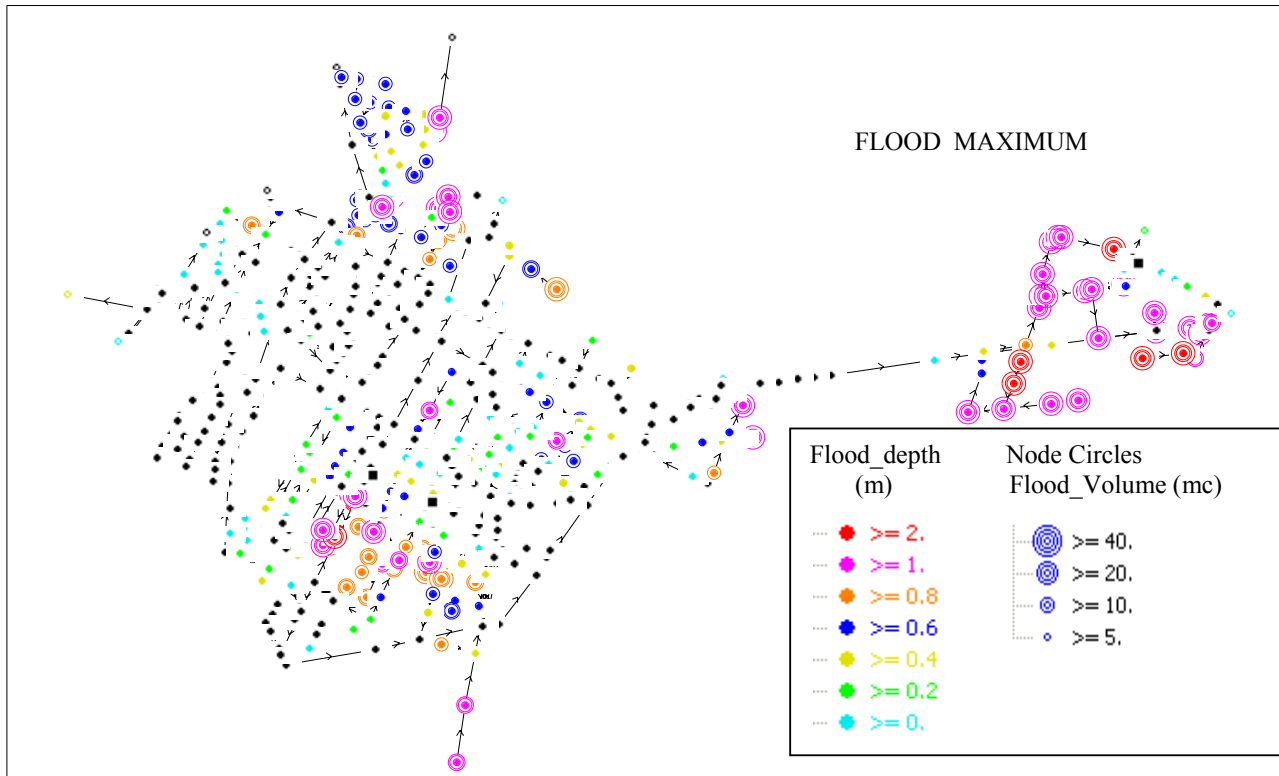


Fig. A2-31: Flood Maximum: 10 years return period and with a wave in the open channel with a 10 years return period.

FLOOD results explain that the zones interested from overflow are the B, C, D (not the south zone) E, F and G, while the zone A (the historical center) resists also with this return period (only a pair of overflows).

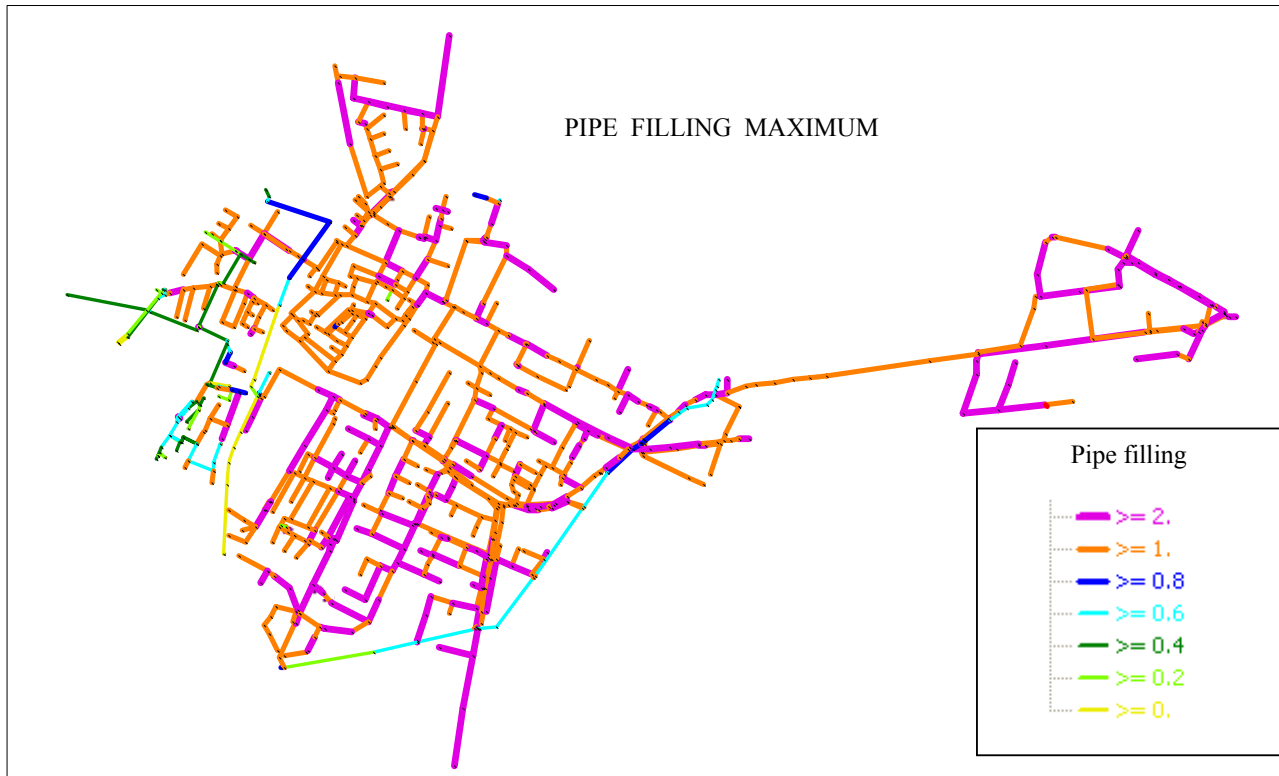


Fig. A2-32: Pipe filling maximum: 10 years return period and with a wave in the open channel with a 10 years return period.

From PIPE – FILLING results it can be noticed that the only pipes that don't work in pressure are the open channels and some pipes of the D zone (the south part of the network).

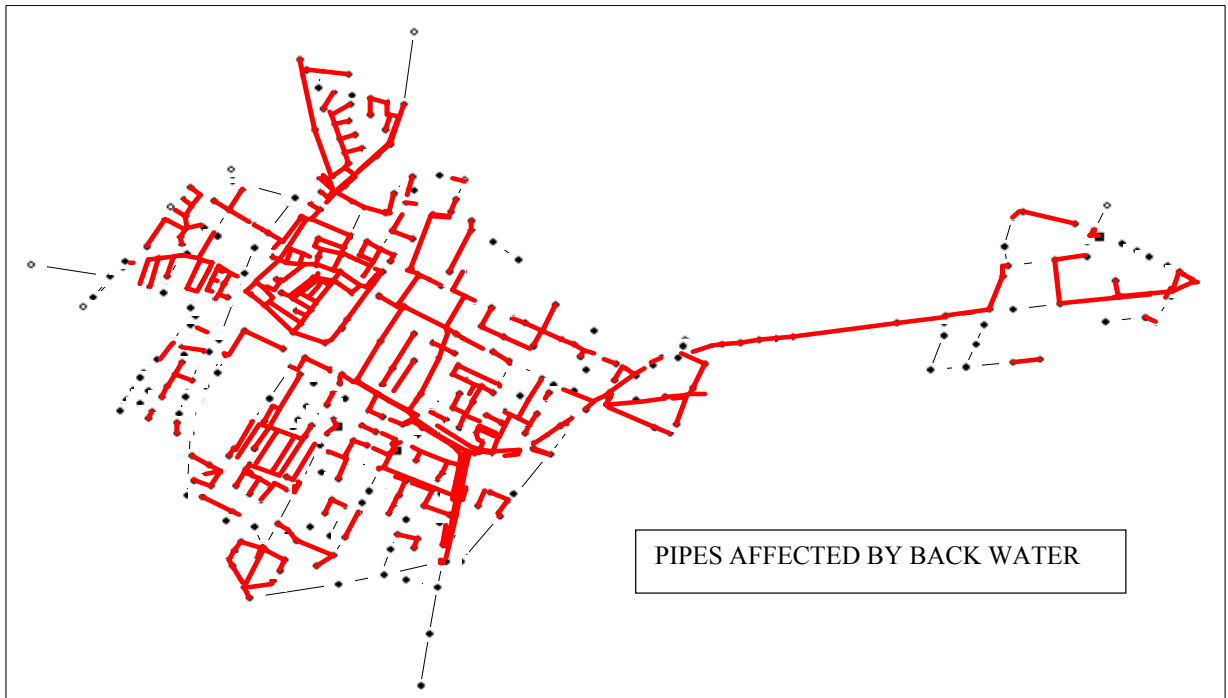
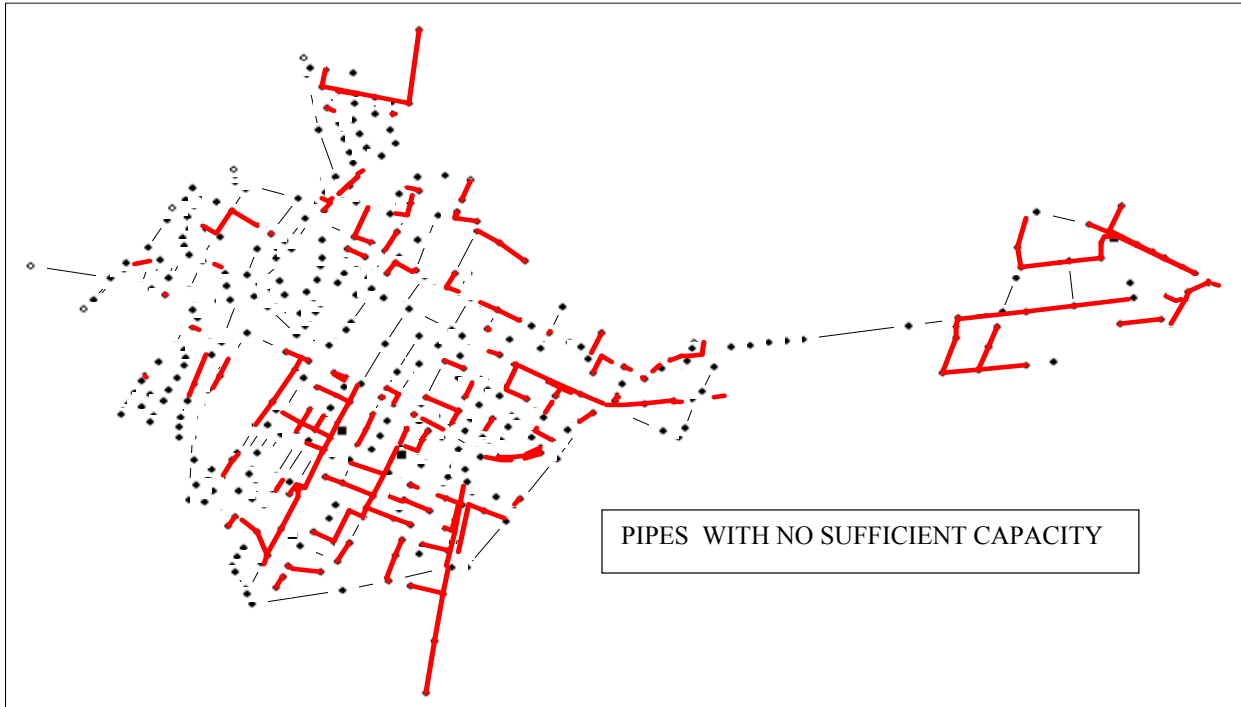
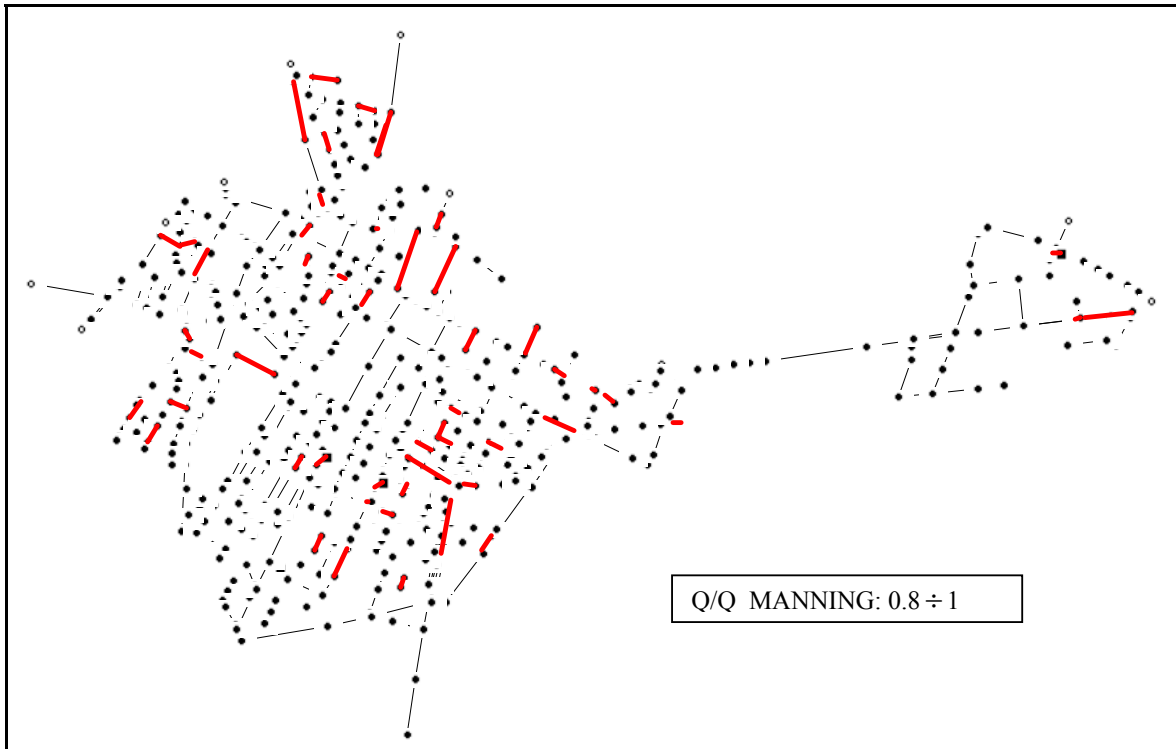
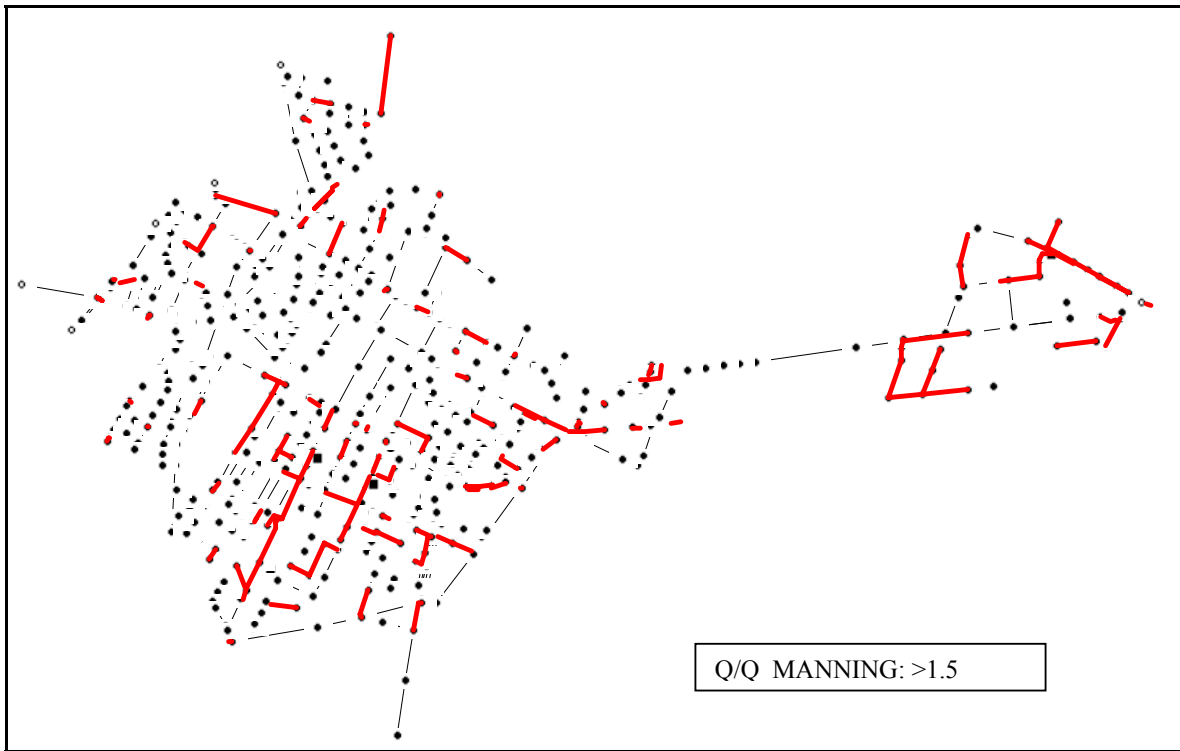
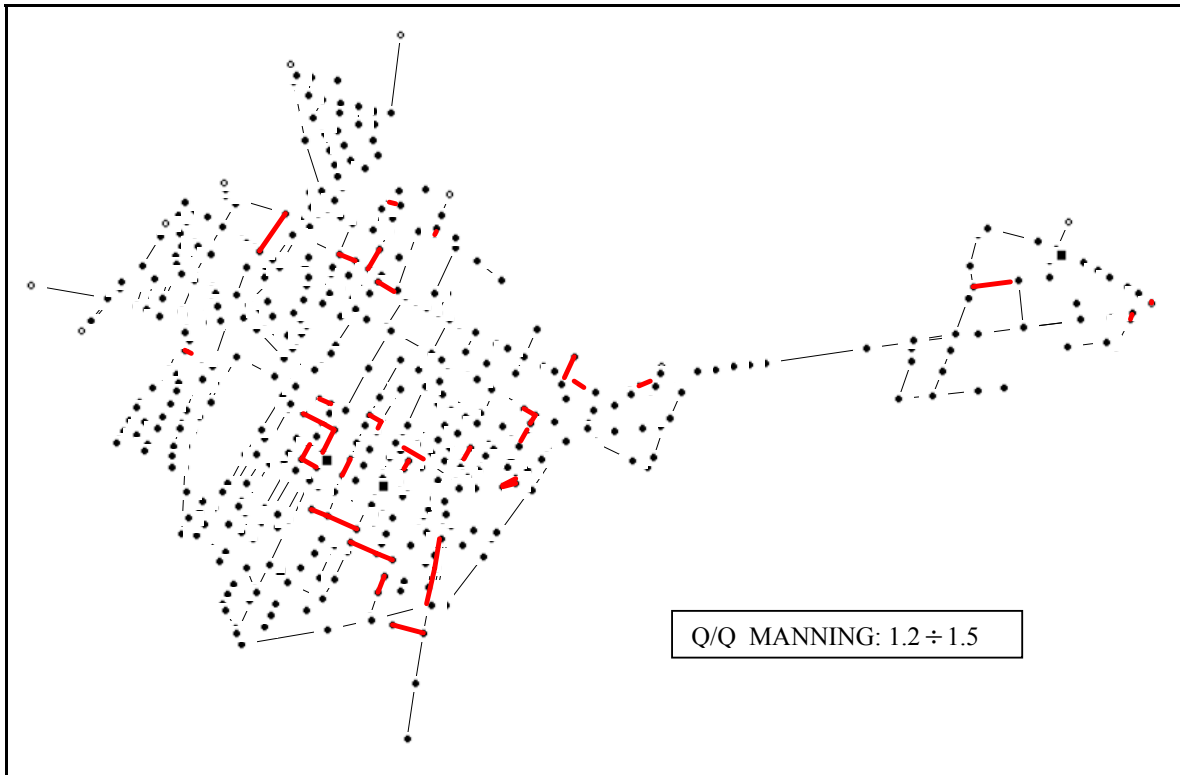


Fig. A2-33: Pipes with no sufficient capacity or affected by backwater: 10 years return period and with a wave in the open channel with a 10 years return period.





4.2.1.4 Conclusions

The scope of the present study has been to estimate the operation of urban drainage system of Correggio (RE). This network, in the last years, has demonstrated serious and diffuse hydraulic insufficiencies that prevent to the effective water-drainage of meteoric waters and the transport of sanitary sewer flow to the system of WWTP of Carpi. The carried out job can be subdivided in three main parts: the codification of the sewer network to the calculation code, the calibration of the model preceded from one sensitivity analysis and the definition of some proposals apt to improve the total operation of the net.

The analysis of sensitivity of the parameters of the Hydrologic - hydraulic model has concurred to estimate the incidence of the variations of every parameter on the final results and to identify in those linked with impervious surfaces the most important coefficient for the calibration of the model.

The calibration of the mathematical model of the network carried out through the comparison between data of the critical historical situations supplied from the Municipality and those obtained through monitoring campaigns, has happened with difficulties: the phenomena of regurgitation that happened in phase of recording and the different configuration of the simulated net respect the one monitored, gave the reconstruction of the first part of the Hydrograph and the computation of the volumes with a sufficient precision, while it was not possible to reconstruct the shape of the wave of flood in the final phase of exhaustion.

It is proceeded subsequently to the simulation of the behavior of the net using of project rain of "Chicago" having different return period to characterize the zones hydraulically more critics; results have put in evidence that , already for events of return period of 2 years, take place the first phenomena of overflow. For this event more of 30% of the net works in pressure and approximately 10 % of it has insufficient sections for the flow transport.

With a return period of 5 years 80% of the net is in pressure and the flooding zones practically coincide with those in which historically overflow phenomena have happened; those results constitutes a verification of the reliability of the model.

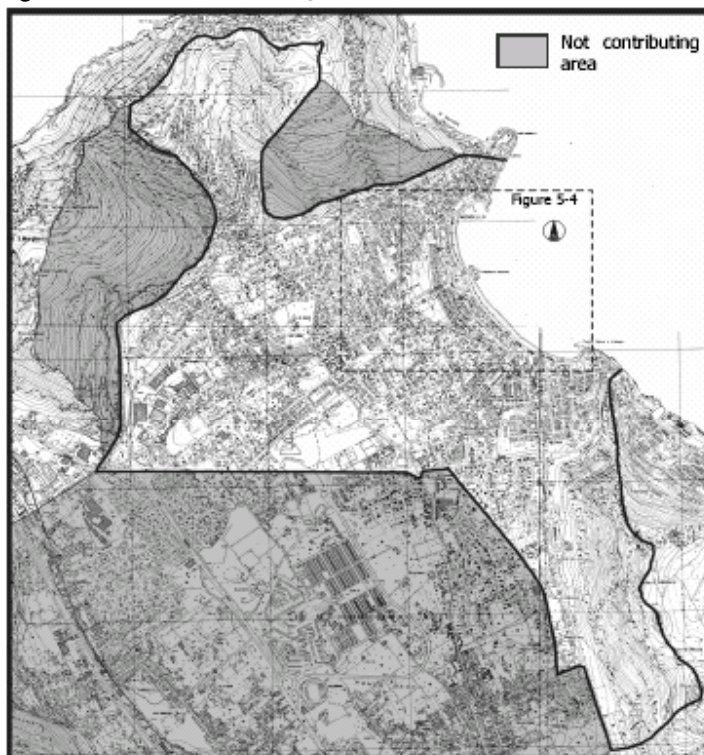
The simulations have underlighted the negative influence of the water level in the channels, being increased the overflow events where the level of the drainage is close to the level of the bottom of the channel. The results of the simulations supply however an interesting source of inspiration for eventual project plans finalized to the reduction of critical situations: rehabilitation plan for insufficient pipes, bound together to the realization of lamination basin, would have to reduce the maximum flow within the insufficient pipes, in particular the end pipes of the zone E of the industrial village in which the water transport is entrusted to pump systems.

4.3 A3 The Mondello catchment case study (Palermo – Italy)

The contributing watershed of Mondello is approximately 25 km². Until the beginning of the past century, the analyzed area was a semi-rural zone, exclusively covered by green surfaces and modest fishermen villages. During the past century, it has been progressively transformed in tourist and bathing area, with a strong urban expansion. Due to this, the analyzed area shows a remarkable variety of building types: single family houses surrounded by small gardens as well as block buildings with scarce pervious areas.

Remarkable differences can be found also when considering the characteristics of the semi-rural areas: the “Pedemontana” zone, in the Northern part of the catchment, has remarkably high slopes while the “Piana dei Colli” low slopes characterize zone, in the South – West part of the catchment. During the last thirty years, fast urbanization processes haven’t been coupled with the construction of the needed drainage systems aimed to collect stormwater. During the rainfall events, therefore, the runoff volumes mainly propagate along the roads, and remarkable flooding is generated in depressed areas, where the water volumes remain up to some days.

Figure 6.3.1: *Mondello contributing catchment overview*



The tourist relevance of the catchment and the strong urbanization of its downstream portion make the adoption of concentrate mitigation measures an unsuitable solution: Mondello thus seems therefore to be an ideal case study in order to estimate the efficiency of set of distributed mitigation interventions.

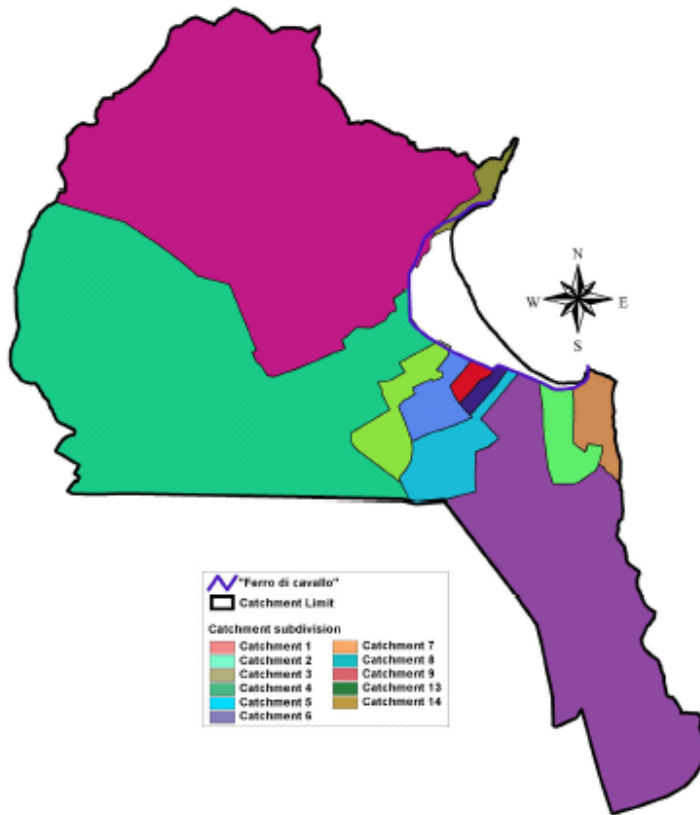
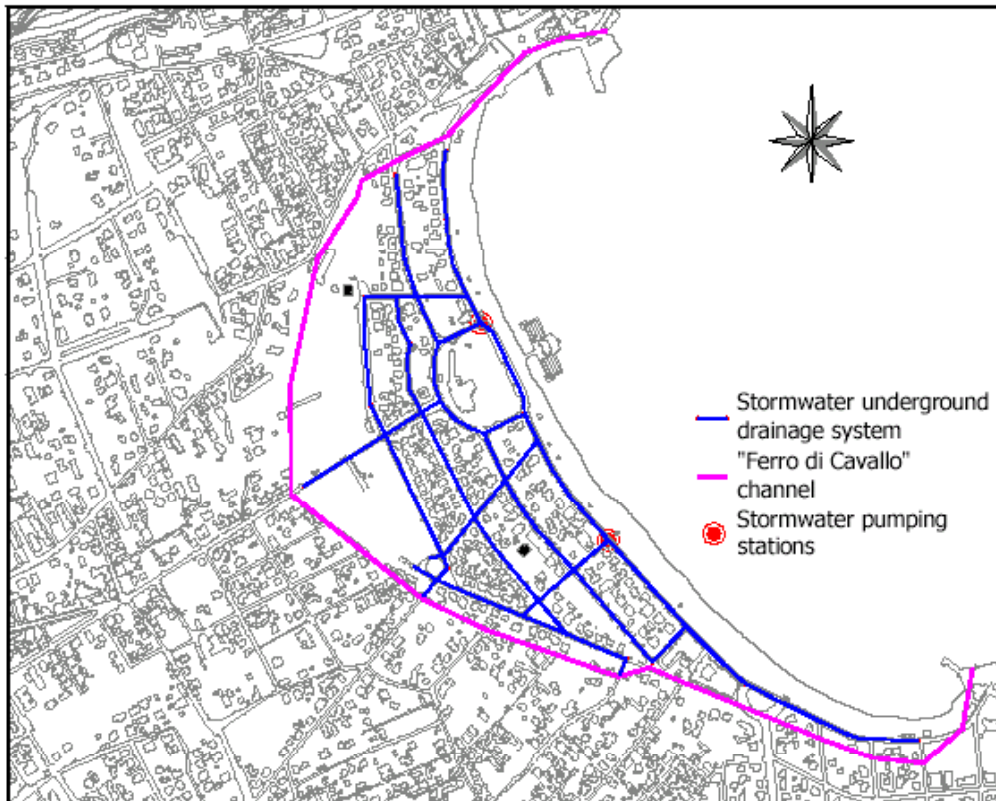


Figure 6.3.2: Independent catchments subdivision

An extended portion of the semi-rural catchment surrounding the urbanized area of Mondello doesn't contribute to the runoff formation: this is basically caused by the detention and the infiltration in pervious areas, by the flooding of many upstream zones during rainfall and by the characteristics of the drainage systems which serve the single areas. In most cases, in fact, these sewers have been built as the sum of single interventions, trying to solve the local particular problems, without an organic planning and design. The lack of a clear planning and management of drainage systems has led also to a remarkable deficiency in the available information regarding the consistency of the realized works, often carried out by different municipal administrators. The collected information about system geometry has been obtained by surveying the municipal offices and by in situ inspections.

The effectively contributing surface is approximately 8 km² wide; it can be divided into 11 independent catchment, whose drainage system is simply represented by the roads (Figure 6.3.2).

Figure 6.3.3: Mondello seaside area drainage system



The 11 catchments runoff volumes concur in a strongly urbanized area near to the sea where the greater flooding and the greater damage for population and tourist activities take place, also during high frequency rainfalls.

Such area is the only part of the catchment that is served by an underground stormwater drainage system; specifically, it's a looped network constituted by 12 loops, with cast iron pipes having a 500 mm diameter. An ancient groundwater drainage channel, the so-called "Ferro di Cavallo" (Horseshoe) because of its planimetric disposition, delimits the area. The channel was transformed in an underground sewer at the beginning of XX century collecting both stormwater and wastewater from the developing urban area. Now the connections to the urban drainage system were discontinued and the channel basically drains groundwater.

The described drainage system is connected to two pumping stations, as shown in figure 6.3.3; each of them pumps up to 0,5 m³/s. During winter, the pumped volumes are discharged to the sea, while during summer the tourist activities don't allow for discharging stormwater near the cost: the water volumes are so collected to two smaller pumping stations (usually used for pumpings wastewaters) and disposed of to a near wastewater treatment plant.

The nature of soils and the characteristics of urban areas allows for adopting either infiltration or retention/detention measures in all the Mondello catchment. The use of swales and vegetated measures is not suggestible because of climatic conditions: long dry periods with hot

temperatures make difficult the implantation of vegetal species. For the same reason in order to prevent septicity and population health consequences, underground measures should be preferred if possible, even if generally more expensive.

No discharge measurement is available in order to carry out a traditional calibration procedure. Starting from parameters literature values, also taking into account their physical significance, a weak calibration has been carried out using as an objective function the flooding volumes.

The computed values have been compared with the ones measured by the municipal Fire Brigades, with the aim to estimate suitable medium values for the parameters and to identify a reasonable variation range. Anyway, this consideration doesn't invalidate the procedure results, because its main purpose is the comparison between various mitigation measures scenarios.

The hydrological parameters values here adopted are shown in tables 5-1 and 5-2:

	r [mm]
Flat impervious surfaces	0,5
Rough impervious surfaces	2,0
Pervious surfaces	8,0

Table 6.3.1: detention storage parameters

	f ₀ [mm/h]	f _c [mm/h]	k [h ⁻¹]
Highly pervious soils	117	17	5.34
Moderately pervious soils	76	13	4.14
Poorly pervious soils	76	6	4.14

Table 6-3.2: Horton infiltration parameters (ASCE e WEF, 1992)

The Manning roughness coefficient has been assumed to be equal to 0,033 s/m^{1/3} for the pervious areas and equal 0.022 s/m^{1/3} for the impervious areas and the gutters; a value of 0.015 s/m^{1/3} has been adopted for the sewer pipes (rough cast iron).

4.3.1 Collected precipitation data

The simulations have been carried out using both synthetic and historical rain events. According to the circumstance that no raingauge exists inside the analysed catchment, the data coming from the nearby Parco d'Orléans meteorological station has been used. The station lies onto the Palermo catchment, the recordings have a one-minute temporal resolution of and a functioning period of 10 years (since 1991 until now). Raw rainfall data, coming from Parco d'Orléans raingauge, has been statistically processed, trough the application of a Gumbel probability law and then, trough an interpolation process, the IDF relationship parameters have been calculated. In the present study a classical three parameters IDF relationship formulation has been used:

$$i = \frac{a}{(t + b)^m}$$

where

i is the rainfall intensity [mm/h]

and t is the rainfall duration [h].

The figure 6.3.4 shows the IDF curves calculated for return periods between 2 and 10 years. In the table 5-3, IDF curves parameters have been collected.

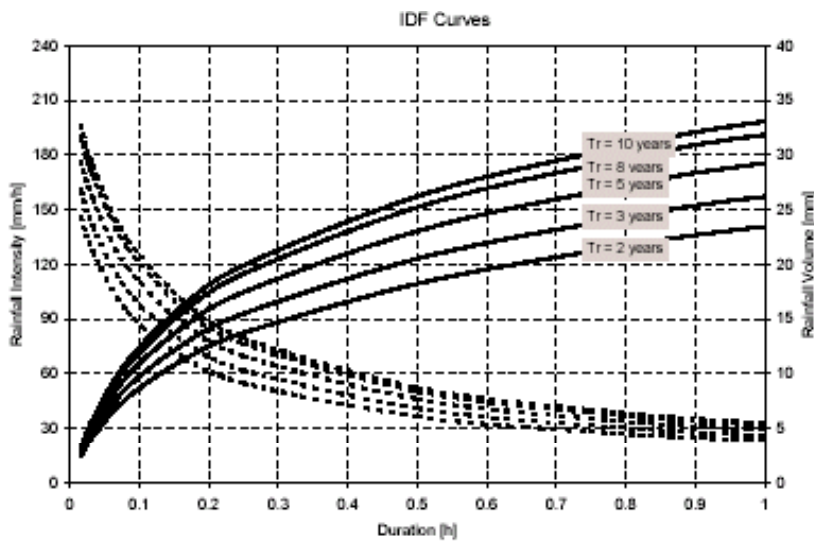
In order to evaluate DSMP measures efficiency in different functioning conditions, synthetic events has been used. In details, different “Chicago” type events have been built changing all the possible parameters: return period, duration and peak position.

Tr (yrs)	a	b	m
2	24.36	0.0577	0.6689
3	27.38	0.0655	0.7078
5	30.76	0.0734	0.7255
8	33.69	0.0796	0.7386
10	35.05	0.0823	0.7440

Table 6-3.3: Parco d’Orléans station – IDF curves parameters

A duration range varying between 30 minutes and 10 hours has been chosen also considering different times to peak. Defining the peak position r as the ratio between time to peak t_p and rainfall duration d , the following values were selected: $r = 0,0; 0,5; 1,0$.

Figure 6-3.4: Parco d’Orleans station – IDF and DDF curves



4.3.2 Simulation results discussion

Mondello example was selected for its urgent needs of a Stormwater Management Plan and because of the absence of an underground stormwater drainage system. As discussed in the second chapter, the construction of a brand new drainage system can be unaffordable for the municipality economical balance. At the same time, the high imperviousness of the catchment (averagely between 40% and 60%) and the presence of small diffuse pervious areas make this case study suited for DSMPs application.

The first step in this application is the verification of objective functions linearity with respect to parameters variation. If the response is positive, it is possible to apply Linearized Sensitivity method sharply simplifying the study and reducing efforts.

4.3.2.1 Objective functions linearity hypothesis verification

It should be observed that the First Order methodology is based on two fundamental assumptions that represent also its limit and approximation:

- The objective functions, generally non linear, are replaced by approximated linear relationships; as better explained later, the acceptability of such hypothesis depends on the characteristics of the adopted model and, in some cases, on the characteristics of the adopted mathematical solver;
- The use of linear relationship implies that the effects of the parameters variation over the objective functions should be overlapped (that is, the variation of the objective function due the variation of more parameters is the sum of the single variations, obtained changing each parameter);

The linearity of the objective functions with respect to the hydrological parameters has been here demonstrated to be acceptable, except when analysing short duration and high occurrence frequency rainfall events, for which the variation of the parameters may introduce a strong non linearity in the output of the model like threshold effects.

Following figures show, for example, the peak discharge and the runoff volume vs. directly connected impervious area and infiltration capacity, in the 11 urban catchments of Mondello area; a synthetic “Chicago” hyetograph with centred peak and 2 – 10 hours duration has been used as input rainfall. The linearity hypothesis has been verified also for the other analysed parameters, for all rainfall events used as input and, of course, for the entire catchment obtaining maximum gaps, with respect to the linear relationships, lower then $\pm 10\%$.

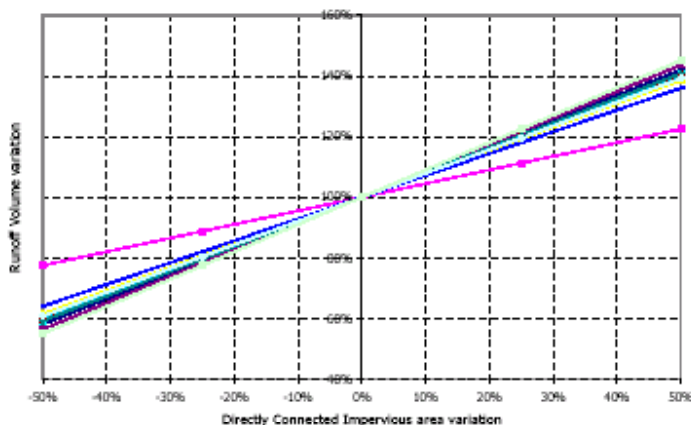


Figure 6.3.5: Mondello catchment: Runoff Volume variation with respect to the catchment impervious area (“Chicago” hyetograph - Return period: 10 yrs – duration: 5 hr – peak ratio 0.5)

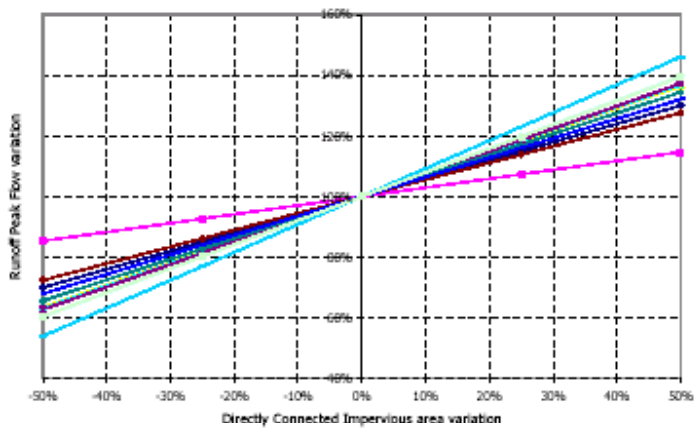


Figure 6-3.6: Mondello catchment: Runoff Peak Flow variation with respect to the catchment impervious area (“Chicago” hyetograph - Return period: 5 yrs – duration: 2 hr – peak ratio 0.5)

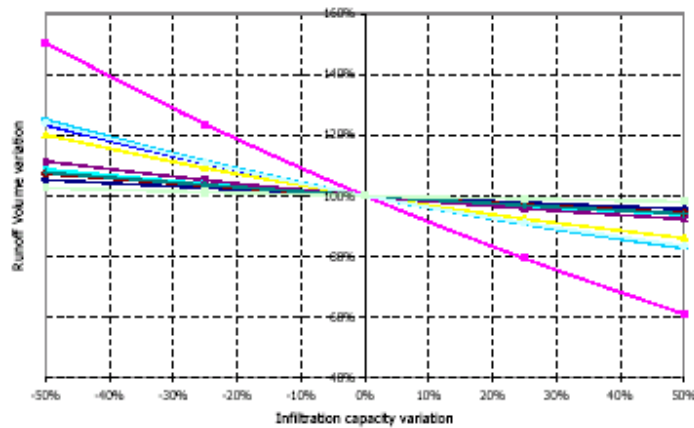


Figure 6-3.7: Mondello catchment: Runoff Peak Flow variation with respect to the infiltration parameters (“Chicago” hyetograph - Return period: 5 yrs – duration: 10 hr – peak ratio: 0.5)

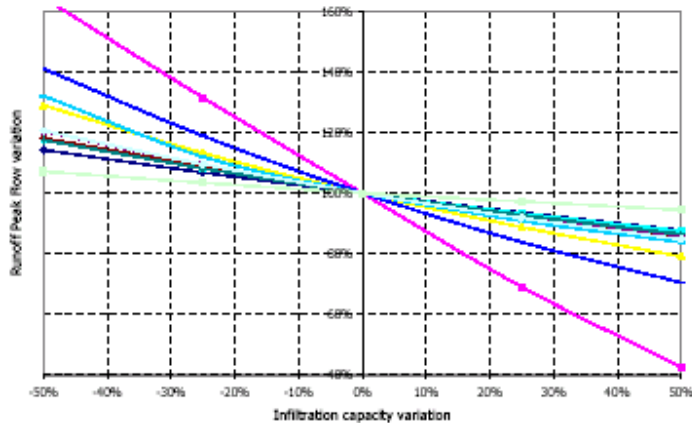


Figure 6-3.8: Mondello catchment: Runoff Peak Flow variation with respect to the infiltration parameters (“Chicago” hyetograph - Return period: 5 yrs – duration: 10 hr – peak ratio: 0.5)

4.3.2.2 Effective measure selection

At the beginning, the simulations have been aimed to the evaluation of the DSMPs mitigation effect using two objective functions: the peak flow and the runoff volume generated from each of the eleven catchments connected to Mondello drainage system.

For each catchment, the efficiency index has been computed in order to discriminate effective and ineffective measures. This has been carried out using a given input hyetograph and evaluating sensitivity of the system to each parameter. Using the same input rainfall event, different measures have been compared in order to find out which could be the most effective in the analysed scenario.

For a design “Chicago” hyetograph having a 5 hours duration, a 5 year return period and $r = 1$, figures 5-10/5-15 show that the largely most effective measures are connected with disconnection of the impervious area even if detention of impervious areas runoff and retention on pervious areas can still be taken into account. Differences between catchments can be explained on the basis of their hydrological characteristics:

- Catchment 1 is basically covered with detached houses and small diffuse public gardens (the whole catchment area is about 50 ha with imperviousness of about 30%); for this reason, impervious area disconnection is, in any case, the most effective mitigation measure for the design rainfall even if detention and retention on pervious areas has an increasing efficiency reducing rainfall frequency because of the increased importance of runoff contributions from that type of surfaces.
- Catchment 2 is characterised by detached houses with small gardens and rooftops connected to streets network through impervious lanes and parking lots; the high imperviousness level allows for a fast hydrological response so that the catchment results very sensitive to impervious area mitigation measures. Differently from Catchment 1, mitigation practices applied to pervious areas efficiency is negligible a part from low frequency events where pervious surfaces contribute substantially to runoff generation.
- Catchment 3 is one of the biggest and the most pervious one in Mondello area (the whole catchment area is about 280 ha with imperviousness of about 15%). Measures on pervious

areas have a high importance in flooding mitigation in this catchment even if retention practices have a higher performance if compared with infiltration facilities. Measures on impervious surfaces maintain a high efficiency in mitigating frequent and short rainfalls.

- Catchments from 4 to 8 have similar characteristics and similar behaviours. The percentage of impervious area is near 40% and the urban pattern is characterised by detached houses and block buildings with some cultivated areas. Because of the balanced presence of pervious sub-catchments and highly impervious ones (concentrated near the sea) all mitigation measures are effective for flooding reduction with the exception of infiltration facilities applied on pervious areas.
- Catchment 9 is the largest one with a total area of more than 350 ha. The catchment can be divided in different areas ranging from pervious cultivated lots to highly urbanised quarters. Impervious area disconnection appears to be the most effective mitigation measure. Retention measures on impervious areas are effective only for short and frequent events because of the fast hydrologic response of that portion of the catchment. Mitigation measures applied to pervious surfaces have minor effects on runoff peak flow while they are effective in runoff volume reduction. The reasons of this behaviour can be found in the presence of major pervious areas in the upstream part of the catchment while downstream sub-catchments are largely impervious. The impervious part of the catchment is basically responsible for peak flow generation while pervious areas runoff, because of the wide hydraulic distance, only influences the hydrograph tail.
- On Catchment 13 considerations similar to Catchment 9 can be made: a large upstream part of the catchment is totally pervious and it contributes to runoff generation only for long, low frequency rainfall events. Thus, mitigation measures on pervious areas are ineffective in peak flow reduction while DCIA disconnection and detention applied to impervious area are the most effective practices.
- Catchment 14 is quite small with a low perviousness level (with about the 75% of impervious area) making DCIA disconnection and impervious area detention the only effective measures. Mitigation measures on pervious areas have a negligible efficiency in runoff objective functions reduction.

Previous considerations can be condensed in the following general statements:

- Small and impervious catchments have a quite fast hydrological response due to directly connected impervious areas. On this type of catchment, DCIA disconnection measures and runoff detention should be further more analysed with respect to flooding volumes and DSMP costs. Measures applied on pervious areas have negligible efficiency and should be analysed only for long low frequency rainfall events.
- For catchments with considerable pervious area (characterised by detached houses and cultivated lots), mitigation measures both on impervious and pervious area should be analysed according to their different efficiency depending on rainfall frequency and temporal pattern.

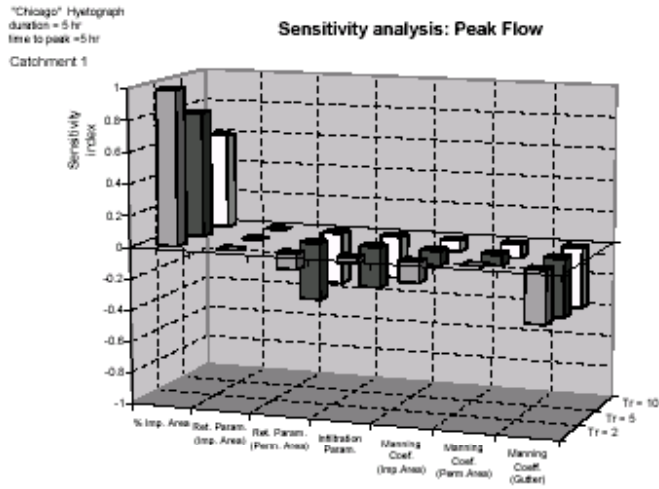


Figure 6-3.9: Sensitivity of the peak flow to the hydrological parameters: CATCHMENT 1 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and return period equal to respectively 2, 5 and 10 years)

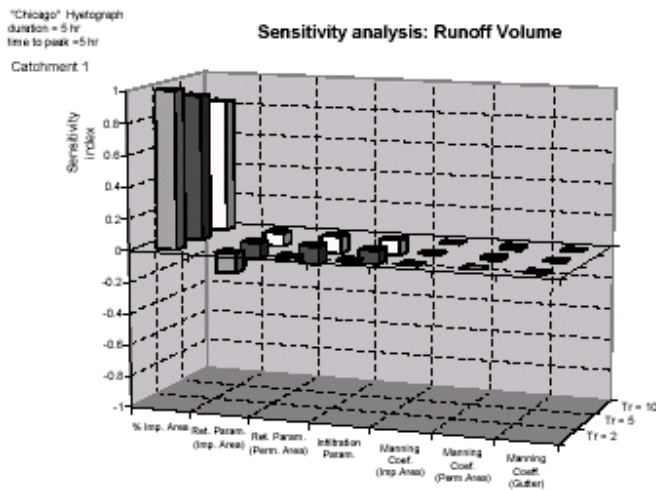


Figure 6-3.10: Sensitivity of the runoff volume to the hydrological parameters: CATCHMENT 1 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and (return period equal to respectively 2, 5 and 10 years)

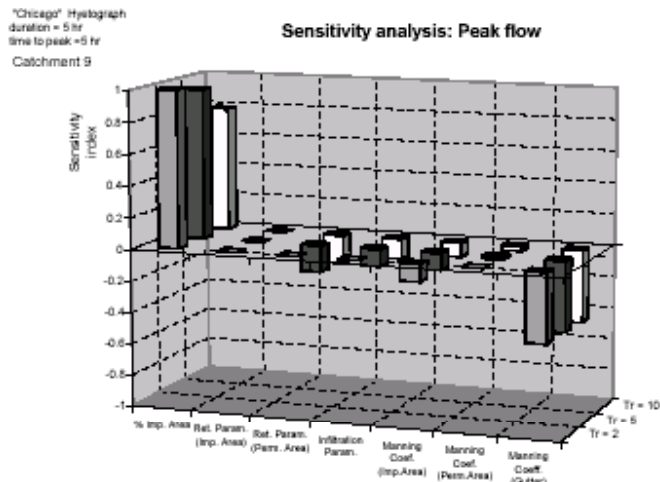


Figure 6-3.11: Sensitivity of the peak flow to the hydrological parameters: CATCHMENT 9 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and return period equal to respectively 2, 5 and 10 years)

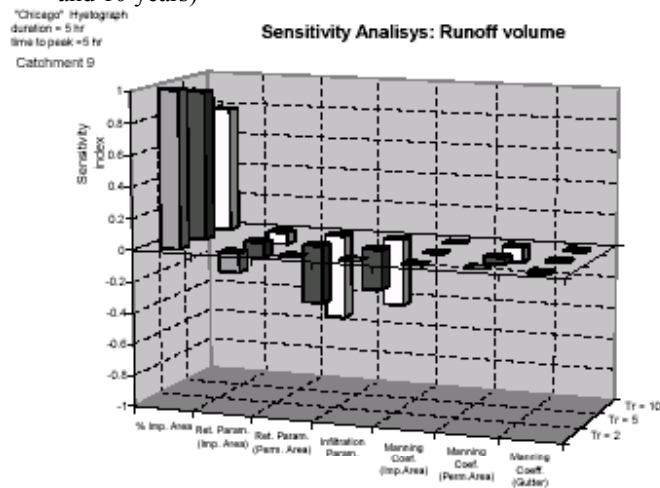


Figure 6-3.12: Sensitivity of the runoff volume to the hydrological parameters: CATCHMENT 9 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and return period equal to respectively 2, 5 and 10 years)

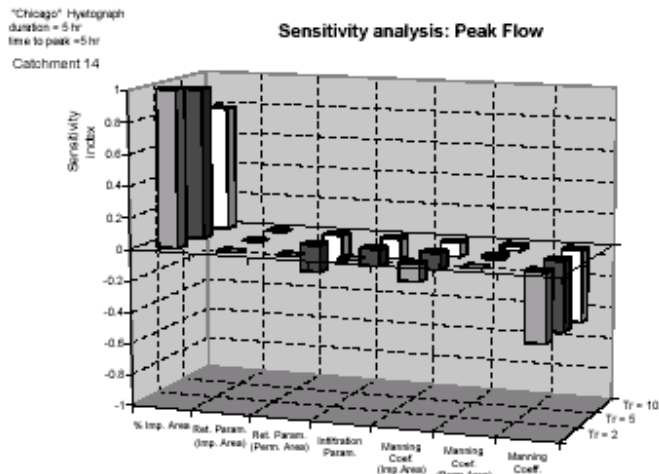


Figure 6-3.13: Sensitivity of the peak flow to the hydrological parameters: CATCHMENT 14 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and return period equal to respectively 2, 5 and 10 years)

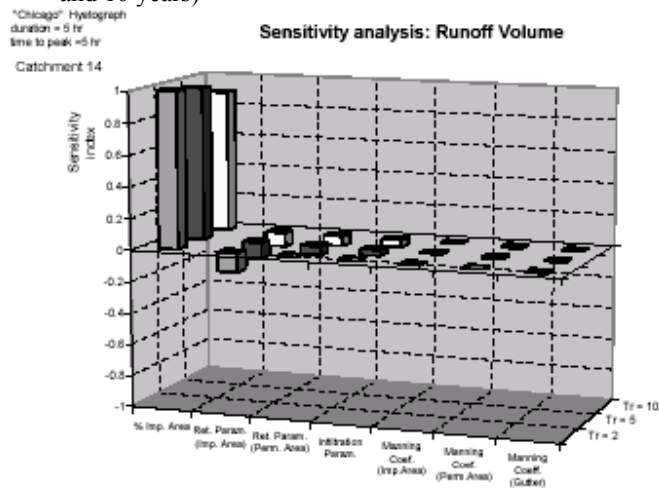


Figure 6-3.14: Sensitivity of the runoff volume to the hydrological parameters: CATCHMENT 14 (“Chicago” synthetic events with 5 hours duration, peak position at the end of the event and return period equal to respectively 2, 5 and 10 years)

4.3.2.3 Robustness analysis and comparison

Later, for each DSMP measure, the stability of the mitigation effect has been evaluated using different event duration, peak position and return period (robustness analysis). Aim of this further study has been the identification of the most robust mitigation measures. This analysis also allows for the evaluation of priority to assign to each measure inside a flooding protection plan: in a condition where financial resources are limited and not sufficient for the complete application of the protection plan, priority can be given to measures mitigating frequent flooding with high robustness so that, provisionally, they can be used to partially reduce damage and, in a further step, can be integrated in order to provide the designed protection. Using a “Chicago” shaped hyetograph, the efficiency of each measure has been analysed evaluating the influence of

rainfall duration and peak position. The analyses allow for evaluating the mitigation effect stability with respect to different possible rainfall patterns.

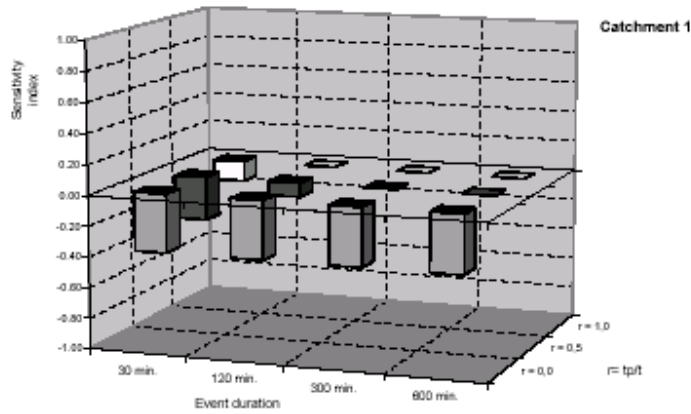
The efficiency of each measure has been analysed using different rainfall input. The rainfall duration has been varied between 30 minutes and 10 hours and the peak position, expressed as the ratio of the time to peak to the event duration, between 0 and 1. The following figures show the mitigation efficiency for two relevant catchments in the study area with respect to the peak flow reduction.

Through robustness analysis, it can be observed how:

- Generally, mitigation measures applied to the pervious areas of the catchment are relevant only for long duration rainfalls with the peak at the end of the event; this behaviour can be explained considering that pervious areas have natural retention and infiltration capacities that are able to damp shorter rainfalls. The pervious part of the catchment effectively participates to runoff generation only when long and low frequency rainfall is considered.
- Detention and DCIA disconnection measures shows a stable efficiency with respect to the variation of the peak position and the event duration; in the first case, the DCIA disconnection works as a permanent reduction of contributing impervious surfaces (hypothesis that can be considered valid as soon as the rainfall frequency is not exceptional) so the measure efficiency reduction is only connected to the increased runoff contribution coming from pervious areas; in the application of detention measures, the use of renewable peak shaving capacities makes the mitigation measure insensitive to the duration of the wet period preceding the peak.

"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Retention on impervious area over Peak Flow



"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Retention on impervious area over Peak Flow

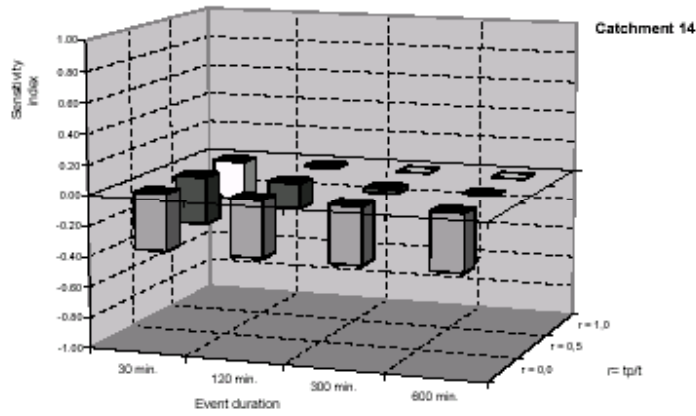
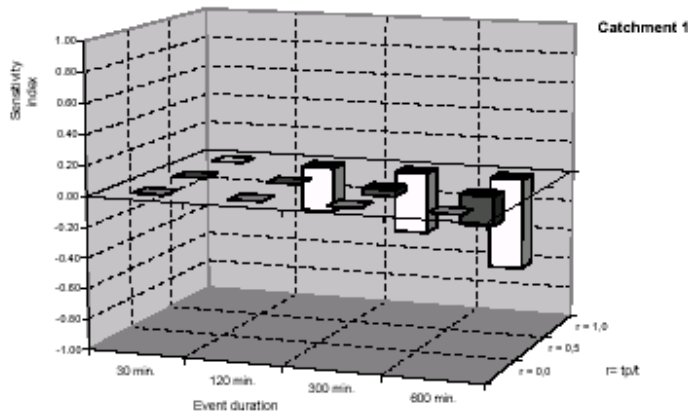


Figure 6-3.15: Runoff retention on impervious areas: peak flow reduction

"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Retention on pervious area over Peak Flow



"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Retention on pervious area over Peak Flow

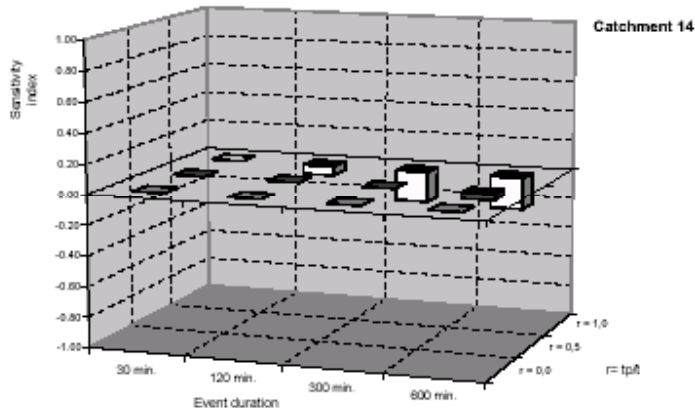
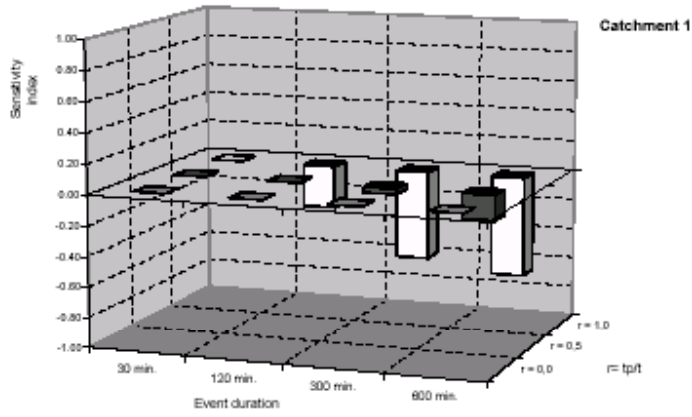


Figure 6-3.16: Runoff retention on pervious areas: peak flow reduction

"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Detention on pervious area over Peak Flow



"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Detention on pervious area over Peak Flow

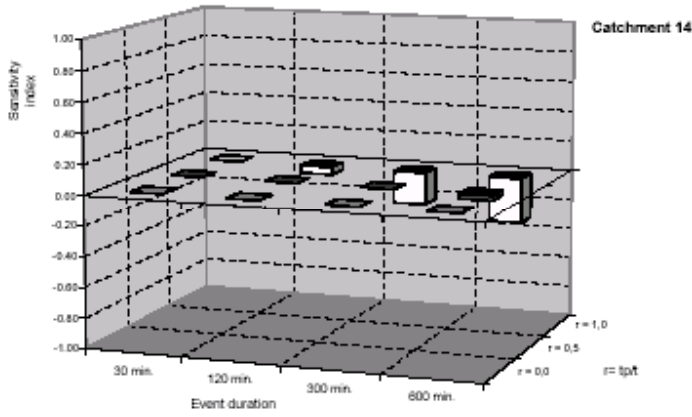
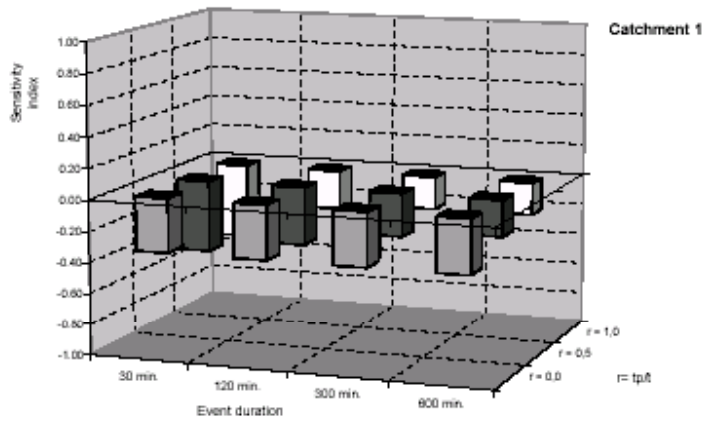


Figure 6-3.17: Runoff detention on pervious areas: peak flow reduction

"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Detention on impervious area over Peak Flow



"Chicago" Hyetograph
Return period = 10 years

Sensitivity analysis: Effect of Detention on impervious area over Peak Flow

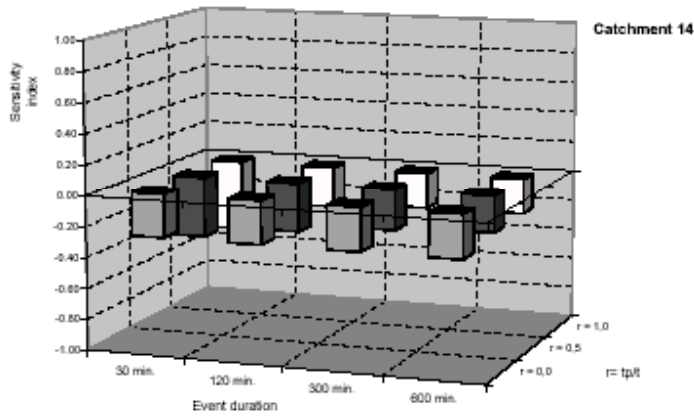
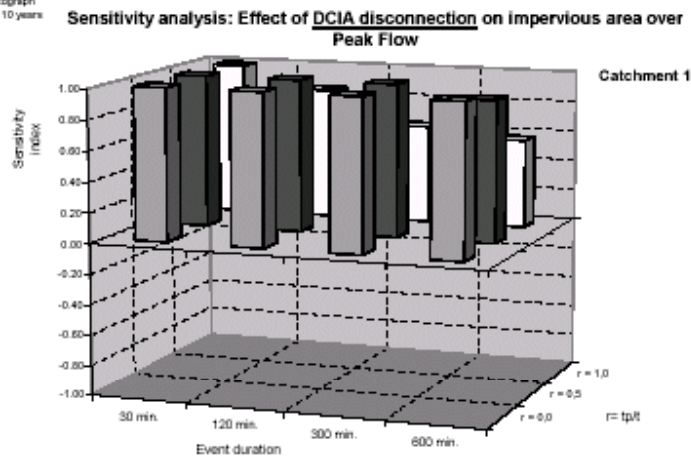


Figure 6-3.18: Runoff detention on impervious areas: peak flow reduction

"Chicago" Hyetograph
Return period = 10 years



"Chicago" Hyetograph
Return period = 10 years

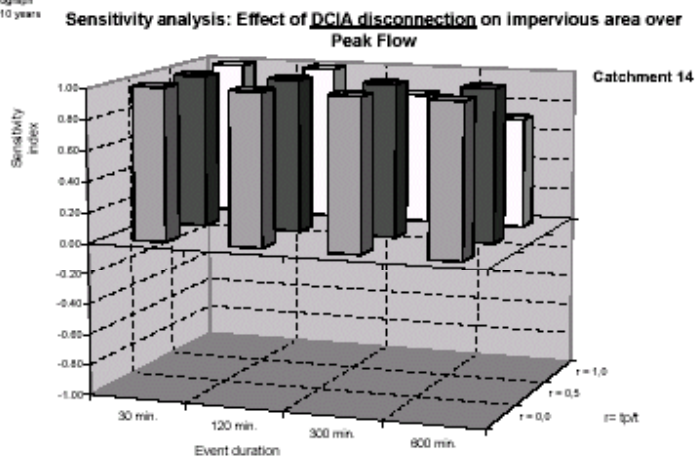


Figure 6-3.19: Directly connected impervious area disconnection: peak flow reduction

The peak reduction efficiency of retention measures on impervious areas decreases sharply increasing the rainfall duration and moving the hyetograph peak towards the end of the event. On the contrary of what happens on pervious areas, rainfall is converted to runoff rapidly and with only small depletions saturating retention capacities before the peak arrival. According to the discussed considerations, the following part of the analysis, using directly flooding volumes for efficiency evaluation, will be performed only on effective measures:

- Directly connected impervious area disconnection
- Impervious area detention
- Pervious area detention
- Pervious area retention

4.3.2.4 Flooding volume reduction efficiency

A very high computational effort is needed to simulate flow propagation inside the drainage system and flooding generation on the surface. For this reason, flooding analysis has to be carried out only on effective measure. These analyses can be also used to consider the dumping effect

connected to the drainage system interposition in order to evaluate if this effect is negligible with respect to the DSMPs efficiency evaluation.

From the analysis of graphs, some considerations can be done:

- Detention and Retention measures on pervious surfaces are almost ineffective for short and frequent rainfall events becoming progressively more important increasing the duration, the peak position and the return period of the input event; both solutions have almost the same efficiency apart from low frequency events where detention peak shaving is more efficient. According to the parameter values adopted in the study, detention/retention measures consisting in 60 m³/haperv (equivalent to 100% variation of the natural surface storage volume) allow for a reduction of about 30% of the flooding volume generated by a 10-years rainfall with 10 hours duration

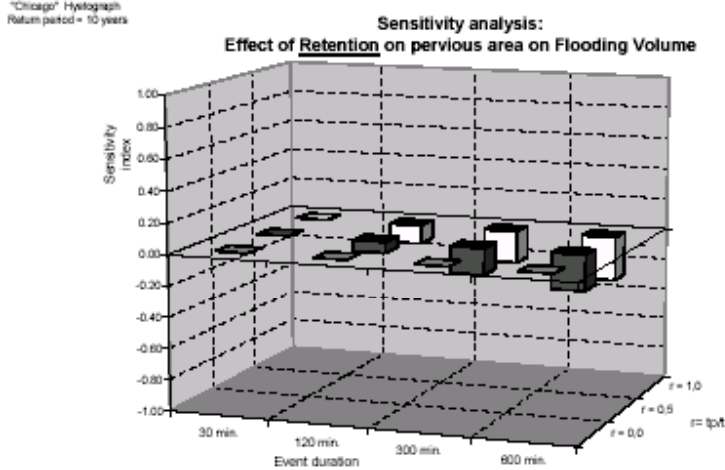
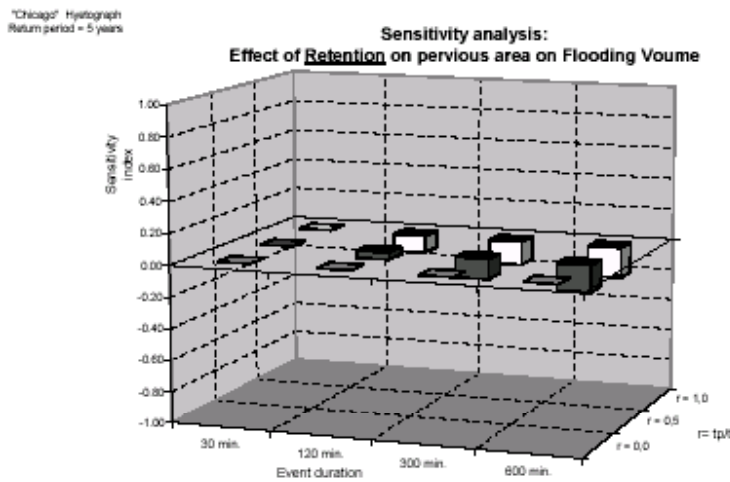
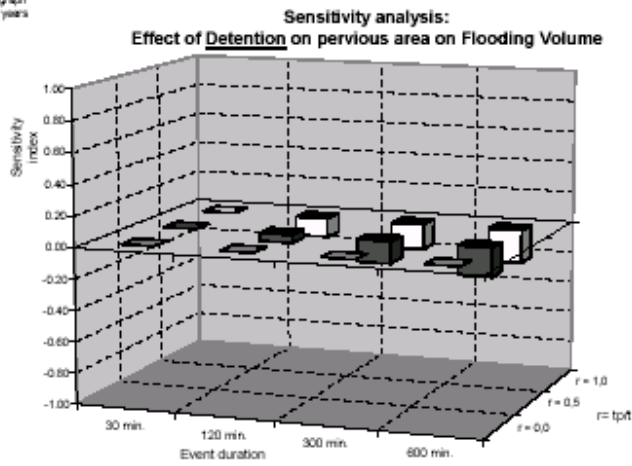


Figure 6-3.20: Retention on pervious area: flooding volume reduction (5-years and 10- years return period event)

"Chicago" Hyetograph
Return period = 5 years



"Chicago" Hyetograph
Return period = 10 years

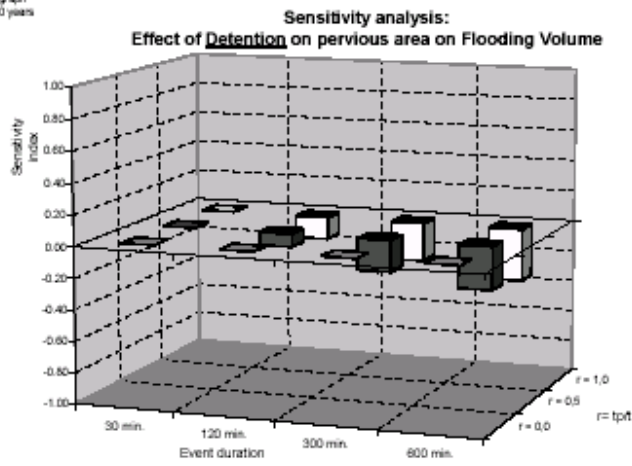
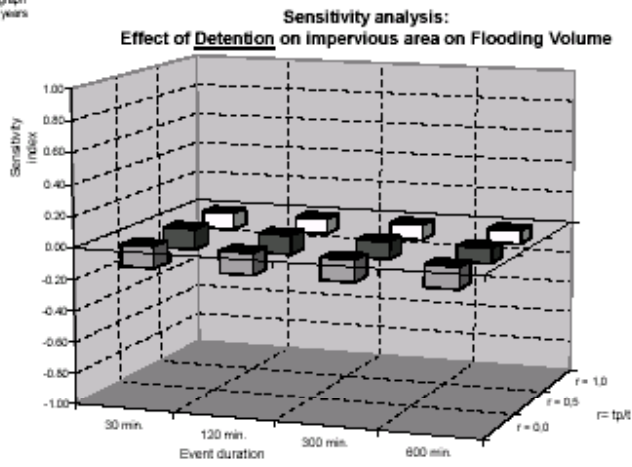


Figure 6-3.21: Detention on pervious area: flooding volume reduction (5-years and 10-years return period event)

"Chicago" Hyetograph
Return period = 5 years



"Chicago" Hyetograph
Return period = 10 years

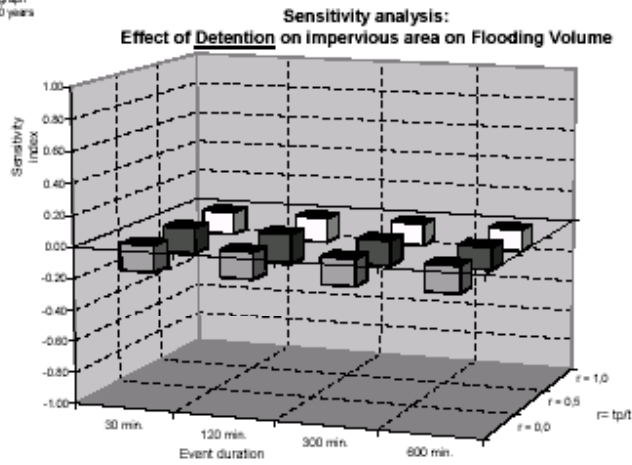
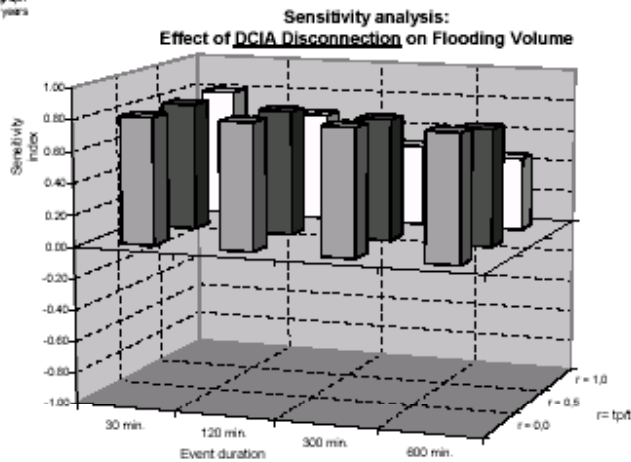


Figure 6-3.22: Detention on impervious area: flooding volume reduction (5-years and 10- years return period event)

"Chicago" Hyetograph
Return period = 5 years



"Chicago" Hyetograph
Return period = 10 years

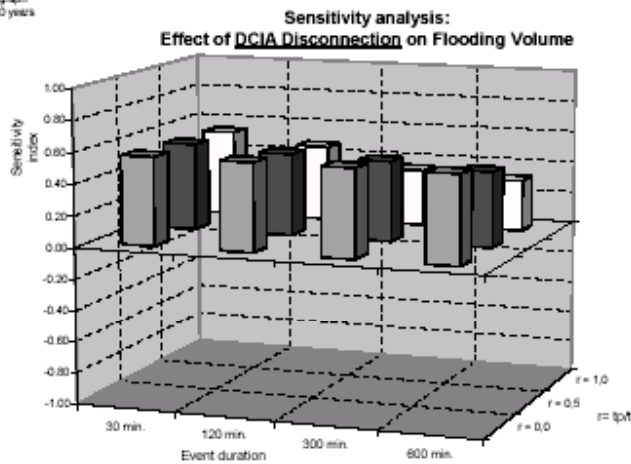


Figure 6-3.23: DCIA disconnection: flooding volume reduction (5-years and 10-years return period event)

- DCIA disconnection measures seems to be largely the most effective for flooding reduction in Mondello catchment confirming the results obtained with runoff analysis: also for a 10-year event with duration equal to 10 hours the flooding reduction efficiency is 0,35, that means 0,35% of flooding reduction for each percent of disconnected impervious area.
- Detention on impervious areas also seems to be an effective mitigation measure especially when it is not possible to disconnect impervious areas because of the high urbanization. According to the parameter values adopted in the study, detention measures consisting in 20 m³/ ha_{imp} (equivalent to 100% variation of the natural surface storage volume) allow for a reduction of about 15% of the flooding volume generated by a 10-years rainfall with 10 hours duration and its effectiveness is almost independent from input rainfall temporal pattern showing a good stability and reliability.

4.3.3 Final conclusions on Mondello catchment

The analysis of the results allow for identifying some guidelines for the mitigation plan preparation:

- In order to mitigate short and frequent rainfalls, the attention should be focused only on impervious areas neglecting the runoff contributions coming from pervious surfaces
- Increasing rainfall duration, return period and peak position, the pervious areas runoff contributions becomes more important, so that a mitigation plan has to contain also interventions on these areas

Retention on perv. Area

		PEAK POSITION		
		0.0	0.5	1.0
RAINFALL DURATION	30 min.	0.0%	0.0%	0.0%
	120 min.	0.0%	-8.3%	-20.2%
	300 min.	0.0%	-23.8%	-27.5%
	600 min.	0.0%	-31.2%	-35.6%

Detention on imperv. Area

		PEAK POSITION		
		0.0	0.5	1.0
RAINFALL DURATION	30 min.	-28.0%	-26.6%	-21.8%
	120 min.	-28.0%	-24.9%	-20.3%
	300 min.	-28.0%	-22.5%	-20.2%
	600 min.	-28.0%	-19.9%	-17.4%

DCIA disconnection

		PEAK POSITION		
		0.0	0.5	1.0
RAINFALL DURATION	30 min.	-36.3%	-36.2%	-36.3%
	120 min.	-36.3%	-35.6%	-30.8%
	300 min.	-36.3%	-34.6%	-23.0%
	600 min.	-36.3%	-33.3%	-20.8%

Table 6-3.4: Flooding volume reduction due to 1.000.000 Euros financial effort using different DSMP measures (“Chicago” rainfall events with 10 years return period)

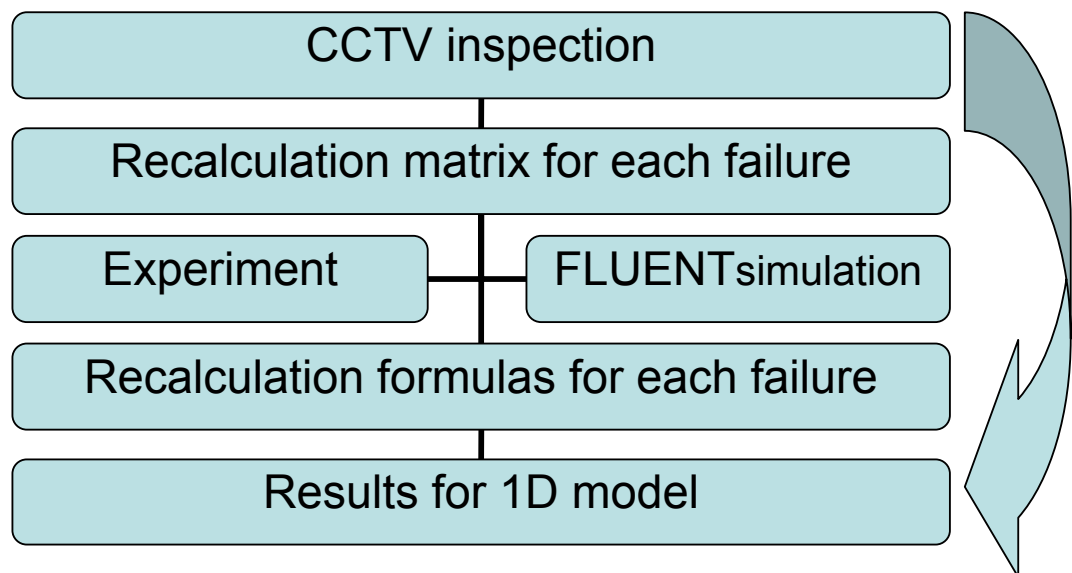
- While the impervious areas fast hydrological response do not allow for the use of retention facilities that are only able to mitigate very short events, the presence of natural capacities on pervious areas allow for a efficient use of both detention and retention measures even if, for high return periods, retention facilities show the same limitations presented on impervious surfaces.

4.4 A4 3D simulations using FLUENT MODEL

4.4.1 Description and validation of hydraulic performance and their environmental impacts (WP 3)

4.4.1.1 Methodology

Simulations using FLUENT 3D model were performed on fictitious pipes, trying to represent all the possible failure conditions. Comparing a new/clean pipe with a pipe affected by failure we derived hydraulic parameters that describe the real pipe condition better than the default values. The methodology is based on pressure losses on filled pipe compared to clean new pipe without any obstacles. Differences between pressure losses of these two pipes can be interpreted as a local head loss due to failure.



The capacity of the pipe is evaluated when it becomes pressurized, without considering the free surface flow conditions. During free surface condition the pipe does not have full flow capacity and the presence of eventual obstacles can only increase the internal water. For the calibration and the validation of FLUENT results, an experimental pipe, divided into three sections, was used. In the middle section obstacles were put. Measuring pressure differences between the beginning and the end of the pipe we compared that value with pressure differences received from FLUENT modelling.

For the simulations and study of failure effects the fictitious pipe used was of 20 m in length and with a circular section with a diameter of 1m.

Simulation done with FLUENT on the fictitious pipe without failures gave pressure and flow conditions. Starting from the initial condition of flow 0,785 [m³/s] and roughness 0,001 m which correspond to an old pipe in concrete material.

Obstacles and failures were simulated considering they occurred in the middle of the pipe. From difference of pressure losses between the beginning and the end of the fictitious pipe and the pipe with failures, we receive local losses due by the failure.

The following step was the definition of a new pipe that will be insert in the network, simulated by the 1D model, instead of the pipe with failures. This new pipe can be considered like an “equivalent pipe” of the real one: it will be described by the hydraulic parameters, defined with the 3D simulations, which will produce in the pipe the same hydraulic performances of the specific failure studied.

For calibration and verification of FLUENT results we used an experimental pipe, which was divided into three sections. into the middle section some obstacles were put as in sewer should be. the pressure differences measured at the downstream of the pipe was compared with the pressure differences resulting from FLUENT modelling.

Then, from the difference of pressure losses between the beginning and the end of the fictitious pipe and the pipe with failures, we receive local losses due by failure.

The equation known as the Darcy-Weissbach formula expresses the losses of head in pipes as given by

$$\Delta h = j \cdot l = \lambda \cdot \frac{l}{D} \cdot \frac{v^2}{2g} \quad (1)$$

Where λ is a friction coefficient. This equation applies to turbulent flow.

Writing the head in terms of pressure:

$$\frac{\Delta p}{\rho \cdot g} = \lambda \cdot \frac{l}{D} \cdot \frac{v^2}{2g} = \beta_r \frac{Q^2}{D^5} \cdot l \quad (2)$$

$$\Delta p = \rho \cdot \lambda \cdot \frac{l}{D} \cdot \frac{v^2}{2} \quad (3)$$

We will consider as roughness coefficient the Manning parameter „n“ [m^{1/3}/s] and λ as friction coefficient. The 1-D UDM use the following roughness coefficient:

- MOUSE: the Manning coefficient [m^{1/3}/s];
- InfoWorks: the Manning coefficient [m^{1/3}/s], the Gauckler Strickler coefficient [s/m^{1/3}]; friction factor λ ; InfoWorks can use either the Colebrook-White equation or the Manning formula to calculate hydraulic roughness. You may use two values, one for the bottom third of the link and one for the rest of the cross section, which is usually smoother. The default value for an individual conduit is the global value specified for the drainage system;
- SWMM: the „n“ coefficient [s/m^{1/3}].

To use the Colebrook-White equation type, typical values for the hydraulic roughness are as follows:

Description (Pipe sewers in good condition)	ks mm
Surface water	0.6
Foul or combined	1.5
Smooth concrete	1.5
Smooth brick	3.0
Rough concrete or brick	15.0
Old sewers -- part blocked	15.0 to 300.0
Smooth earth channels	60.0
Rough earth channels	300.0

Overgrown earth channels	600.0
--------------------------	-------

To use the Manning equation, select **Manning** or **N** as the hydraulic roughness type. Historically, HydroWorks treated the Manning's value input by the user as 1/n. We retain this treatment of Manning's for former HydroWorks users. You can now select **N** as roughness type and enter the normal Manning's n value. Typical values are:

Description	1/n (metric) (select Manning)	n (select N)
Smooth concrete	83	0.012
Rough concrete or brick	50	0.02
Smooth earth channels	33	0.03
Rough earth channels	5 to 25	0.2 to 0.04

If you define a large depth of sediment it is recommended that the bottom part of the link be made rougher to represent the high roughness of irregular sediment deposits. A ks value of 30 to 50 mm would be appropriate.

Relationship between Manning roughness factor “n” and the friction coefficient “λ”.

Starting from the beginning section and the end section of the fictitious pipe and pressure difference in the same sections of the pipe with failure, it is possible to evaluate the new “n” or “λ” values, that will be used to simulate the failure presence in the pipe. Local headloss can be calculated using the following equation:

$$\Delta h_{loc} = \frac{kv^2}{2g} \tag{4}$$

Where “k” is the headloss factor . “k” can be estimated with:

$$k = \frac{\Delta h_{loc} \cdot 2g}{v^2} = \frac{2\Delta p_{loc}}{\rho v^2} \tag{5}$$

If you need to recalculate for some reason Manning coefficient instead of k, the following procedure can be done:

$$n_{new} = \sqrt{n_{orig}^2 + \frac{k \cdot D^{\frac{4}{3}}}{124,5 \cdot l}} \tag{6}$$

4.4.1.2 Results

Results we can divided into 3 parts: (1) modelling of flow using 3D model, (2) experimental data and (3) sensitivity analysis. Results from each part was compared with results from the other parts.

4.4.2 Mathematical modeling

Mathematical modeling of flow in pipe was done with FLUENT program, which contains several turbulent models. Before starting any simulation, the turbulence model was tested on fictitious pipe and compared with Darcy-Weissbach formula. The k-ω turbulent model was chosen as the

best model. One of the advantages of the chosen model is the wall roughness model, which has the model for shear flow correction. It helps to save memory due to the wider calculation mesh.

This section presents the standard and shear-stress transport (SST) $k-\omega$ models. Both models have similar forms, with transport equations for k and ω . The main differences between the SST model and the standard model are as follows:

- gradual change from the standard $k-\omega$ model in the inner region of the boundary layer to a high-Reynolds-number version of the $k-\epsilon$ model in the outer part of the boundary layer
- modified turbulent viscosity formulation to account for the transport effects of the principal turbulent shear stress

The transport equations, methods of calculating turbulent viscosity, and methods of calculating model constants and other terms are presented separately for each model.

4.4.2.1 Calculation mesh

For simulations in 3D is necessary to create a calculating mesh. This mesh must comply with these criteria:

- Triangle mesh was used: it fits better into circular pipe than squares
- Number of cells around walls must be thick enough, because for pressure losses are important boundary layer.
- Ratio between sizes of smallest and biggest triangle cannot be higher than 10 (Fig. 1)
- Other criteria for stable calculations – quality of the mesh

For each obstacle separated mesh was created. Next pictures show some examples of mesh quality for intruding pipe $D=0.7$ m, 0.5 m of intrusion:

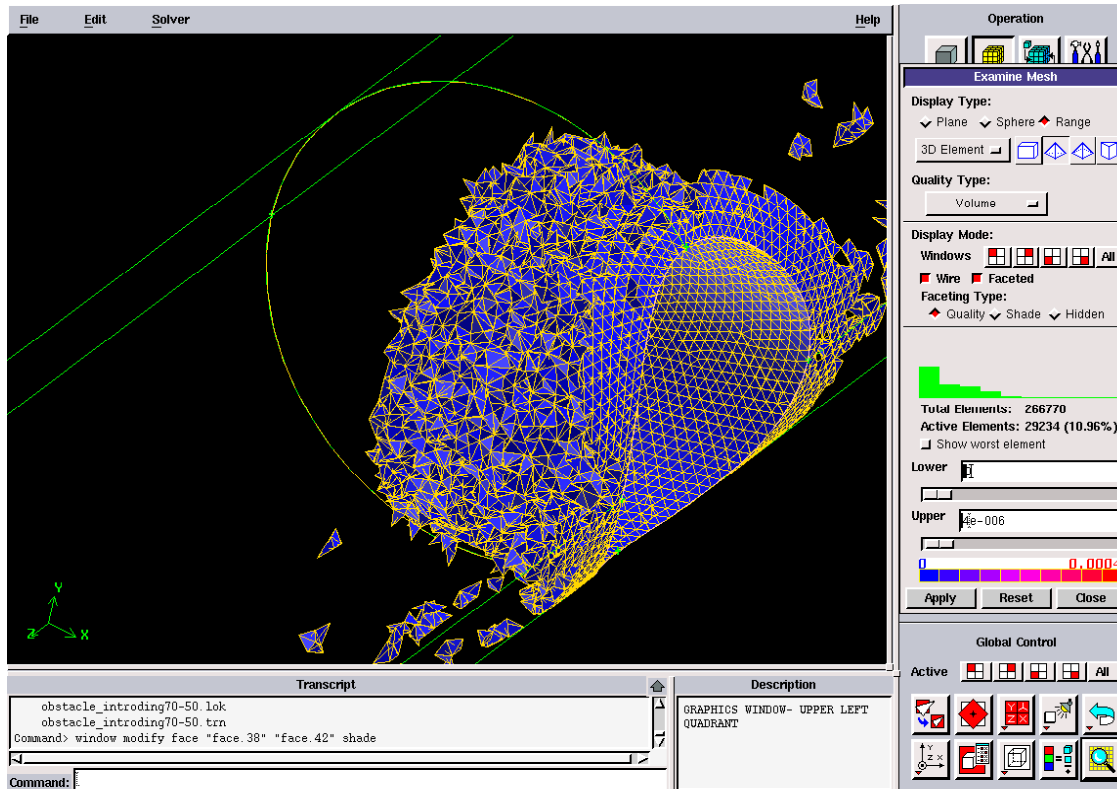


Fig. 1 minimal volume of triangle mesh

4.4.2.2 Fictitious pipe

For the simulation of any failures in pipe is necessary to have long pipe to establish flow field. We are using fictitious pipe without any failures of 20 m length and with a circular section with a diameter of 1m.

Simulations done with FLUENT on the fictitious pipe without failures give pressure and flow conditions. Starting from the initial condition of flow 0,785 [m³/s] and roughness 0,001 m which is like old concrete:

4.4.3 Simulation of obstacles

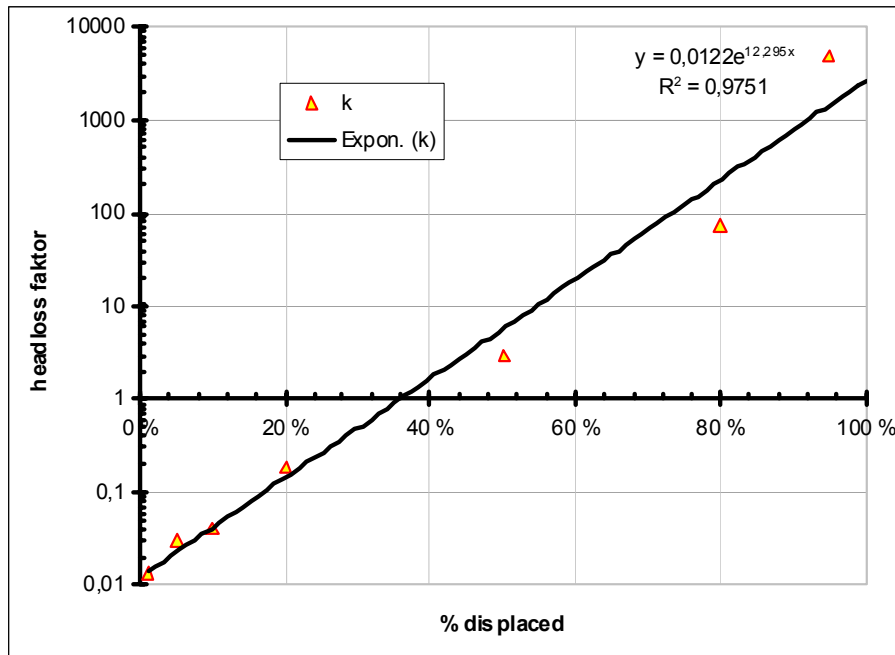
Obstacles were divided into groups and those groups were simulated separately. Obstacle was always putted into the middle of fictitious pipe.

4.4.3.1 Simulated types and sizes of obstacle

For the simulation of obstacles we simplify the types. Each type was simulated in several different sizes.

4.4.3.1.1 Displaced pipe

This first simulation of obstruction in sewer system was simulated as two pipes of 10 m in length in different levels connecting.



Graph 1 head loss factor due to the displacement of pipe

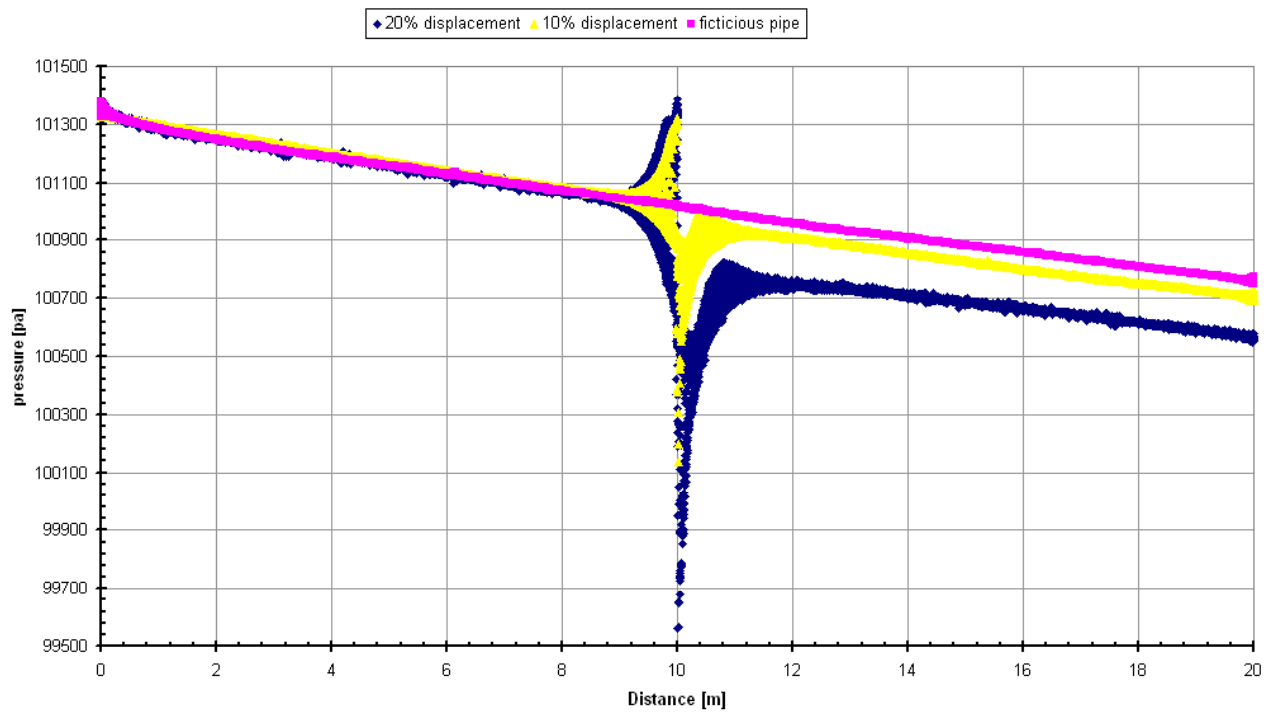
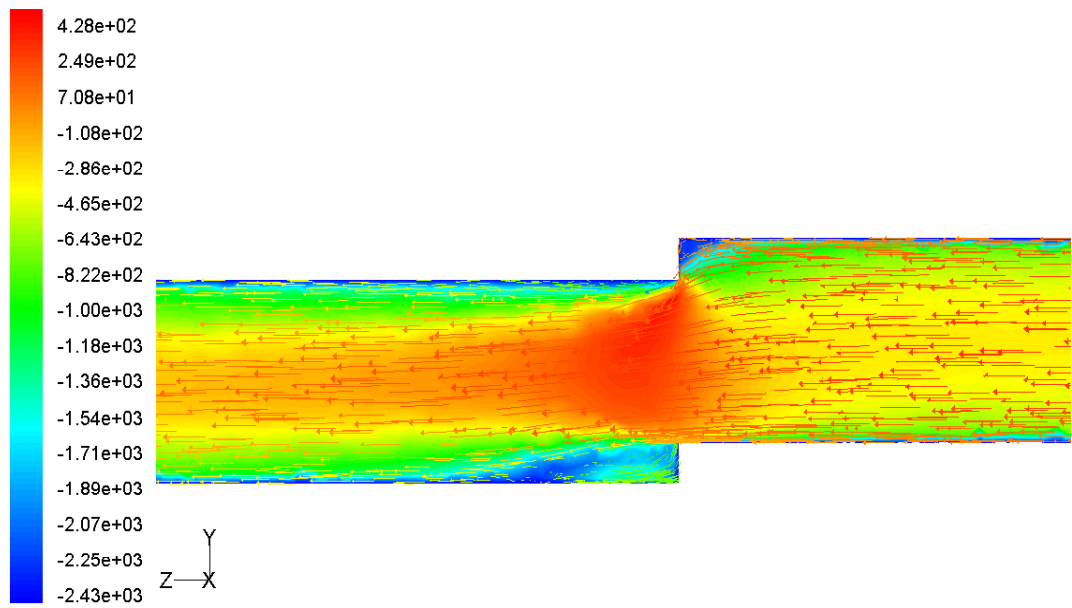


Fig. 2 pressure losses in the pipe compared with fictitious pipe



Velocity Vectors Colored By Total Pressure (pascal) Oct 08, 2003
FLUENT 6.0 (3d, segregated, skw)

Fig. 3 velocity profile with velocity vectors colored by static pressure (20% displacement)

4.4.3.1.2 Obstacle “brick”

This obstacle is like a brick lying on the pipe surface. The simulation considers a brick inside the pipe, where water can flow under the brick.

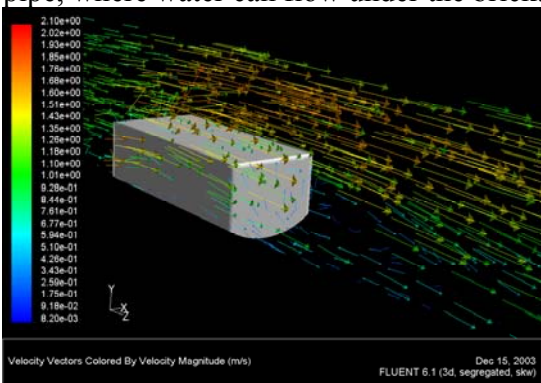


Fig. 4 velocity vectors for brick

4.4.3.1.3 Obstacle “Pipe through pipe”

This is the simulation of pipe or cables intruding through sewer.

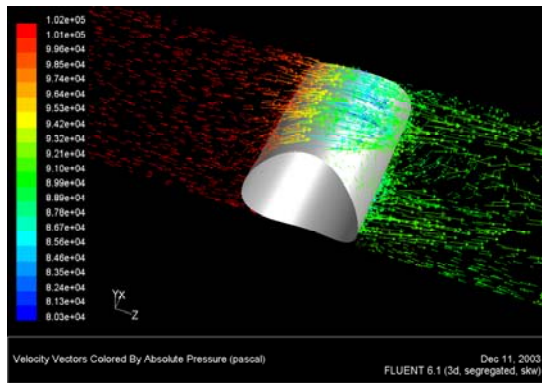


Fig. 5 velocity vectors colored by pressure around the 0,8 m pipe intruding into the transversal direction

4.4.3.1.4 Obstacle “Partly intruding pipe”

This is the simulation of pipe or cables intruding sewer. This model is used for intruding connection and for obstacle model. Several different sizes and distance of intrusion was simulated.

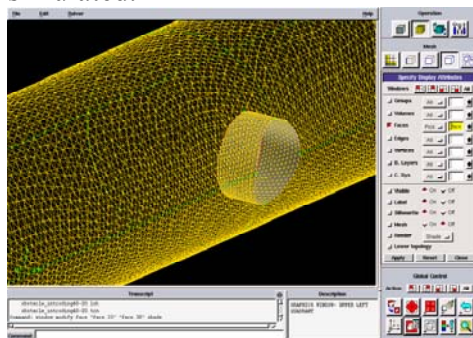


Fig. 6 mesh of 0,4 m pipe intruding into main pipe (0,2 m)

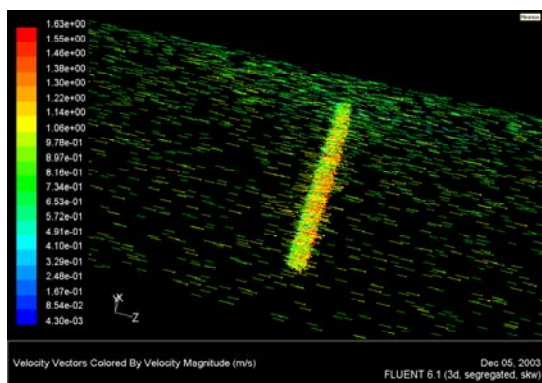


Fig. 7 velocity vectors for 0,2 m pipe intruding 0,7 m inside

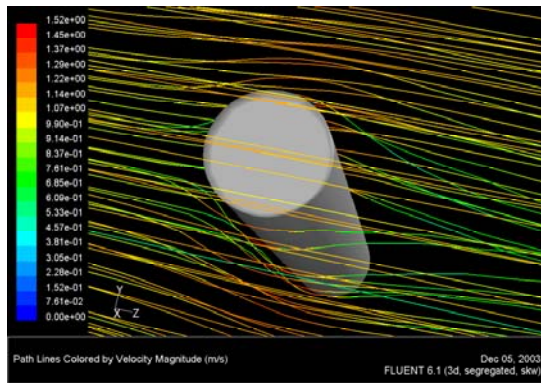


Fig. 8 path lines colored by velocity magnitude around 0,4 m pipe partly intruding into sewer

4.4.3.1.5 Obstacle “Local solid sediment”

This type of obstacle is like local solid sediment on the bottom of the pipe.

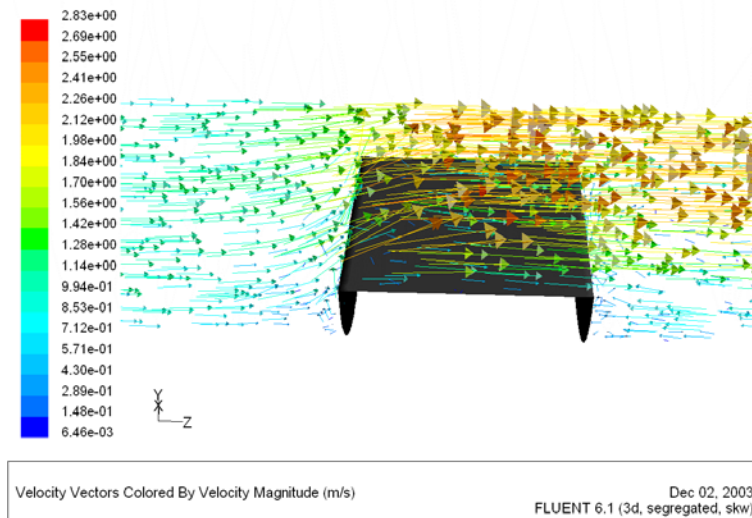


Fig. 9 velocity vectors on 50% sediment

4.4.3.1.6 Roots

For simulation of the effects of roots intruding into sewer was developed a different model. Roots were simulated as a porous media.

4.4.3.1.6.1 Inertial Losses in Porous Media

At high flow velocities, the constant C_2 in Equation (7) provides a correction for inertial losses in the porous medium. This constant can be viewed as a loss coefficient per unit length along the flow direction, thereby allowing the pressure drop to be specified as a function of dynamic head. For modeling a perforated plate or tube bank, we can sometimes eliminate the permeability term and use the inertial loss term alone, yielding the following simplified form of the porous media equation:

$$\nabla p = -\sum_{j=1}^3 C_{2,j} \left(\frac{1}{2} \rho v_j v_{\max} \right) \quad (7)$$

or when written in terms of the pressure drop in the x , y , z directions:

$$\begin{aligned} \Delta p_x &\approx -\sum_{j=1}^3 C_{2,j} \Delta n_x \frac{1}{2} \rho v_j v_{\max} \\ \Delta p_y &\approx -\sum_{j=1}^3 C_{2,j} \Delta n_y \frac{1}{2} \rho v_j v_{\max} \\ \Delta p_z &\approx -\sum_{j=1}^3 C_{2,j} \Delta n_z \frac{1}{2} \rho v_j v_{\max} \end{aligned} \quad (8)$$

Again, the thickness of the medium (Δn_x , Δn_y , or Δn_z) is the thickness defined in our model.

Coefficient used in our roots model for inertial resistance is:

$$\Delta p = C_2 \cdot \frac{1}{2} \cdot \rho \cdot v \cdot v_{\max} \quad (9)$$

$$C_2 = \frac{\frac{1}{2} \cdot \rho \cdot v \cdot v_{\max}}{\Delta p}$$

$C_2=5$ in all directions x , y , z

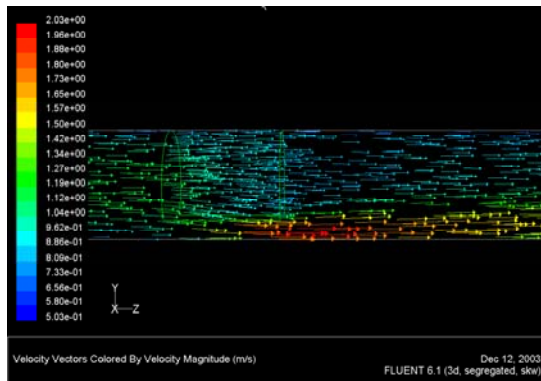
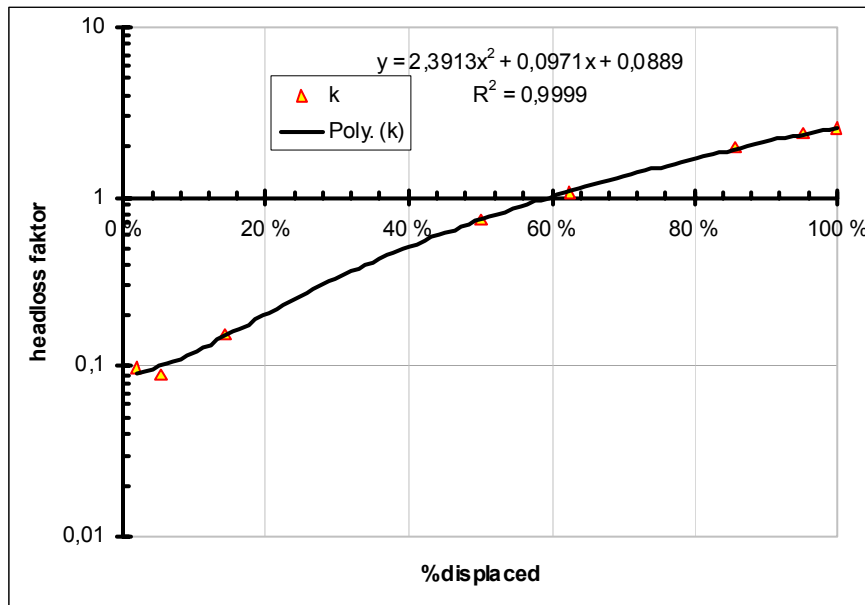


Fig. 10 velocity vectors of flow through intruding roots



Graph 2

4.4.3.2 Overall results

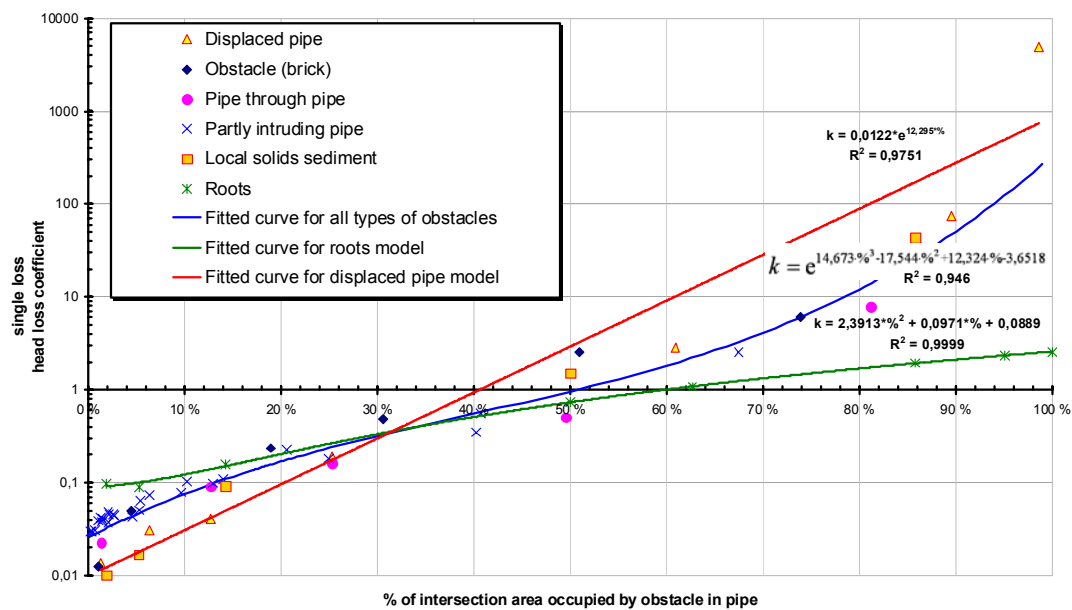


Fig. 11 Overall results

4.4.4 Experiment

For the validation of resulting data an experimental pipe was created (Fig. 12).

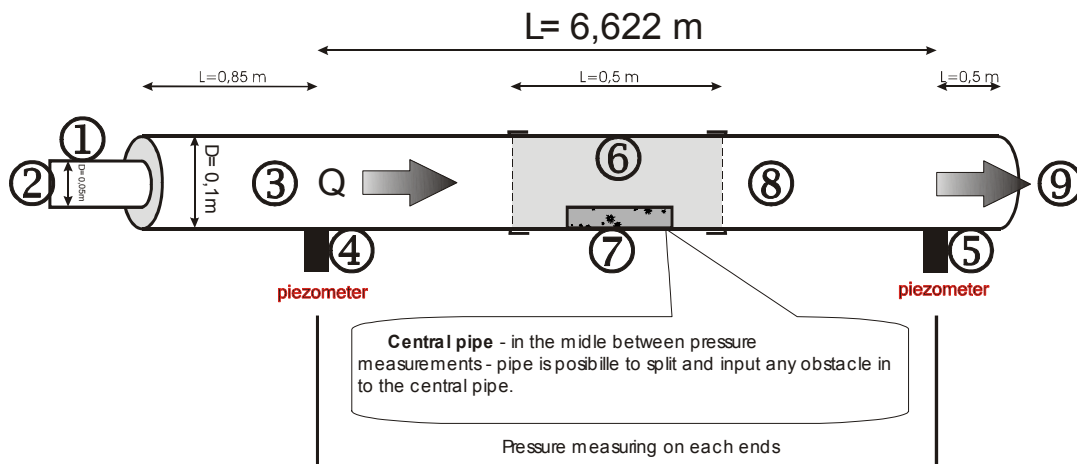


Fig. 12 experimental pipe

The experimental pipe consists of (Fig. 12):

1. inflow pipe $D=0,05 \text{ m}$
2. flowmeter on inflow pipe
3. beginning of experimental pipe for development flow field
4. inflow piezometer
5. outflow piezometer
6. middle piece, which can be removed and obstacle can be inserted
7. removable obstacle
8. experimental pipe
9. outflow



Fig. 13 Experimental field

The first experiment was run without any obstacle to compare pressure losses due to the wall friction. The second part of the experiment was performed with the insertion of obstacles in the middle of the experimental pipe. We inserted 2 types of obstacles (Fig. 14).

1. obstacle was 0,1 m in length and 0,05 in width and height (it filled pipe till the middle of the section). It was placed in the middle bottom of the experimental pipe.
2. obstacle was 0,1 m in length and 0,05 in width and height (it filled pipe till the top). It was placed in the middle bottom of the experimental pipe.

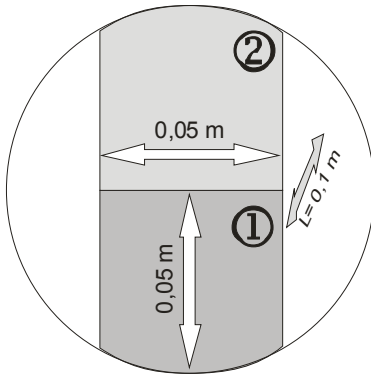


Fig. 14 Obstacle in experimental pipe

Exactly the same procedure (same pipe, flow rate, geometry) was done with mathematical model FLUENT. We use same turbulent model for flow as in the previous model to compare results from experiment (Fig. 15).

Pressure losses on experimental pipe were modeled. On the beginning of the curve you can see pressure drop due to the inflow. On zero is first pressure measurement and second pressure measurement is on the end. In the middle is pressure loss due to the obstacle.

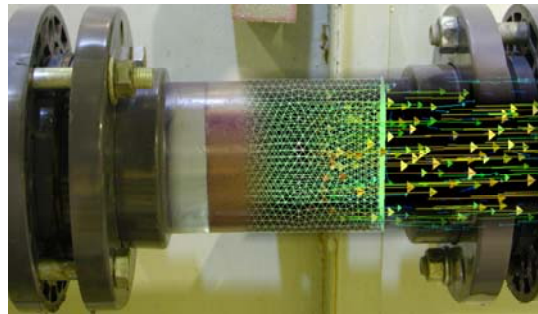
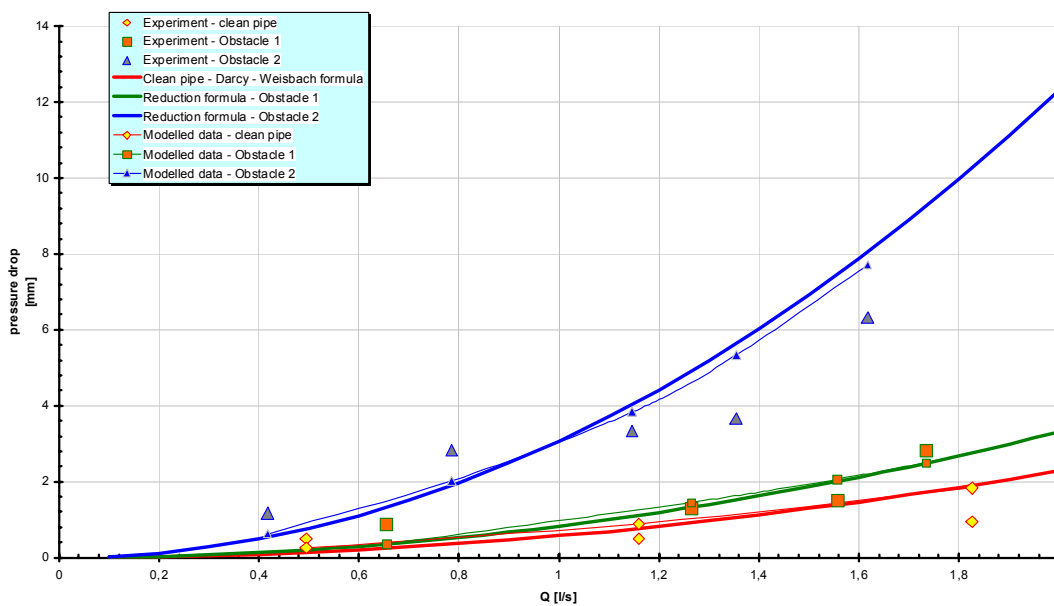


Fig. 15 Middle exchangeable piece with obstacle 2 - from experiment to mathematical model

Graph 3 - Verification of mathematical modeling



Results (Graph 3) from experiment show good accordance with modeled data. Data from experiment and FLUENT simulations for the same flow rate were compared. The line on the chart is the comparison with developed obstacle substitution. Only comparison between pipes without obstacles shows difference between experiment and modeled data. It was due to small difference and fluctuating inflow rate.

4.4.5 Sensitive analysis

Two type of sensitivy analysis was done. The first analysis is only the test of the recalculation of failure (displacement of connection) to manning number. The second was the test with real data in Norberg catchment.

4.4.6 Pipe sensitivity analysis

First test was done with program MOUSE 2000. Only one pipe (250 m in length an 1 m in diameter) was modeled. Inflow was 0,185 m³/s. results are shown in table (Tab. 1):

simulation No.:	Displacement	% displacement	n _{orig}	k	n _{new}	H [m]	Q [m ³ /s]
1	0	0 %	0,014	0	0,0140	0,347	0,185
2	0,1	10 %	0,014	0,042	0,0140	0,347	0,185
3	0,2	20 %	0,014	0,143	0,0141	0,349	0,185
4	0,3	30 %	0,014	0,488	0,0143	0,351	0,185
5	0,4	40 %	0,014	1,668	0,0149	0,358	0,185
6	0,5	50 %	0,014	5,704	0,0170	0,383	0,185
7	0,6	60 %	0,014	19,51	0,0226	0,442	0,185
8	0,7	70 %	0,014	66,7	0,0356	0,559	0,185
9	0,8	80 %	0,014	228,1	0,0621	0,793	0,185
10	0,9	90 %	0,014	779,9	0,1128	1,638	0,185

Tab. 1 results from sensitive analysis for one pipe

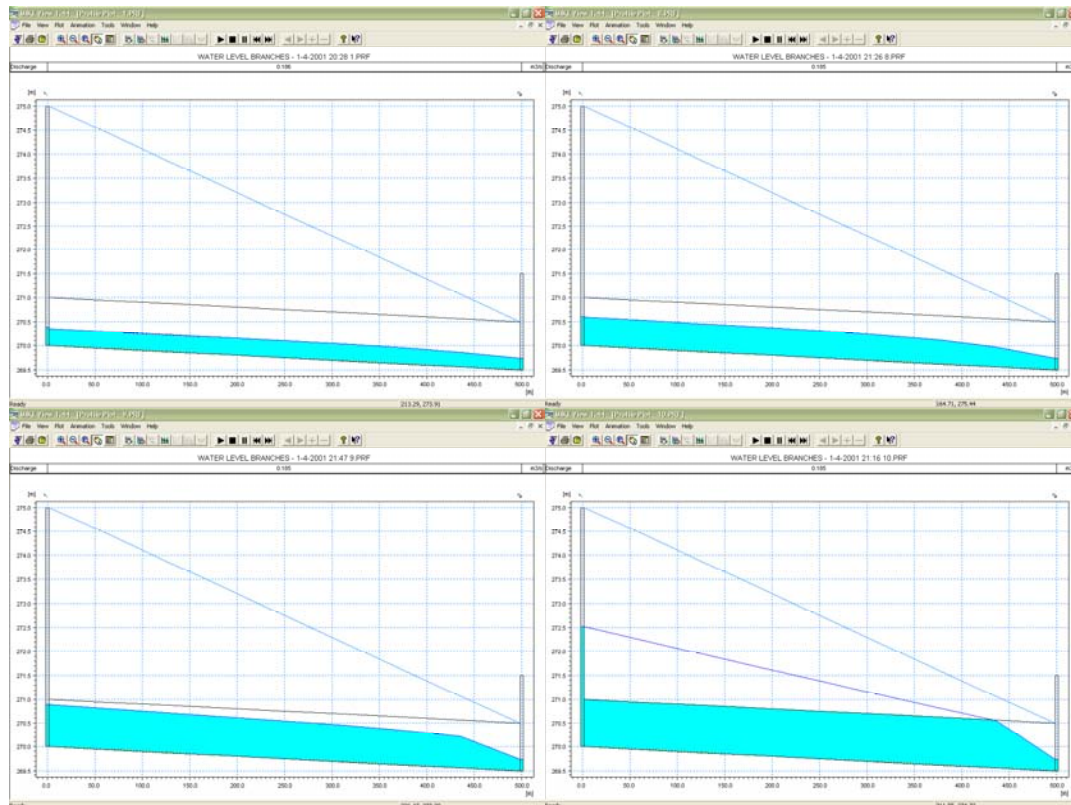


Fig. 16 graphic results from analysis for simulation 1,8,9,10
 In pictures (Fig. 16) it is possible to follow changes of water level due to fictitious displacement.

4.4.7 Matrix

A recalculation matrix was developed for the translation of CCTV data collected as required by EN 13508 to inputs of 1D UDM. The matrix consists of three parts. All other codes, which are not included in that table, are negligible.

4.4.7.1 Inputs from CCTV inspection

Failure EN 13508	code EN 13508	Characterization EN 13508		Quantification EN 13508	
		C1	C2	Q1	Q2
Deformation	BAA	A,B		%	
Break/collapse	BAC	A,B,C		mm	
Defective Brickwork or Masonry	BAD	A,B,C,D	A,B	mm	
Missing Mortar	BAE			mm	
Surface Damage	BAF	A,B,C,D,E,F,G,H,I,J,Z	A,B,C,D,Z		
Intruding Connection	BAG			%	
Displaced Joint	BAJ	A,B,C		mm, °	
Roots	BBA	A,B,C		%	
Settled Deposits	BBC	A,B,C,Z		%	
Ingress of Soil	BBD	A,B,C,D,Z		%	
Other Obstacles	BBE	A,B,C,D,E,F,G,H		%	

4.4.7.2 Recalculation matrix

This part was created to translate CCTV codes into numbers, which are used in next part – recalculation formulas.

Code	Quantification of characterization 1								quantification of characterization 2							
	A	B	C	D	E	F	G	H	I	J	Z	A	B	C	D	Z
BAA	100 %	00 %														
BAC	95 %	80 %	00 %													
BAD	95 %	00 %	00 %	00 %								F	05 %			
BAE																
BAF	110 %	00 %	10 %	10 %	10 %	10 %	10 %	10 %	10 %	10 %	10 %					
BAG																
BAJ	0 %	00 %	00 %													
BBA	100 %	50 %	00 %													
BBC	100 %	00 %	00 %								00 %					
BBD	100 %	00 %	00 %	00 %							00 %					
BBE	100 %	00 %	00 %	00 %	00 %	00 %	00 %	00 %								

4.4.7.3 Recalculation formula

For calculating the changed characterization of deteriorated sewer, the following formulas was developed. New roughness or headloss coefficient for deteriorated pipes can be calculated and used on 1D model instead of the usual default values. This operation is very easy once it are known the previous and the new characterization as defined by EN 13508. 100 % means no change between the pipe with failure and the pipe without failure.

Code	Roughness	shape/area
BAA	100 %	$A_{reduced} = A_{projected} - C1 \cdot Q1 \cdot A_{projected}$
BAC	100 %	$A_{reduced} = C1 \cdot A_{projected}$
BAD	100 %	$A_{reduced} = C1 \cdot C2 \cdot A_{projected}$
BAE	$n_{new} = C1 \cdot n_{orig}$	100 %
BAF	$n_{new} = C1 \cdot n_{orig}$	100 %
BAG	$k = C1 \cdot 0,0384 \cdot e^{\frac{6,3692 \cdot Q1_{BCA} \cdot Q1}{\pi \cdot C1_{ACB} \cdot D^4}}$	100 %
BAJ	$k = C1 \cdot 0,0122 \cdot e^{12,295 \cdot \frac{Q1}{D}}$	100 %
BBA	$k = C1 \cdot (2,3913 \cdot Q1^2 + 0,0971 \cdot Q1 + 0,0889)$	100 %
BBC	100 %	$A_{reduced} = C1 \cdot A_{projected}$
BBD	100 %	$A_{reduced} = C1 \cdot A_{projected} / \text{shape}$
BBE	$k = C1 \cdot e^{14,673 \cdot Q1^3 - 17,544 \cdot Q1^2 + 12,324 \cdot Q1 - 3,6518}$	100 %

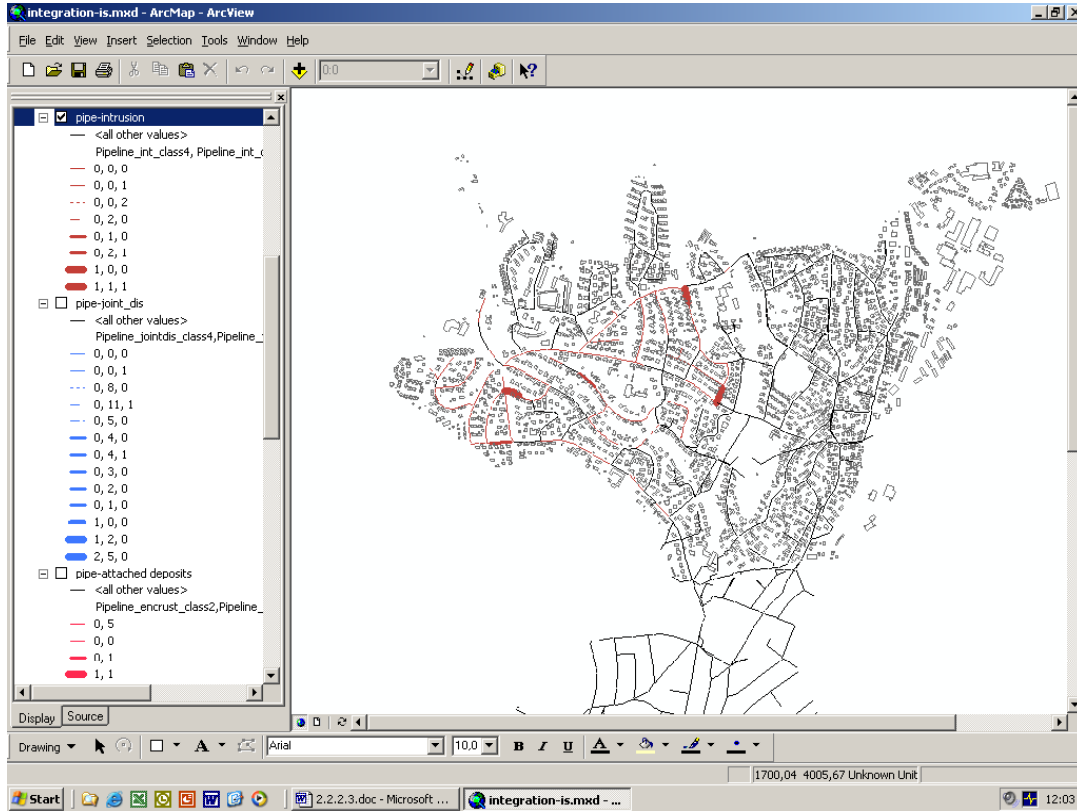
4.4.8 Conclusion

Recalculation matrix for deteriorating sewer was developed. Using recalculation matrix is easy to change input values to 1D model using CCTV analysis and change the characterization of sewer.

Those values are used instead of the default parameters commonly used for hydraulic simulations.

Data was verified by experiments and results were tested by sensitive analysis.

4.5 APPENDIX 6.5 The Nordberg catchment case study #2 - Spatial distribution of the failures



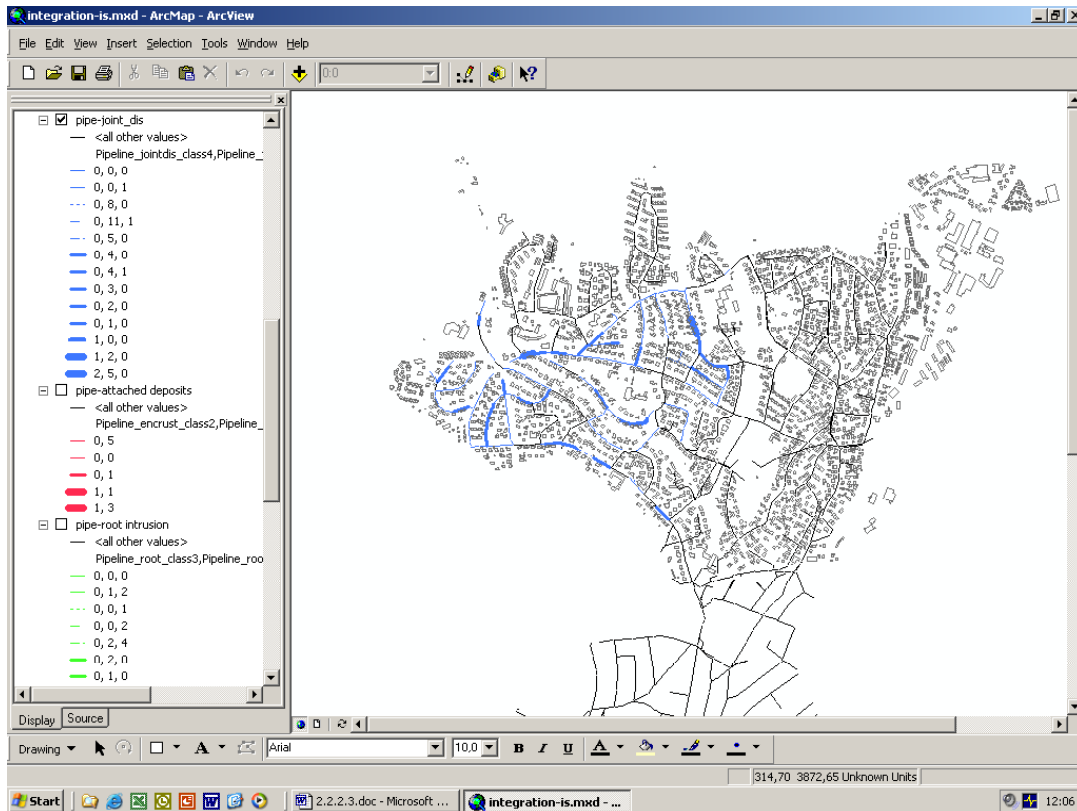


Figure 6.5.1 Spatial distribution of pipeline intrusion (above) and displaced joint (bellow).

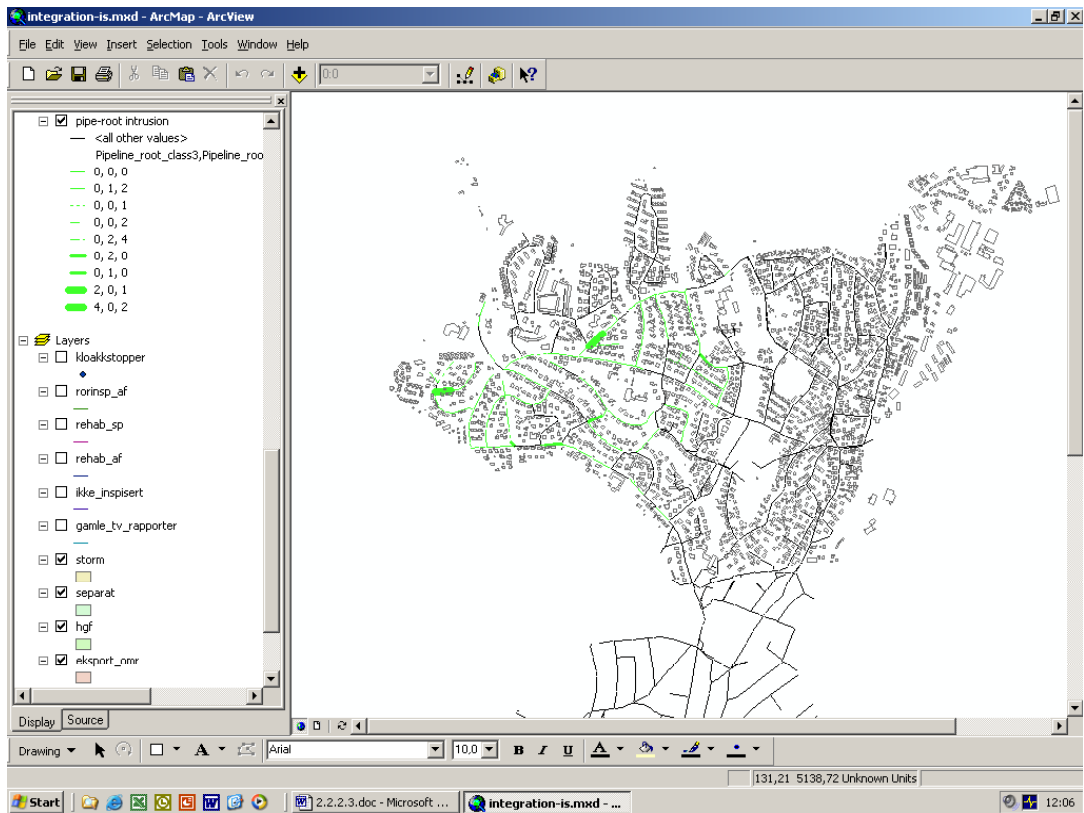
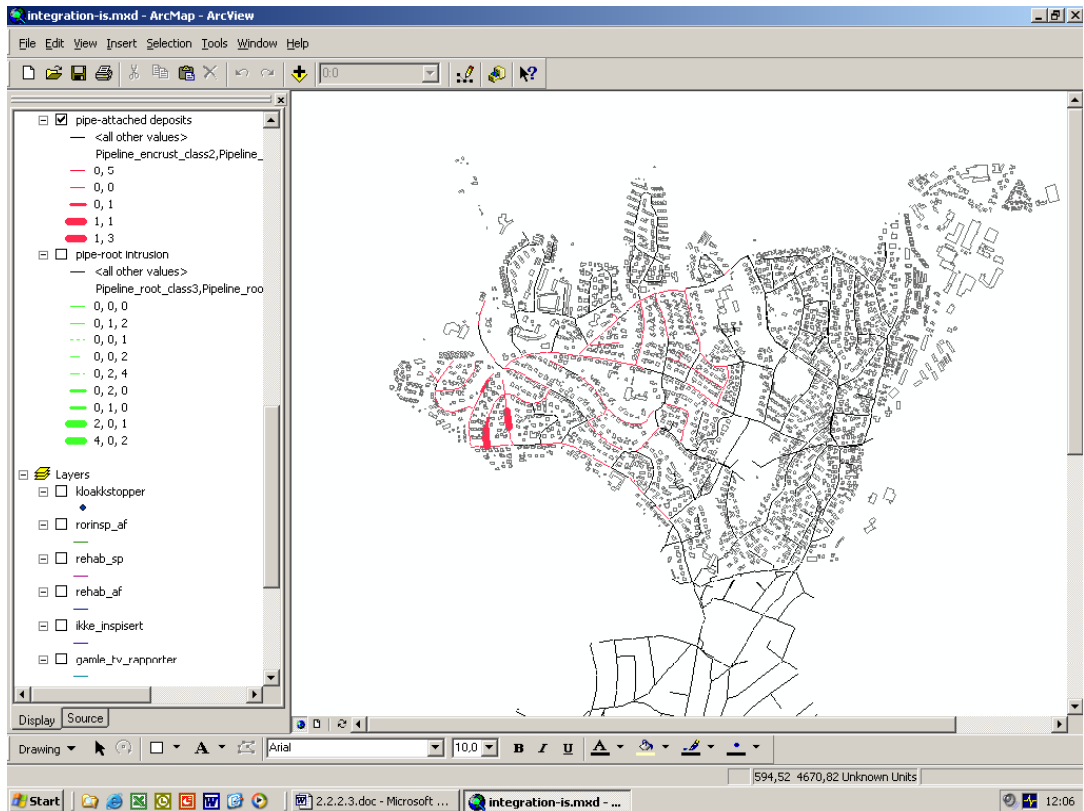


Figure 6.5.2 Spatial distribution of attached deposits (above) and root intrusion (below).

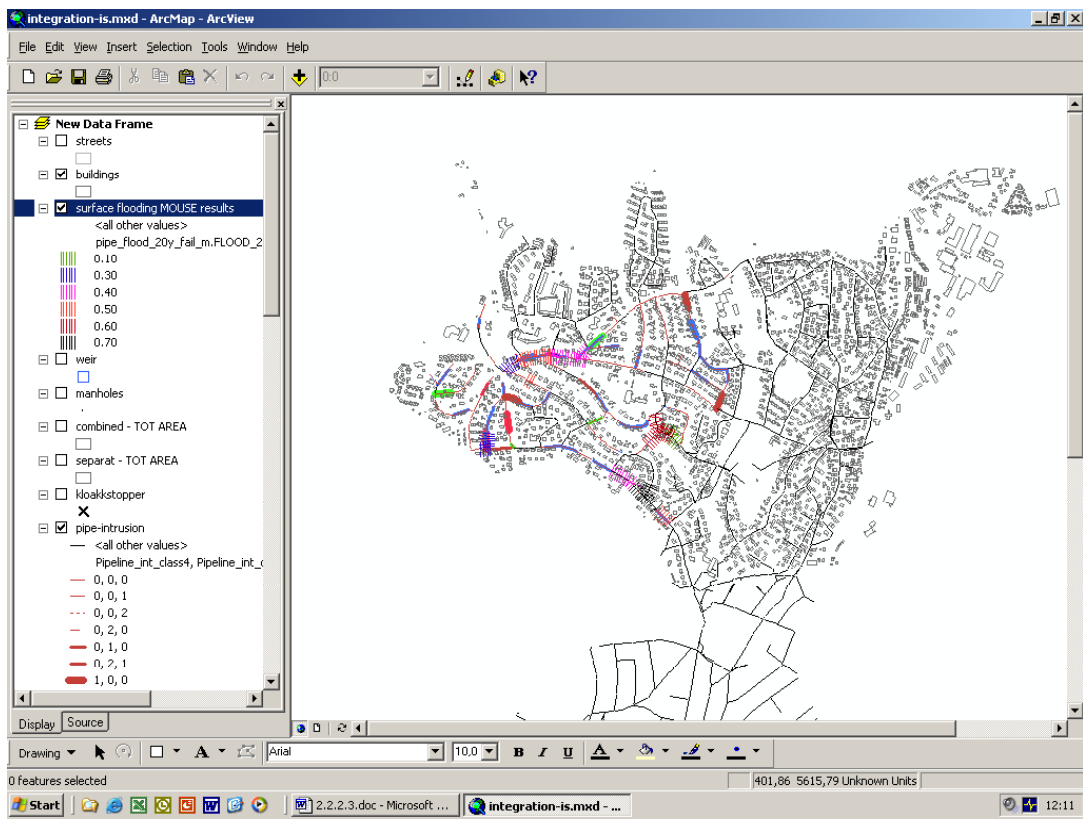


Figure 6.5.3 Different failures integrated in ArcView with the results of hydraulic modelling showing the severity of surface flooding.