



Horizon 2020 Societal challenge 5: Climate action, environment, resource efficiency and raw materials

# BINGO

## Bringing INnovation to onGOing water management -

## a better future under climate change

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Changes with respect to the DoA

Not applicable

**Dissemination and uptake** 

This report is public.

#### Short Summary of results (<250 words)

D3.3 – Calibrated water resources models for past conditions was developed by IWW, LNEC, NTNU, KWR, AQUALOGY, CYI, and all local partners within WP3 - Integrated analysis of the water cycle.

This Deliverable presents the model applications at all six research sites. Special focus in this Deliverable is being paid to the modelling objectives, the model types, the data used and produced, the model results, as well as the model evaluation and discussion. D3.3 shows the wide variability of models developed and applied in order to bring innovation into water management practices. This is due to the fact that European water problems are diverse and BINGO aims at providing as many solutions as possible to mitigate those climate change related problems. Thus, D3.3 should not be seen as a model intercomparison experiment but as an effort to solve multiple water problems together in a community of scientists, local governments, water suppliers and many more stakeholders.

D3.3. is identical with Milestone 16, as the availability of calibrated and working models is essential for the calculation of climate and land-use scenarios.

#### **Evidence of accomplishment**

This report as well as the model output.



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#### 1. INTRODUCTION

This document is developed as part of the BINGO (Bringing INnovation to onGOing water management – a better future under climate change) project, which has received funding from the European Union's Horizon 2020 Research and Innovation programme, under the Grant Agreement number 641739. The Project website (http://www.projectbingo.eu) represents Deliverable 3.3 of Work Package 3 (WP3) – Calibrated water resources models for past conditions.

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#### 2. BINGO Calibrated water resources models for past conditions

The following chapters provide an overview of the hydrological model results for past conditions at the six BINGO research sites in Cyprus, Germany, the Netherlands, Norway, Portugal, and Spain. Each model development/application description is reported per site and includes information on model objectives, model application, model description, data used for modelling, calibration and validation strategies, model results, and model evaluation and discussion. At some sites, e.g. in Portugal and Cyprus, multiple model domains exist. Here, D3.3 provides structured information for each domain.



#### 3. Cyprus

In Cyprus, two case studies are being studied. The Pedieos case study focuses on the impacts of climate and land-use change to single flood events. Thus, HEC-HMS with HEC-RAS models have been employed to simulate the flood profiles and the extent of the flooded area. The Peristerona case study focuses on the impacts of climate change to droughts. Here the use of WRF-Hydro is being explored to couple the atmospheric (WP2) and hydrologic processes (WP3).

#### 3.1. Peristerona Watershed

#### 3.1.1. Model objectives in BINGO

Due to limited rainfall concentrated in the winter months and long dry summers, storage and management of water resources is of paramount importance in Cyprus. For the purpose of water storage, the Cyprus Water Development Department is responsible for the operation of 56 relatively large dams (total volume of 310 Mm<sup>3</sup>) and 51 smaller reservoirs (total volume of 17 Mm<sup>3</sup>) over the island. Climate change is also expected to heavily affect Cyprus water resources with a 1.5 % - 12 % decrease in mean annual rainfall (Camera et al. 2016) projected for the period 2020-2050, relative to 1980-2010. This will make reliable seasonal water inflow forecasts even more important for water managers. The overall aim of this study is to set-up the Weather Research and Forecasting (WRF) model with its hydrologic extension (WRF-Hydro) for seasonal forecasts of water inflow into dams located in the Troodos Mountains of Cyprus. The specific objectives of this study are: i) the calibration of WRF-Hydro for the simulation of past observed seasonal rainfall events and flows in the Troodos Mountains; ii) a sensitivity analysis of the model parameters.

WRF-Hydro was the selected modelling instrument since it allows the coupling of hydrologic and atmospheric processes, perfectly matching the overarching goal of the BINGO project, which is to analyze effects of climate change on the hydrologic cycle. Research on the coupling of atmospheric models with hydrologic models or the improvement of hydrologic processes in land surface component of atmospheric models started during the last decade. Thus, there are relatively few coupled models available (Senatore et al. 2015). WRF-Hydro represents one of the most complete model environments for the analysis of coupled atmospheric-hydrologic processes. The model source code and documentation are freely available and there is an active and growing user community (Gochis et al. 2015). WRF is a well-established and widely used atmospheric model. The Atmosphere and Climate Group of the Cyprus Institute is using WRF for the modeling of past and future climate in WP2.

The WRF-Hydro calibration is conducted in the Peristerona Watershed (Figure 1), located along the northern slopes of the Troodos Mountains. The Peristerona Watershed has an area of 120 km<sup>2</sup> at its outflow in the Serrachis River, just north of the Green Line. The watershed is characterized by an ephemeral river with an average long-term annual stream flow of 11.75 Mm<sup>3</sup> (1980-2010) for the 77 km<sup>2</sup> upstream area, as observed at Panagia Bridge station in the foothills of the mountains (Figure 1 and Figure 2). The main rainfall season is from October to May, while summers are very hot and dry. December, January and February are the wettest months. In the upstream area of the watershed we find sclerophylous vegetation and agricultural terraces with dry stone walls. The fractured volcanic formations in the steep sloping midstream areas are covered by state forests, which are dominated by *Pinus brutia* trees. In the upstream and midstream areas of Peristerona Watershed agricultural lands are often located on terraces next to the streams, while the forested areas covers the steeper slopes above these lands. In the foothills and downstream, both rainfed and irrigated crops can be found. Cereals, especially barley, are the main rainfed crop. Irrigated crops are found on small fields and terraces along the river (olives, vegetables).





Figure 1 Location of the island of Cyprus and WRF-Hydro domain and calibration catchment



Figure 2 Panagia Bridge streamflow station



#### 3.1.2. Model application

#### 3.1.2.1. Model description

The WRF-Hydro model is an extension package of the traditional one-dimensional Noah Land Surface Model (Noah LSM) of WRF that provides a framework for multi-scale representation and multiple physics options for surface overland flow, saturated subsurface flow, channel routing, lake/reservoir routing, and baseflow processes (Gochis et al., 2015). A summary of the model structure is presented in Figure 3.

The Noah LSM allows the modelling of hydrologic processes such as throughfall and interception of rainfall, the re-evaporation of intercepted precipitation, water infiltration in soil, vertical movement of water in the soil, direct evaporation from the soil, surface and subsurface runoff (Ek et al., 2003). Among the outputs, the Noah LSM model can return values of soil moisture (both in liquid and frozen state), soil temperature, skin temperature, snowpack water equivalent, canopy water content, and the energy and moisture fluxes at the earth surface (Niu et al., 2011). The Noah LSM is run on a 1 x 1 km<sup>2</sup> resolution grid.

The WRF-Hydro extension package allows an improvement of the 1D land surface model. Water unable to enter the soil due to infiltration exceedance is not removed from the domain but it is kept as ponded water, laterally redistributed, and summed to streamflow (if it reaches the channel network) or throughfall in the following step (if it does not reach the river network). As reported in Yucel et al. (2015), "*subsurface lateral flow is calculated before the surface routing so that exfiltration from fully saturated soil columns can be combined with existing infiltration capacity excess prior to surface routing. The method used to calculate the lateral flow of moisture in saturated soil columns is that of Wigmosta et al. (1994) and Wigmosta and Lettenmaier (1999), which is also implemented in the Distributed Hydrology Soil Vegetation Model (DHSVM). Overland flow is calculated when the depth of ponded water in a grid cell exceeds a specified retention depth, which is a tunable parameter. Ponded water depths below the retention depth do not move and are subject to future infiltration. A steepest descent directionality explicit diffusive wave formulation of Julien et al. (1995) with a later modification by Ogden (1997) calculates the propagation of shallow overland flow waves". For more details see the description of surface and subsurface routing modules in Gochis et al. (2015).* 

Deep percolation might be added to channel discharge as baseflow with the use of an optional exponential bucket model, which is based on an empirical-derived function of recharge (see Gochis et al. 2015 for more details). However, preliminary runs indicated that little improvements could be achieved using the baseflow routine, while being computationally costly to run. Similar findings were reported by Senatore et al. (2015). For these reasons, the baseflow routine is not activated in the current study,. The channel routing module makes use of an algorithm based on the explicit, one-dimensional, variable time-stepping diffusive wave formulation, which is a simplification of the more general St. Venant equations (Gochis et al. 2015 for more details). WRF-Hydro also includes an optional routing sub-routine to model water storage and outflow from reservoirs along the channel network. The sub-routine makes its calculation based on mass balance equations and a level-pool threshold so to estimate the impact of reservoirs on hydrologic response (Gochis et al. 2015). However, also this routine is not activated in this study. Our current research focusses on the areas uphill of reservoirs since our main aim is the simulation of dam inflow.

For each LSM time step (1 day in our case), soil column moisture states (maximum soil moisture, infiltration capacity excess, lateral saturated hydraulic conductivity and soil moisture content) are disaggregated and



passed to the WRF-Hydro routines (Yucel et al. 2015). WRF-Hydro routines

can in fact make use of a higher resolution grid, which needs to include an integer number of cells inside a single pixel of the LSM grid. In our study we use a high resolution grid of 100 x 100 m<sup>2</sup>, thus having 100 cells inside each pixel of the LSM model. For this high resolution grid, some additional base maps need to be defined: latitude, longitude, topography, flow direction, channel network, stream order, retention depth routing factor, overland roughness routing factor, and location of streamflow gauges for exporting modelled river discharge. The time step for the routing modules is set to 6 seconds, as suggested in the WRF Hydro User's Guide (Gochis et al. 2015) for grids with a horizontal resolution of 100 m.





#### 3.1.2.2. Data

For parameterization of the Noah LSM, all the necessary base maps - including soil type, landuse categories, monthly green fraction, monthly leaf area index and elevation – were derived from MODIS datasets with resolution 30" (around 900 m) and re-interpolated on the 1 x 1 km<sup>2</sup> grid specified in section 3.1.2.1 (Model description). Initial values of soil moisture and soil temperature were derived for four soil layers (0-10 cm, 10-40 cm, 40-100 cm, 100-200 cm) using field data as a reference. These data come from 5TM DECAGON sensors that were installed for the BINGO project (Task 3.2) at two environmental monitoring stations Cyl is running in the Peristerona Watershed and in its proximity. The two stations are located in Agia Marina/Xyliatos forestry area (around 600 m a.s.l., just outside the watershed boundary), and in the terraced slopes uphill the village of Alona (around 1300 m a.s.l., inside the watershed boundaries). The set initial values were considered constant over the entire model domain (see Figure 1) and were used to initialize the spin-up runs of the model.

Most of the high resolution datasets (100 x 100 m<sup>2</sup>) necessary to run the WRF-Hydro model routines (see section 3.1.2.1 Model Description for the detailed list) were derived using the hydrology toolbox of ArcGIS. The main input was the digital elevation model (DEM) of the island of Cyprus, provided by the Cyprus



Geological Survey Department, with an original cell size of 25 x 25 m<sup>2</sup> and re-

scaled to the selected 100 m horizontal resolution. In detail, the maps derived using these methods are: flow direction, stream order, channel network (cutoff threshold of 250,000 m<sup>2</sup> contribution to be considered a stream cell), and watershed areas. Latitude and longitude where automatically extracted with ArcGIS functions. The exact location of the monitoring stream gauge (Panagia Bridge station) to generate time series of modelled streamflow were provided by the Cyprus Water Development Department, together with daily average discharge data for the period 1980-2010. The retention depth and overland roughness routing factors were adjusted during calibration.

The meteorological data necessary to run WRF-Hydro coupled with the Noah LSM model include: incoming shortwave and longwave radiation (W/m<sup>2</sup>), specific humidity (kg/kg), air temperature (K), surface pressure (Pa), near surface wind in the u- and v- components (m/s), and precipitation (mm). For our simulations we used daily forcing. Precipitation was input from the observed daily gridded dataset (1 x 1 km<sup>2</sup>) that was produced for Cyprus - interpolating data from 145 stations - for the period 1980-2010 by Camera et al. (2014). All the other data came from long term (2000-2009) WRF simulations run with initial conditions constrained, and boundaries forced every 6 hours, with ERA-INTERIM reanalysis data. The original horizontal resolution of these simulations is 12 km but data were re-gridded on the 1-km Noah LSM grid for input. These simulations are part of the climate modeling activities of BINGO WP2.

#### 3.1.2.3. Calibration and validation strategies

WRF-Hydro is calibrated and validated over 1-year periods (October 2006 – September 2007 for calibration, October 2000 – September 2001 for validation) following a single month spin-up (September). The two periods were selected as years with an annual rainfall close to the long-term average and with the presence of multiple extreme rainfall and flow events. The spin-up is limited to a single month due to the very dry conditions occurring at the end of the summer period in Cyprus. Soil moisture is very low and rivers are usually completely dry, so not much time is needed to bring conditions to the actual initial state. The initial soil moisture and soil temperature values discussed in the previous section are presented in Table 1. As initial conditions for the spin-up run, the rivers were considered completely dry (see HLINK in Table 2).

|              | Depth [cm] | Moisture [kg/kg] | Temperature [K] |
|--------------|------------|------------------|-----------------|
| Soil layer 1 | 0-10       | 0.06             | 298             |
| Soil layer 2 | 10-40      | 0.07             | 296             |
| Soil layer 3 | 40-100     | 0.10             | 292             |
| Soil layer 4 | 100-200    | 0.12             | 290             |

Calibration was performed manually with a stepwise approach since the use of an automated calibration method would have required a very high number of model runs and an excessive computational time. Calibration focused on two aspects: the average streamflow for the whole calibration period, and the match between observed and modelled time series. Four statistics were derived for the second purpose: i) Nash-Sutcliffe Efficiency (NSE), to assess the fit between observed and modelled time series with emphasis on high-flow simulations; ii) logarithmic Nash-Sutcliffe efficiency (LNSE) to assess the fit between observed and modelled time series with emphasis on high-flow simulations; iii) logarithmic Nash-Sutcliffe efficiency (LNSE) to assess the fit between observed and modelled time series with emphasis on low-flow simulations (Oudin et al. 2006); iii) mean absolute error (MAE); and iv) root mean squared error (RMSE) to describe the difference between the model simulations and observations in the units of the variable (Legates and McCabe, 1999). The calibration was



considered successful for errors in the average streamflow lower than 20%,

positive NSE and LNSE coefficients, and maximum MAE comparable to the average daily streamflow. Following the indications of previous WRF-hydro studies (Senatore et al. 2015; Yucel et al. 2015; Arnault et al. 2016; Givati et al. 2016) calibration focused on the following parameters: partitioning of rainfall into runoff and infiltration (REFKDT); empirical coefficient ranging from 0.1 and 1 which controls the drainage out the bottom of the deeper soil layer (SLOPE); water retention depth routing factor (RETDEPRTFAC); overland roughness routing factor (OVROUGHRTFAC); and channel Manning coefficients factor (MANN). WRF-Hydro assigns channel Manning coefficients, together with other channel properties, through a simple table based on stream order (Table 2). For this reason, instead of changing Manning coefficients for each stream order independently, during the calibration phase all the values were adjusted through a multiplicative factor (MANN), which is the actual calibrated parameter. This is the same approach as used by Yucel et al. (2015). The reported Manning roughness values for the Peristerona Watershed were artificially high, because here we calibrated WRF-Hydro with observed daily rainfall data. New simulations with observed hourly data had Manning roughness coefficients of 0.14, 0.12, 0.9, for the first, second and third order streams, respectively. In the Pedieos Watershed the Manning coefficient for the upstream area, which is somewhat similar to Peristerona Watershed, were not calibrated, because of low sensitivity. The selected Manning coefficients ranged between 0.045 and 0.060.

Firstly, the parameters controlling the volume were calibrated following the order REFKDT, SLOPE, RETDEPRTFAC. Secondly, those controlling the shape of the hydrograph were tuned following the order OVROUGHRTFAC, MANN. Once a parameter was considered tuned, it was not modified anymore. REFKDT, SLOPE, and MANN parameters are pre-defined tabulated values constant for definition over the all model domain, while RETDEPRT and OVROUGHRT parameters are pixel specific (Yucel et al. 2015). For this study we kept also the mapped values constant throughout the modelled catchment. All the other parameter values were kept at their default values as provided in the hydrologic (HYDRO.TBL) and general parameters (GENPARM.TBL) tables of WRF-Hydro V3.0 release. These parameters include overland roughness coefficients defined based on landuse type and several parameters defined on soil type (saturated soil hydraulic conductivity, maximum volumetric soil moisture value, reference volumetric soil moisture value wilting point for volumetric soil water).

The calibrated parameters were investigated for sensitivity, too. For this purpose, the best (calibrated) value for each parameter was increased or decreased by 20% and the results analyzed in terms of average daily discharge impact, and statistics impacts. Also, a sensitivity index (SI) was derived as follows:

$$SI = \frac{(AvFlow_{Cal} - AvFlow_{Sen})/AvFlow_{Cal}}{(Par_{Cal} - Par_{Sen})/Par_{Cal}}$$
(1)

where AvFlowCal is the daily average stream flow resulting from the calibrated model, AvFlowSen is the daily average steamflow resulting from a parameter modification, ParCal is the calibrated value of the parameter under analysis and PArSen is the modified value of the parameter under analysis.



Table 2 Default channel parameter values, based on stream order, used for WRF-Hydro simulations in this study. Bw is the channel base width, HLINK the initial water depth at the beginning of the spinup period, Ch SSIp is the channel slope and MannN the Manning coefficient

| Stream order | Bw [m] | HLINK [m] | Ch SSIp [%] | MannN[-] |
|--------------|--------|-----------|-------------|----------|
| 1            | 5      | 0.00      | 1           | 0.65     |
| 2            | 10     | 0.00      | 0.6         | 0.50     |
| 3            | 20     | 0.00      | 0.3         | 0.45     |



#### 3.1.2.4. Results

The time series comparisons between observed and modelled streamflow for the calibration and validation years are presented in Figure 4 together with the statistics for model evaluation. Total rainfall over the watershed upstream of Panagia Bridge was 714.1 mm during the 2006/07 calibration season and 623.6 mm during the 2000/01 validation season. The observed average daily streamflow for the calibration period is 0.29 m<sup>3</sup>/s, while the calibrated modelled value is 0.23 m<sup>3</sup>/s. For the validation period the observed average daily streamflow is 0.30 m<sup>3</sup>/s, while the modelled value is 0.23 m<sup>3</sup>/s. In Figure 5 zoom-in of the two main extreme events that occurred during the calibration period is presented. The calibrated parameters are presented in Table 3, while the results of the sensitivity analysis are summarized in Table 4.



Figure 4 Comparison between observed and modelled time series for calibration (October 2006 – September 2007, on the left) and validation (October 2000 – September 2001, on the right) time periods. Flow obs is the daily average observed flow at Panagia Bridge station, flow mod the corresponding modelled streamflow and rain obs is the observed rainfall. NSE is Nash-Sutcliffe efficiency, LNSE is logarithmic Nash-Sutcliffe efficiency, MAE is mean absolute error, and RMSE is root mean squared error



Figure 5 The two extreme events that occurred during the calibration year in February 2007 (left) and in May 2007 (right). Flow obs is the daily average observed flow at Panagia Bridge station, flow mod is the corresponding modelled streamflow and rain obs is the observed rainfall



Table 3 Values of the calibrated model parameters. The parameter REFKDT controls the partitioning of rainfall into runoff and infiltration, LOSS\_BASE controls the partitioning of deep percolation between losses and baseflow contribution, RETDEPRTFAC is the water retention depth factor, OVROUGHRTFAC is the overland flow roughness factor, and MANN is the channel Manning coefficient

|                    | REFKDT | LOSS_BASE | RETDEPRTFAC | OVROUGHRTFAC | MANN |
|--------------------|--------|-----------|-------------|--------------|------|
| Calibration values | 0.01   | 0.1       | 0.2         | 1            | 10   |

Table 4 Sensitivity analysis results expressed in terms of Nash-Sutcliffe efficiency (NSE), logarithmic Nash-Sutcliffe efficiency (LNSE), mean absolute error (MAE), root mean squared error (RMSE), and sensitivity index (SI). Parameters were changed  $\pm 20\%$  based on their calibrated values. See Table 3 for parameter descriptions

|                   | NSE   | LNSE | MAE        | RMSE       | SI    |
|-------------------|-------|------|------------|------------|-------|
|                   | [-]   | [-]  | [m3/s day] | [m3/s day] | [-]   |
| Calibration       | 0.13  | 0.23 | 0.30       | 0.77       | NA    |
| REFKDT +20%       | 0.15  | 0.21 | 0.29       | 0.76       | -0.18 |
| REFKDT -20%       | 0.10  | 0.22 | 0.30       | 0.78       | +0.30 |
| LOSS_BASE +20%    | 0.13  | 0.22 | 0.30       | 0.76       | 0.00  |
| LOSS_BASE -20%    | 0.13  | 0.22 | 0.30       | 0.76       | 0.00  |
| RETDEPRTFAC +20%  | 0.13  | 0.24 | 0.30       | 0.77       | +0.01 |
| RETDEPRTFAC -20%  | 0.13  | 0.23 | 0.30       | 0.77       | -0.01 |
| MANN +20%         | 0.19  | 0.22 | 0.28       | 0.74       | -0.03 |
| MANN -20%         | -0.03 | 0.16 | 0.32       | 0.83       | +0.10 |
| OVROUGHRTFAC +20% | 0.13  | 0.21 | 0.30       | 0.76       | -0.02 |
| OVROUGHRTFAC -20% | 0.13  | 0.22 | 0.30       | 0.77       | +0.03 |

#### 3.1.3. Model evaluation and discussion

The NSE value obtained during the calibration of the model cannot be considered a good result. According to Moriasi et al. (2007) a satisfactory NSE value is greater than 0.5. However, some considerations need to be made. Our study deals with ephemeral streams in semi-arid environments, which represent peculiar conditions particularly difficult to model. This is mainly due to generally (intermittent) low flow and sudden peaks. A higher value was obtained for the LNSE than for the standard NSE, which is a better evaluation metric for low flows than the NSE. Also, considering peak flows, it should be noted that the model is run with daily data and precipitation is applied all at once at midnight of each day, while the following routing is performed with a 6 seconds time step. Very high Manning coefficients were used to delay the peak to the next day, to match the observations, but it is not always sufficient. Another shortcoming of modelling flow peaks can be seen in Figure 5. For both examples, the second rainfall peak does not produce a second rise of the hydrograph, although in many other circumstances the model demonstrated to react fast to rainfall input. This model behavior is still under investigation. Conversely, the first discharge peak is usually



overestimated. The relative high MAE and RMSE (comparable to average daily streamflow) also point out this general underestimation and overestimation of peak flows.

The results of the sensitivity analysis strengthen the statement that the main problems of the model relate to extreme behavior. This is pointed out by a LNSE value similar to that of the calibration run but a much worse NSE value, which is even negative for the sensitivity run. Also, comparing calibration and validation it is possible to notice that in both cases the peaks at the beginning of the rainfall season are overestimated, while the peaks in the middle and end of the rainy season are usually underestimated. Probably, soil and watershed conditions are changing radically over the season, so that hydrologic processes are not comparable, and the model, although physically based, does not have the flexibility to catch those modifications with parameter values that are constant over the year. As an example, it is probable that rainfall partitioning into runoff and infiltration - which is controlled by REFKDT, one of the most sensitive parameters of the model (Table 4) – changes during the year, possibly even after each rainfall event.

The channel Manning coefficients, as already said, are set to very high values, which makes this parameter more a calibration than an actual physical parameter. However, because of the intermittent nature of the flows and sedimentation due to erosion of degrading mountain terraces in the upstream areas, heavy vegetation growth can be observed inside the river bed, especially for streams of low order (Figure 6).

The results of the sensitivity analysis show that REFKDT and MANN are the most sensitive parameters (highest SI values), affecting both the average daily streamflow (volume) over the modelling period and the fit and error statistics. REFKDT is suggested as the most sensitive parameter also by other authors (Senatore et al. 2015; Arnault et al. 2016; Givati et al. 2016). The high sensitivity of the MANN parameter could be partly due to the combination of very high calibrated values and the sensitivity analysis strategy of modifying values of ±20%, resulting in large differences between the values used for the calibration and the sensitivity runs.



Figure 6 Overgrown streambed in the upstream areas of Peristerona Watershed, in Platanistasa (left) and Fterikoudi (right)



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#### 3.2. Pedieos Watershed

#### 3.2.1. Model objectives in BINGO

The six research sites that BINGO is built around, include a Cyprus part focused on the Pedieos watershed (see Figure 7) for which climatic impacts such as floods are evaluated. The development of a hydrologic model calibrated over past conditions will enable assessment and management for coping with future climate change challenges.

As per the objectives of WP2 – "Climate predictions and downscaling to extreme weather", Task 2.3 "Highresolution downscaling (3-1km/1h) for floods" for the research site of Pedieos watershed in Cyprus, the episode of the extreme precipitation event of 8<sup>th</sup> - 11<sup>th</sup> of January of 1989 of present climate reanalysis and decadal climate predictions to a high resolution for use in the hydrological and hydraulic model was performed.

These rainfall data and as per the objectives of WP3 – "Integrated analysis of the water cycle" the model performance for Pedieos watershed at the Cyprus site has been run, calibrated on the basis of past conditions (rain event of  $8^{th} - 11^{th}$  of January 1989) and evaluated. This will allow subsequently a step by step adjustment to become a reliable tool for local users, for future climate events.

The Cyprus Research Site is located along the northern slopes of the Troodos Mountains in Cyprus. The Troodos Mountains form the water tower of the island, with many streams running down its steep slopes, in deeply incised valleys (see Deliverable 3.1). The northern slopes are in the rain shadow of the mountains and are less endowed with water resources than the southern slopes. Investigations in the agro-ecological and hydrological processes along the northern slopes of the Troodos Mountains could also present insights in the potential effects of climate change on the southern slopes.



Figure 7 The Troodos mountains in Cyprus with the Peristerona and Pedieos Watersheds along the northern slopes



The water resources model focusses on the Pedieos watershed which is one

of the two representative watersheds along the northern slopes of the Troodos Mountains: The Pedieos and the Peristerona Watersheds (Figure 7). This river with a watershed area of 120 km<sup>2</sup> flows across the buffer zone into the northern part of the island, inhabited by the Turkish Cypriot community. Figure 8 shows the Pedieos sub basins, the two weirs just upstream the Tamassos Dam and the grid of the downscaled rainfall.



Figure 8 The Pedieos basin, subbasins, weirs location and the grid of downscaled rainfall at  $1 \times 1$  Resolution



Flood modelling studies covering, among others, the Pedieos watershed, have been conducted by the Water Development Department as a requirement of the European Flood Directive (2007/60/EC). Flood hazard and flood risk maps and a flood management plan have been prepared for the flood sensitive areas (WDD 2015).

The flood modelling which has been conducted for the Pedieos Watershed by WDD was with the use of the HEC (Hydrologic Engineering Center) models, developed by the U.S. Army Corps of Engineers (http://www.hec.usace.army.mil/software/). Both the Hydrologic Modeling System (HEC-HMS) and the River Analysis System (HEC-RAS) were used.

In view of the above, and so that this work is comparable and compatible with existing studies, the models which were chosen to be used for BINGO are the same, namely the HEC-HMS rainfall /runoff model and the HEC-RAS together with the HEC-GeoRas for the hydraulic modelling and the flooding configuration.

#### 3.2.2. Model application

#### 3.2.2.1. Model description

A brief description of the models that were used is provided here below together with some graphical information (main screen, or user interface, or process flowchart):

#### > The HEC-HMS (Hydrologic Engineering Center – Hydrologic Modeling System)

HEC-HMS is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is designed to be applicable in a wide range of geographic areas for solving a broad range of problems. This includes large river basin water supply and flood hydrology to small urban or natural watershed runoff. Hydrographs produced by the program can be used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, wetlands hydrology, and systems operation. Figure 9 shows the "main software screen" as used for the Pedieos model.

#### > The HEC-RAS (Hydrologic Engineering Center – River Analysis System)

HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking environment. The system is comprised of a graphical user interface, separate analysis components, data storage and management capabilities, graphics and reporting facilities.

The HEC-RAS system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines. In addition to the four river analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed. Figure 10 shows the user interface of the HEC-RAS software.

#### > The HEC- GeoRAS (Hydrologic Engineering Center – Geospatial River Analysis System)

HEC-GeoRAS is set of ArcGIS tools specifically designed to process geospatial data for use with the Hydrologic Engineering Centre's River Analysis System (HEC-RAS). The extension allows users to create an HEC-RAS import file containing geometric data from an existing digital terrain model (DTM) and



complementary data sets. Results exported from HEC-RAS may also be

processed for the production of the flood maps. Figure 11 shows the "Process flow Diagram" of the HEC-GeoRAS.



Figure 9 The event model structure and the main program screen of the HEC-HMS as used for Pedieos watershed



Figure 10 Main user interface of HEC-RAS





Figure 11 Process flow diagram of HEC-GeoRAS

#### 3.2.2.2. Data

#### 3.2.2.2.1. Parameter input and data of the WDD Hydrologic Model for Pedieos

The HEC-HMS Hydrologic Model for the Pedieos watershed as it was calibrated by the Water Development Department was used for the BINGO WRF precipitation configurations. As such, no further calibration or adjustment of the model was made, so that the results of the present work are comparable and compatible to those of the WDD which are already available to decision and stake-holders of the area. Some pertinent points on the parameters used for the WDD model and assumed as input data are as follows:



#### Delineation of the basins:

This was carried out with the tools of Geo-HMS. The original DEM was modified and a new DEM, with a linear increase of the elevations to the desired limits of the sub-basins except where streams pass and the ground is not changed, was done (with the use of the command "Build wall".)

Mean basin slope:

Automatically determined with the use of the DEM. The Geo-HMS tool was used for the calculation of the percent mean slope of each basin used on the slope grid determined of the larger area.

Curve Number (CN):

This was determined in ArcMAP, based on soil characteristics defined by the layers Soil250k, for the whole area, but at reduced analysis, and Soil250k covering only part of the basin but at higher analysis. For each soil group in the area, the four hydrological soil groups A to D were determined<sup>1</sup>. The land use map of Corine2006 was used and compared to the SCS tables of TR-55 as well as to other papers on the subject. A CNGrid raster was finally constructed with a value for CN at each cell.

Dimensions of the stream:

These were based on LiDAR where this was available. Elsewhere, these were based on site visits and on Google Earth.

Manning's roughness coefficients:

These were based on evidence on hydraulic structures from site visits with values based on literature<sup>2</sup>.

Stream gradient:

Done by ArcMAP and the GeoHMS tool.

Hydraulic evaluations:

All the hydraulic evaluations were done with HEC-RAS for non-steady flow. Conditions of subcritical flow were adjusted accordingly in areas where flow becomes supercritical (Mixed flow regime).

Geometry for hydraulic model:

Carried out with the use of HEC-GeoRAS in ArcGIS with micro-adjustments with HEC-RAS.

Info used for the hydraulic model:

Land model as per LiDAR, aerial photos during LiDAR, 5-meter Raster DEM of WDD, Photos from field visits for evaluating coefficients of linear friction losses, sketches of dimensions of bridges, culverts etc.

Boundary conditions:

Location of entry of hydrographs: a) u/s entry Flow Hydrograph, b) from side basins Lateral Inflow Hydrograph, and c) in between sub-streams Uniform Lateral Inflow)

Normal Depth assumed = stream gradient (=0.65%)

Linear Losses

The evaluation of coefficients of linear losses is carried out using the Manning's coefficient and the Cowan method<sup>3</sup>.

<sup>&</sup>lt;sup>1</sup> Part 630 Hydrology National Engineering Handbook of Natural Resources Conservation Service (ex SCS) – Chapter 7, Hydrologic Soil Groups

<sup>&</sup>lt;sup>2</sup> Ven Te Chow: Open Channel Hydraulics, table D page 112

<sup>&</sup>lt;sup>3</sup> Ven Te Chow (1959): Open Channel Hydraulics pg 106-109


The coefficient of local losses at the entry of culverts in HEC-RAS are as listed in its manual  $\!\!^4$ 

## Method used for the Time of Concentration:

The method described in the TR-55 publication of "The Natural Resource Conservation Service (NRCS (ex SCS)" was used. The data needed are:

- The 24-hour rainfall for a 2-year return period,
- The basin slope,
- The flow distance on the land surface for the three flow conditions as defined by NRCS and which are: sheet flow, shallow concentrated flow and stream flow.

The time of concentration is calculated as the sum of the time of concentration of the upstream basin and the flow time of each sub-basin which is included in the main flow path of the stream.

The 24-hour rainfall for a 2-year return period is computed, for each sub-basin, by considering the weight of each meteorological station on the sub-basin. These values are written on the Table of ArcMAP. Subsequently, and using the tools of HEC-GeoHMS, the  $t_c$  (concentration time) is evaluated, for the definition of which, a roughness coefficient for "sheet flow" is given (as shown in Table 3.1 in the WDD Final report for Pedieos).

The stream gradient is derived by ArcMAP and the GeoHMS tool.

> Conversion of point rainfall to areal:

For the cases of large (>20km<sup>2</sup>) and very large (>150km<sup>2</sup>) basins the point rainfall is reduced to areal by using the diagrams provided by WMO<sup>5</sup>.

The assumed ARF (Area Reduction Factors) are:

- ARF= 0.93 was assumed for the total of the basin at the point of discharge (122.44 km<sup>2</sup> and  $t_c$ =9.6 hours)
- ARF=0.98 at the weir location 6-1-1-85 (30.18km<sup>2</sup> and t<sub>c</sub> = 3.26 hours)
- At weir 6-1-1-80 no reduction factor was used (<20 km<sup>2</sup>).

The Area Reduction Factors (ARF) are included in the Simulation Runs for each position as a ratio of rainfall.

- ength of data series:
- The maximum annual values (m<sup>3</sup>/s) of the period 1968 to 2010 (or 42 values available with the year 1973 missing) were used for the weir 6-1-1-80.
- The maximum annual values (m<sup>3</sup>/s) of the period 1968 to 2001 (or 33 values available with the year 1973 missing) were used for the weir 6-1-1-85.
- > Probability Distribution:
- The selected as the most suitable distribution was the EV2-Max (L-Moments) after examining all possible distributions.
- Validation:
- A comparison of the peak flows obtained by the statistical analysis of the measured annual maximum values for 20, 100 and 500 years was made with the peak flows derived by the hydrological models based on design storms of such frequency.

<sup>&</sup>lt;sup>4</sup> HEC-RAS - Hydraulic Reference Manual AND Hydraulic Design of Highway Culverts (FHWA 1985).

<sup>&</sup>lt;sup>5</sup> Ven Te Chow et al, 1988: Applied Hydrology, International Edition.



## Initial Moisture (Ia):

This is based on the SCS instructions and constitutes 20 % of the maximum storage S on the occasion that the antecedent soil condition is wet or dry, and is a function of the coefficient of CN, where:

## Ia = 0.2xS, and $S(mm) = (25400-254xCN)/CN^{-6}$ thus $Ia = 0.20 \times (25400-254 \times CN)/CN)$

## > Antecedent soil moisture conditions before the start of storm:

For the simulation of the 20-year storm event, a mean initial moisture was assumed for which the CN was calculated and the initial losses were evaluated.

For the 100 and 500-year storm event, the soil is assumed saturated and the initial conditions were considered wet (AMCIII) and the relevant CN values and initial losses were accordingly calculated.

Additional models with other antecedent soil moisture conditions (wet or dry) instead of the initial model which assumed mean moisture conditions were examined<sup>7</sup>.

The SCS Synthetic Unit Hydrograph (UH) used:

This is a single-peak dimensionless UH. This UH shows the flow of Ut as a ratio to the peak flow Up for each time t, a fraction of Tp, the time to peak, Up.

Where:

- Up=C (A/Tp) where A= area, and C=2.08, and
- Tp=(Δt/2)+tlag where Δt=duration of excess rain, and tlag = basin time lag (time difference between the mass of excess rain and the peak flow of the UH)

and

- Ut is the volume of the Unit Hydrograph (m<sup>3</sup>)
- Up is the unit hydrograph peak discharge (m<sup>3</sup>/s)
- Tp is the time to peak discharge (hr)
- A is the area of the watershed (km<sup>2</sup>)
- The coefficient C of 2.08 is for unit rainfall of 1 cm (In the case study the rainfall input to the model is in mm)

With known Tp and Up the UH could be constructed from the dimensionless shape.

For the basins that have no rainfall and runoff measurements tlag (lag-time) can be related to the time of concentration tc, as: tlag =0.6tc

> Summary of the calibration procedure:

Initially for all events, a choice of moisture conditions on the basis of antecedent rainfall was made.

This was followed by optimization of the basin hydrologic parameters at all stations for all events, through the optimization trials of HEC-HMS.

The SCS model that is used is an "event model" and not for long-term simulation (continuous). Thus some historic rain graphs that were available with the WDD could not be accepted. In this case the calibration aimed at the optimal modelling of the first main peak event.

The parameter values of the optimization trials of a single event or the mean of these from a number of single events for which the procedure gave values of the same order, were tried in the remaining historic events.

<sup>&</sup>lt;sup>6</sup> Soil Conservation Service (USA) where (S (inches)=1000/CN -10)

<sup>&</sup>lt;sup>7</sup> Ven Te Chow et al 1988: Applied Hydrology, International Edition Page 149



The CN, the quantity of initial losses and the lag time of the basins are the

parameters which were modified for improved model adaptation to observed values during calibration.

Finally, the optimal values of these parameters that gave the optimal result to most of the historic cases were selected.

> <u>Time step used for calibration/validation runs:</u>

Historic floods used for the calibration procedure:

- Floods of 09/01/1989 and 12/02/2003 at the weir 6-1-1-80
- Floods of 09/01/1989 and 02/12/1992 at the weir 6-1-1-85

both weirs are upstream of the Tamassos Dam.

The storm data are introduced as "Time series Data", "Precipitation Gage".

➤ <u>Time step:</u>

A time window was set up starting at 08:05 am and lasting for some multiple of 24hours. The time window normally covered 2 days before the time of the peak flow and 1 day after.

This way, the information on the amount of rain of the previous days is given and thus an evaluation of the soil moisture condition can be made when the storm event, that is to be used for the calibration, starts.

The time step was 5 min. For the cases that the time step of 5 min was greater than (0.29\*tlag) for any basin, then the time step of 1 min was applied. This did not produce any changes to the results.

## Evaluation method for the calibration run

Calibration by comparison with statistically estimated flood peak flows:

Analysis of flood frequency is done by use of the software "Hydrogromon Ver. 4.1.0" developed at the National Technical University of Athens, Greece. Use of the sub-system "Pythia-Statistical Analysis" was made where after introducing the maximum annual rainfall there is the capability to examine which theoretical distribution fits best to the sample with the maximum annual values for each station. The suitability control is made with the X<sup>2</sup> test with an importance level for a=10 %.

Finally, the best distribution was selected and the flow at the return period of 20, 100 and 500 years was selected, as shown below:

#### Independent calibration and validation

Peak flow estimation was made for validation using Fuller's equation<sup>8</sup>:

 $Q=Qi \times F0.8 \times (1 + 0.80 \times logT) \times \{1+(2.66 / F0.30)\}$ 

Where:

Q is flow in m<sup>3</sup>/s, F the area of the basin in km<sup>2</sup>, T the period and Qi a coefficient depending on the return period.

<sup>&</sup>lt;sup>8</sup> Fuller and the incremental coefficients are derived from the Directions of Writing Studies of Road works ( $O.\Sigma.M.O.$  chapter 8, revision A3) of the Egnatia Road A.E. (WDD 2015)



Fuller's is an approximation of flow being used when there are no detailed data for the characteristics of the basin and of rainfall. The incremental coefficients Qi that have been used

are 2.0 for 20-year, 2.6 for the 100-year and 3.0 for the 500-year.

The Fuller equation has been developed from statistical analysis of a large number of basins and accepts input on the extent of the area and the return period. It is normally used for evaluation of results computed more accurately by other methods that require more data.

No calibration was made for the hydraulic evaluations, which is the flooding through the use of the HEC-RAS model, downstream the weirs 6-1-1-80 and 6-1-1-85 since no flow data exist within this part of the examined stream. The calibration was made only for the hydrologic evaluation (HEC-HMS) at the point of the two weirs.

# 3.2.2.2.2. Sources and Input data, together with format, spatial and temporal resolution for the BINGO Hydrologic Model for Pedieos watershed

For the present model application and for calibration on past conditions an ensemble set of five precipitation configurations was used. Each rainfall configuration was used as input data for the calibration by the WDD model, and its response in terms of runoff is evaluated. In addition to this, the mean precipitation of the ensemble set was examined as well. The ensemble set was derived as follows:

Precipitation output from WP2 was used as input for the hydrological modelling. In more detail, extreme precipitation events of the recent past were simulated using the widely-used Weather Research and Forecasting (WRF) model (Skamarock et al. 2008) for the research site of Cyprus. The coarse-resolution global ERA-Interim reanalysis dataset (Dee et al. 2011) was dynamically downscaled to 1-km (1-hour) horizontal (time) resolution.

The most crucial atmospheric model components for rainfall generation are the parameterization schemes of convection and microphysical processes that occur in sub-grid scales. These processes in atmospheric models are empirically described either because the complexity and small scales involved make them too expensive to be modelled or because there is insufficient knowledge about a specific process to represent it mathematically (Warner 2011). Therefore, after testing a large number of microphysics and convection parameterization schemes in WP2, our final ensemble set includes precipitation output from five WRF configurations.

The WRF ensemble set is made up of five configurations "p16, p17, p18, p19 and p20" as shown in the following Figures. The precipitation data series was confined to the Pedieos watershed for the storm event of the 8<sup>th</sup> to 11<sup>th</sup> of January 1989.

The runoff data for the above period at the two weir stations 6-1-1-80 and 6-1-1-85 on the two main tributaries of the Pedieos River were obtained from the records of the WDD.

The rest of the parameters for the model were adapted as these were used for the calibrated model developed by WDD under the flood modelling implemented in meeting the European Flood Directive (2007/60/EC) by the WDD. It should be noted that this hydrologic modelling was calibrated then for the same storm as in the present case.



# 3.2.2.2.3. Sample Rainfall – Runoff graphs for each of the

# WRF configuration of the ensemble set

A sample of the rainfall-runoff graphs for each of the WRF as produced by the HEC-HMS model for the largest sub-basin (W910, see Figure 8) of Pedieos watershed is shown here-below. The graph for the same subbasin produced by the WDD hydrologic model using the actual recorded rainfall, is shown first (Figure 12), so that a comparison can be made between the recorded rainfall event and the one depicted by the ensemble set (Figure 13 to Figure 18).



Figure 12 Hyetograph of the rain event (loss in red and effective in blue) and the simulated runoff by the WDD model





Figure 13 Hyetograph of configuration "p16" and the simulated runoff for sub-basin W910







Figure 14 Hyetograph of configuration "p17" and the simulated runoff for sub-basin W910

n

2 4 6

10

Flow (cms)

0

12:00

08Jan1989

-- Run:Bingo\_event Element:VV910 Result:Baseflow

Legend (Compute Time: 03∆ɛx2016, 09:50:11) Run:Bingo\_event Element:VV910 Result:Precipitation

00:00







Run:Bingo\_event Element:W910 Result:Precipitation Loss

00:00

12:00

10Jan1989

00:00

12:00

11Jan1989

Run:Bingo\_event Element:W910 Result:Outflow

12:00

09Jan1989







Figure 16 Hyetograph of configuration "p19" and the simulated runoff for sub-basin W910





Figure 17 Hyetograph of configuration "p20" and the simulated runoff for sub-basin W910







Figure 18 Hyetograph of the mean of the ensemble set and the simulated runoff for sub-basin W910



# 3.2.2.3. Calibration and validation strategies

As indicated earlier, the model that was calibrated by the WDD was employed for the BINGO research on Pedieos for consistency purposes with the decision authorities and local stakeholders. As such, the present effort adapted in full the model as it was calibrated by the WDD and assumed the same parameters that were used for running the model. Some pertinent points on the parameters, the calibration etc. are as shown above.

The final calibration run for the WDD model which appears to have reached a good matching level comparing the simulated runoff to the observed at the weirs 6-1-1-80 and 6-1-1-85 is shown on Figure 19 and Figure 20 respectively.



Figure 19 Final calibration of simulated against observed runoff at weir 6-1-1-80 for the storm event of 8 to 11 January 1989 of the hydrologic model developed by WDD under the EU Flood Directive 2007/60/EC

As it is mentioned in D3.3 the model that was calibrated by the Water Development Department (WDD) for the Flood Directive implementation was employed for the BINGO research on Pedieos for consistency purposes with the competent authority (WDD) and local stakeholders. The choice of this calibrated model was made, so as to have compatible results with the results of the competent authority when the Flood Directive study was implemented.

The methodology used for the rainfall to runoff transformation is the SCS unit hydrograph methodology, which is applied to single flood event simulation and not continuous simulation. The objective of the WDD calibration procedure was to optimize the first main flood event which this was successfully achieved regarding the hydrograph pattern the peak flow and the flood volume. As is shown in the rainfall distribution graph below there is one main event followed by two smaller events. It is obvious that the transformed methodology creates one hydrograph for each event, fact that does not happen in reality, as the two other smaller events were not able to create their own hydrograph. Therefore, the objective of the WDD calibration procedure was to optimize the first main flood event which this was successfully achieved regarding the hydrograph pattern the peak flow and the flood volume







Figure 19b: Rainfall distribution graph for the storm event of 8 to 11 January 1989



Figure 20 Final calibration of simulated against observed runoff at weir 6-1-1-85 for the storm event of 8 to 11 January 1989 of the hydrologic model developed by WDD under the EU Flood Directive 2007/60/EC

# 3.2.2.4. Results

# 3.2.2.4.1. Results as time series compared to measured data, e.g. runoff

In Figure 21 to Figure 23 the results of the BINGO Pedieos hydrologic model showing the simulated runoff at each of the two weirs as a result of each precipitation configuration (WRF "p16" to "p20" and for the mean values for the whole set) within the ensemble set are shown in comparison to the observed runoff for the same weirs. These results refer to the storm of the 8-11 January 1989.



The simulated runoff is shown in blue colour and the observed runoff in black.



Figure 21 Simulated runoff as a result of the WRF configuration "p16" against the observed runoff at the weir 6-1-1-85



Figure 22 Simulated runoff as a result of the WRF configuration "p16" against the observed runoff at the weir 6-1-1-80





Figure 23 Simulated runoff as a result of the WRF configuration "p17" against the observed runoff at the weir 6-1-1-85



Figure 24 Simulated runoff as a result of the WRF configuration "p17" against the observed runoff at the weir 6-1-1-80





Figure 25 Figure 25 Simulated runoff as a result of the WRF configuration "p18" against the observed runoff at the weir 6-1-1-85



Figure 26 Simulated runoff as a result of the WRF configuration "p18" against the observed runoff at the weir 6-1-1-80





Figure 27 Simulated runoff as a result of the WRF configuration "p19" against the observed runoff at the weir 6-1-1-85



Figure 28 Simulated runoff as a result of the WRF configuration "p19" against the observed runoff at the weir 6-1-1-80





Figure 29 Simulated runoff as a result of the WRF configuration "p20" against the observed runoff at the weir 6-1-1-85



Figure 30 Simulated runoff as a result of the WRF configuration "p20" against the observed runoff at the weir 6-1-1-80





Figure 31 Simulated runoff as a result of the WRF Ensemble set mean of the five configurations against the observed runoff at the weir 6-1-1-85



Figure 32 Simulated runoff as a result of the WRF Ensemble set mean of the five configurations against the observed runoff at the weir 6-1-1-80

# 3.2.2.4.2. Flooding as per the results for each of the Ensemble precipitation configurations

The spatial distribution of the flood in the riparian areas of the Pedieos River downstream the Tamassos dam for each of the WRF precipitation configurations will be carried out using the HEC-RAS and the HEC-GeoRas presented earlier.

This will be carried out at the next stage of the research activity when the past conditions will be compared to future climate change conditions.



# 3.2.3. Model evaluation and discussion

# 3.2.3.1. Evaluation of the goodness of fit of the model

The configurations "p16" and "p17" exhibit a definite lag (about 10 hours) in the arrival of the peak runoff as compared to the observed runoff. The volume and the peak flow appear to be sufficiently close.

The hydrograph pattern of the simulated runoff of configuration "p18" coincides with the observed runoff but it appears to be of smaller volume.

The configuration "p19" exhibits a peak runoff advanced by about 10 hours as compared to the observed runoff at both weirs. Volume wise it appears sufficiently reasonable.

The configuration "p20" shows similarly as the "p19" an advancement of the simulated runoff but also a sizable reduction of volume and peak flow.

The mean of the ensemble set shows reasonable coincidence of the simulated runoff hydrograph to that of the observed runoff but both the peak and volume are much less.

Table 4 shows a summary of the results on volume, lag time of peaks and other statistical evaluations for all the configurations both for the simulated and the observed flows at each weir.

# 3.2.3.2. Discussion on the model results as presented above

The variation in the simulated results to the observed runoff for the various precipitation configurations are the result of the downscaling, mainly on the rainfall time distribution as compared to the real occurrence of the storm. These results should enable an improvement on the choice and manipulation of the rainfall as obtained from the WRF.

A potential reason for some shortcomings could be the accuracy of the weirs especially on low flows where diversions may be carried out.

The model was applied on an extreme event which is rather well represented which is important for decadal predictions.

As per the NSE results shown in Table 5 below, the runs for the WRF ensemble configurations number "p18" and for the mean of all configurations appear to give acceptable matching of the simulated flows to the observed ones. These results could be improved if further adjustment is made on the precipitation configurations, mainly on the time distribution, so that the distribution of flow in time is more in line with the observed flows or if the model parameters are further adjusted, something though that would come in contrast with the available final flooding model for the Pedieos as produced by the WDD.

The hydrologic modelling results obtained should enable further refinement in the production of the precipitation ensemble set configurations.



#### Table 5 Summary of results for each WRF configuration

|   |   |                     | WRF configuration   |                     |                     |                     |                     |                     |                     |                     |                     |                     |                     |  |
|---|---|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|---------------------|--|
|   |   | p16                 |                     | p17                 |                     | p'                  | p18                 |                     | p19                 |                     | P20                 |                     | Ensemble Mean       |  |
|   |   | Weir                |  |
|   |   | 6-1-1-85            | 6-1-1-80            | 6-1-1-85            | 6-1-1-80            | 6-1-1-85            | 6-1-1-80            | 6-1-1-85            | 6-1-1-80            | 6-1-1-85            | 6-1-1-80            | 6-1-1-85            | 6-1-1-80            |  |
| NS                                      | SE*                                       | -0.856              | -3.103              | -0.286              | -1.187              | 0.279               | 0.011               | -0.543              | -1.036              | -0.465              | -0.724              | 0.252               | 0.152               |  |
| Mean abs error**<br>(m <sup>3</sup> /s) |   | 14.1                | 5.2                 | 10.9                | 4.5                 | 8.6                 | 3.6                 | 12.8                | 4.8                 | 11                  | 4.0                 | 7.5                 | 2.9                 |  |
| RMS Erro                                | or***(m³/s)                               | 19.5                | 9.6                 | 16.2                | 7.0                 | 12.1                | 4.8                 | 17.8                | 6.8                 | 17.3                | 6.2                 | 12.4                | 4.4                 |  |
| Volume<br>(×1000                        | Residual<br>)m <sup>3</sup> )****         | 75.2                | -34.6               | 411.6               | -130.7              | -1746.5             | -910.3              | -1664.9             | -659.3              | -2798.5             | -1257.4             | -1255.8             | -645.2              |  |
|   | Peak<br>Discharg<br>e (m <sup>3</sup> /s) | 81.7                | 49                  | 56.6                | 25.9                | 24.9                | 14.3                | 33.4                | 18.8                | 8.7                 | 3.7                 | 19.7                | 9.3                 |  |
| Simulate<br>d Results                   | Volume<br>(×1000m <sup>3</sup><br>)       | 3278.9              | 1389.5              | 3619.7              | 1293.3              | 1456.8              | 513.8               | 1619.0              | 765.8               | 490.7               | 167.5               | 1992.2              | 779.4               |  |
|   | Date/Tim<br>e of peak<br>discharge        | 09Jan198<br>9 08:00 | 09Jan198<br>9 08:00 | 09Jan198<br>9 07:00 | 09Jan198<br>9 06:00 | 09Jan198<br>9 17:00 | 09Jan198<br>9 16:00 | 10Jan198<br>9 07:00 | 10Jan198<br>9 06:00 | 10Jan198<br>9 13:00 | 10Jan198<br>9 06:00 | 09Jan198<br>9 08:00 | 09Jan198<br>9 07:00 |  |
|   | Peak<br>Discharg<br>e (m <sup>3</sup> /s) | 69.6                | 18.3                | 69.6                | 18.3                | 69.6                | 18.3                | 69.6                | 18.3                | 69.6                | 18.3                | 69.6                | 18.3                |  |
| Observe<br>d Results                    | Volume<br>(×1000m <sup>3</sup><br>)       | 3202.2              | 1424.1              | 3202.2              | 1424.1              | 3202.2              | 1424.1              | 3202.2              | 1424.1              | 3202.2              | 1424.1              | 3202.2              | 1424.1              |  |
|   | Date/Tim<br>e of peak<br>discharge        | 09Jan198<br>9 18:00 |  |

\* Nash-Sutcliffe efficiency (NSE): The Nash-Sutcliffe efficiency (NSE) is a normalized statistic that determines the relative magnitude of the residual variance ("noise") compared to the measured data variance ("information") (Nash and Sutcliffe, 1970). NSE indicates how well the plot of observed versus simulated data fits the 1:1 line. NSE is computed as shown in equation 1:

where  $Y_i^{obs} - Y_i^{sim}$  where  $Y_i^{obs}$  is the  $i^{th}$  observation for the constituent being evaluated,  $Y_i^{sim}$  is the  $i^{th}$  simulated value for the constituent being evaluated,  $Y^{mean}$  is the mean of observed data for the constituent being evaluated, and n is the total number of observations.

 $\frac{1}{1000} \text{NSE ranges between } -\infty \text{ and } 1.0 \text{ (1 inclusive), with NSE = 1 being the optimal value. Values between 0.0 and 1.0 are generally viewed as acceptable levels of performance, whereas values <0.0 indicates that the mean observed value is a better predictor than the simulated value, which indicates unacceptable performance.}$ 

Sevat and Dezetter (1991) found NSE to be the best objective function for reflecting the overall fit of a hydrograph.

\*\*Mean absolute error is a quantity used to measure how close forecasts or predictions are to the eventual outcomes. The mean absolute error is given by

 $MAE = \frac{1}{n} \sum_{i=1}^{n} |f_i - y_i| = \frac{1}{n} \sum_{i=1}^{n} |e_i|.$  As the name suggests, the mean absolute error is an average of the absolute errors, where is the prediction and the true value. Note that alternative formulations may include relative frequencies as weight factors.

\*\*\* Root mean square error is the square root of the mean/average of the square of all of the error. The use of RMSE is very common and it makes an excellent general purpose error metric for numerical predictions.

$$RMSE = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (y_i - \hat{y}_i)^2}$$

NSE =1-

Compared to the similar Mean Absolute Error, RMSE amplifies and severely punishes large errors.

\*\*\*\*Volume residual: the difference between the simulated volume of flow and the observed volume of runoff (based on exactly the same period for which data are available) as applied to the observed data. Negative values indicate less volume of simulated runoff



# 3.2.4. Bibliography

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# 4. Germany

# 4.1. Model objectives in BINGO

The two main goals in BINGO are: a) improving existing hydrological models for the Wupper River Basin by including more detailed field data input, and b) enhancing the models by using ensemble predictions and circulation models. Under BINGO framework, Wupperverband is focusing on two pilot areas: the Dhünn catchment area (ca. 200 km<sup>2</sup>) and the Mirke catchment area (ca. 8 km<sup>2</sup>). The Dhünn River is the main tributary of the Wupper River. In 2010, Wupperverband commissioned a subcontractor (Hydrotec) to support the development of a detailed hydrological model for the Dhünn catchment area. Nine sub-basins of the Dhünn River Basin were modelled individually due to the availability of previously built up models and the need of performing separated calculations. The models are able to reproduce the whole water balance for both flood and water scarcity scenarios. NASIM software is used for water balance modelling due to its high level of detail regarding structures (capabilities, accuracy, flexibility, and deep level of detail), which makes it adequate to represent complex urban areas. The deep level of detail must go together with data availability, which is fortunately the case.

In 2016, Wupperverband commissioned another subcontractor (Sydro Consult) to support the update of an existing TALSIM model for the Dhünn catchment. The strength of TALSIM is the possibility of simulating reservoirs operation according to different rules or scenarios. The resolution of the TALSIM model is lower in comparison to NASIM since the former runs operationally (i.e., online) for Wupperverband's forecast and warning system (FEWS = flood early warning system). The Mirke model has been operated, updated, and improved permanently since 2008 for flood risk management by using radar data and defining thresholds for runoff generation. The Mirke basin is of particular importance since initiatives for flood risk management have been implemented in line with the EU water framework directive. These models and the two pilot areas are considered representative for modelling purposes as well as for historical and upcoming challenges regarding water management (flood protection and water supply).

# 4.2. Model application

# 4.2.1. Model description

NASIM and TALSIM are sub-catchment based (lumped), physically-based, water balance models based on the water balance equation. They estimate effective precipitation considering vertical processes, transforming it into surface runoff as translation or horizontal process (see Figure 33).

The vertical processes simulated by NASIM/TALSIM are:

- 1. Interception
- 2. Infiltration
- 3. Exfiltration / percolation
- 4. Evapotranspiration (ET)
- 5. Capillary rise (in floodplains)
- 6. Snow: estimation by temperatures under 0°C (Snow-Compaction-Procedure)



Atmosphere - Vegetation - Soil Interface: Overview



Figure 33 Vertical and horizontal hydrological processes (source: TALSIM)

The horizontal processes simulated by NASIM are

- 1. Surface runoff
- 2. Interflow
- 3. Deep interflow
- 4. Baseflow

Runoff is calculated according to the following equation:

$$EP = P(t) - PET(t) - I(t) - \frac{dO}{dt} - \frac{dS}{dt} \qquad \text{eq. (1)}$$

Where:

- EP: Effective precipitation
- P: Precipitation
- PET: Potential evapotranspiration

I: Infiltration

- O: Surface water storage
- S: Snow storage

The main processes that describe the soil water storage are: infiltration, exfiltration, and evapotranspiration (ET). Each of them depends on soil moisture (non-linear function).

For sealed areas, where no infiltration and subsurface storage occur, the terms  $\frac{ds}{dt}$  and I(t) are not considered (see eq. 2):

$$EP = P(t) - PET(t) - \frac{dO}{dt} \qquad \text{eq. (2)}$$

For unsealed areas, effective precipitation is estimated either by: i) constant runoff coefficient; ii) SCS-CN procedure (depends on land cover and soil type); or iii) soil moisture simulation. The soil moisture simulation method is applied in the developed models of the Wupperverband.

The soil moisture simulation parameters are:

• Land cover input parameters: root depth, sealing degree, leaf area index



• Soil type input parameters: wilting point, field capacity, total void volume, saturated hydraulic conductivity, maximum infiltration capacity, maximum capillary rise rate, and classification per soil class (e.g., sand, silt, clay)

The processes that influence the soil moisture simulation (see eq. 3) are:

- Infiltration (Inf)
- Percolation (Perc)
- Actual ET (AET)
- Interflow (Int)
- Capillary rise (Cap)

$$\frac{d\Theta(t)}{dt} = Inf(t) - Perc(t) - AET(t) - Int(t) + Cap(t) \quad \text{eq. (3)}$$

Where  $\theta(t)$  is soil moisture as a time dependent function.

Potential evapotranspiration (PET) must be calculated *prior to* entering it into the model with Haude or Penman procedures, depending on data availability. In this case, PET was calculated using the Haude method. Table 6 shows the input and output time series.

## Table 6 Input and output time series

| Input time series                            | Output time series                         |  |  |
|--|--|--|--|
| Precipitation                                | Runoff                                     |  |  |
| Air temperature (for snow layer calculation) | Interflow                                  |  |  |
| Potential evapotranspiration                 | Baseflow                                   |  |  |
|  | Evaporation (from the reservoir's surface) |  |  |
|  | Actual evapotranspiration                  |  |  |
|  | Effective precipitation                    |  |  |
|  | Soil moisture                              |  |  |
|  | Snow depth                                 |  |  |

# 4.2.2. Model set-up

Figure 34 shows the catchment area of the Dhünn River, with an area of ca. 203 km<sup>2</sup>, and Figure 35 presents the Dhünn River Basin divided into nine sub-basins (NASIM model). Figure 36 to Figure 44 show the nine hydrological models with their respective sub-basins. Figure 45 and Figure 46 show the sub-basins of the Dhünn River Basin (TALSIM model) and the TALSIM model structure, respectively. Figure 47 shows the main rivers and streams of the Dhünn catchment area as well as the hydrometric stations used for the model.

Figure 48 shows the catchment area of the Mirke Creek, with a total area of ca. 8 km<sup>2</sup> with the delineated sub-basins (NASIM model).





Figure 34 Dhünn River Basin and hydro-meteorological stations used for the Dhünn model



Figure 35 Nine modelled sub-basins of the Dhünn River Basin within the NASIM Model





Figure 36 Upper Große Dhünn



Figure 37 Dhünn reservoir



Figure 38 Eifgenbach



Figure 39 Scherbach



Figure 41 Mutzbach



Figure 43 Ophovener Mühlenbach





Figure 42 Linnefe





Figure 44 Lower Große Dhünn



Figure 45 Sub-basins of Dhünn River Basin – TALSIM model (GDT catchment area)





Figure 46 TALSIM model structure - Dhünn River Basin (GDT catchment area)



Figure 47 Rivers and streams and hydrometric stations of the Dhünn catchment area (source: Hydrotec, 2011)





Figure 48 Mirke catchment area (NASIM model)

# 4.2.3. Hydro-meteorological data

Figure 34 shows the location of the hydro-meteorological stations used for the models. All stations are operated by the Wupperverband except Manfort hydrometric station (operated by LANUV<sup>9</sup>). The format used for input time series into the model is *uvf* (Universal Variable Format).

Table 7 presents a summary of the **precipitation stations**. Temporal resolution is not equidistant: the meteorological stations register only changes in precipitation.

<sup>&</sup>lt;sup>9</sup> Landesamt für Natur, Umwelt und Verbraucherschutz Nordrhein-Westfalen



Table 8 presents a summary of the **hydrometric stations** used for model calibration with the respective period of observed discharge values. Temporal resolution is not equidistant: the hydrometric stations register only changes in water level. Table 9 shows typical flow rates of each hydrometric station.

| Station name            | Station code | From       | То         | Temporal resolution |
|-------------------------|--------------|------------|------------|---------------------|
| Bever-Talsperre         | SBEV         | 06.12.1967 | 31.10.2015 | Minutely            |
| Burscheid               | SBUR         | 01.01.2002 | 31.10.2015 | Minutely            |
| Klärwerk Leverkusen     | SLEV         | 01.11.1973 | 31.10.2015 | Minutely            |
| Klärwerk Dabringhausen  | SDAB         | 01.11.2002 | 31.10.2015 | Minutely            |
| Klärwerk Odenthal       | SODT         | 01.11.1973 | 31.10.2015 | Minutely            |
| Klärwerk Wermelskirchen | SWER         | 01.11.1991 | 31.10.2015 | Minutely            |
| Lindscheid              | SLIN         | 01.11.1973 | 31.10.2015 | Minutely            |
| Neumühle                | SNEM         | 01.11.1973 | 31.10.2015 | Minutely            |
| Unterpilghausen         | SPIL         | 01.11.2001 | 31.10.2015 | Minutely            |
| Vorsperre Große Dhünn   | SVO\$        | 02.11.1987 | 31.10.2015 | Minutely            |
| *Westhofen              | SWES         | 01.11.1973 | 02.05.2010 | Minutely            |

## Table 7 Precipitation stations used for the Dhünn model

\*Westhofen precipitation station has been out of operation since 2010. Time series of Westhofen precipitation station were generated synthetically from 02.05.2010 to 31.10.2015 with Neumühle station (closest neighbour precipitation station) by using an incremental correlation factor of 1,03 (MAP<sup>10</sup> of Westhofen is higher).

| Station name    | Station code | From       | То         | Temporal resolution |
|-----------------|--------------|------------|------------|---------------------|
| Dünnwald        | SDUE         | 01.10.1994 | 29.08.2000 | Minutely            |
| Finkenholl      | SFIN         | 01.11.2002 | 01.03.2011 | Minutely            |
| Hummelsheim     | SHUM         | 01.11.1988 | 01.11.2015 | Minutely            |
| Linnefe         | SLIF         | 03.11.2003 | 08.11.2004 | Minutely            |
| Loosenau        | SLOS         | 01.11.1988 | 01.11.2015 | Minutely            |
| Manfort         | SMAN         | 01.11.1988 | 19.01.2015 | Minutely            |
| Markusmühle     | SMAR         | 20.10.2002 | 01.11.2015 | Minutely            |
| Neumühle        | SNEM         | 11.12.1964 | 01.11.2015 | Minutely            |
| Paffrath        | SPAF         | 01.08.1995 | 04.11.2002 | Minutely            |
| Scherbach       | SSHE         | 21.09.2000 | 19.09.2011 | Minutely            |
| Schlebusch      | SSLB         | 01.11.2006 | 01.11.2015 | Minutely            |
| Unterpilghausen | SPIL         | 01.11.1988 | 01.11.2015 | Minutely            |

#### Table 8 Hydrometric stations used for the Dhünn model

<sup>&</sup>lt;sup>10</sup> Mean Annual Precipitation



| Station name    | Station<br>code | MQ <sup>11</sup><br>[m³/s] | MNQ <sup>12</sup><br>[m³/s] | MHQ <sup>13</sup><br>[m³/s] | River / stream |
|-----------------|-----------------|----------------------------|-----------------------------|-----------------------------|----------------|
| Dünnwald        | SDUE            | 0,073                      | 0,032                       | 0,408                       | Mutzbach       |
| Finkenholl      | SFIN            | 0,265                      | 0,064                       | 3,644                       | Eifgenbach     |
| Hummelsheim     | SHUM            | 1,679                      | 0,551                       | 21,000                      | Dhünn          |
| Linnefe         | SLIF            | n/a                        | n/a                         | n/a                         | Linnefe        |
| Loosenau        | SLOS            | 0,682                      | 0,078                       | 8,410                       | Dhünn          |
| Manfort         | SMAN            | 2,253                      | 0,426                       | 13,47                       | Dhünn          |
| Markusmühle     | SMAR            | 0,497                      | 0,106                       | 8,337                       | Eifgenbach     |
| Neumühle        | SNEM            | 0,513                      | 0,037                       | 8,960                       | Große Dhünn    |
| Paffrath        | SPAF            | 0,027                      | 0,050                       | 0,629                       | Katterbach     |
| Scherfbach      | SSHE            | 0,500                      | 0,181                       | 3,736                       | Scherfbach     |
| Schlebusch      | SSLB            | 2,017                      | 0,517                       | 20,800                      | Dhünn          |
| Unterpilghausen | SPIL            | 0,280                      | 0,023                       | 4,770                       | Kleine Dhünn   |

## Table 9 Typical flow rates of the hydrometric station

Potential evapotranspiration (PET) was estimated with Haude. The necessary data are temperature at 14:00 and air humidity. Table 10 presents a summary of the **meteorological stations** used for each model. Past data (i.e., before 2000) from Buchenhofen climatological station comes from the German Weather Service (DWD<sup>14</sup>).

Table 10 Climatological time series used for the Dhünn model

| Parameter       | Station name | Station code | From       | То         | Temporal resolution |
|-----------------|--------------|--------------|------------|------------|---------------------|
| Air temperature | Buchenhofen  | SBUC         | 01.11.1951 | 01.11.2015 | Daily               |
| Air temperature | Leverkusen   | SLEV         | 01.01.1970 | 31.10.2015 | Daily               |
| PET (Haude)     | Buchenhofen  | SBUC         | 01.01.1959 | 01.11.2015 | Daily               |
| PET (Haude)     | Leverkusen   | SLEV         | 01.01.1970 | 01.11.2015 | Daily               |

# 4.2.4. Calibration and validation strategies

Calibration of the nine models was performed manually, event-based by comparing observed and simulated discharge, with a time step for all simulations of 15 min. The precipitation time series for each sub-catchment of each model was obtained using the Thiessen polygon method. The same precipitation assignment procedures were applied for both calibration and validation processes. Calibrated parameters are presented in Table 11.

<sup>&</sup>lt;sup>11</sup> Mean runoff (Mittlerer Abfluss)

<sup>&</sup>lt;sup>12</sup> Mean low runoff (Mittlerer Niedrigwasserabfluss)

<sup>&</sup>lt;sup>13</sup> Mean maximum runoff (Mittlerer Hochwasserabfluss)

<sup>&</sup>lt;sup>14</sup> Deutscher Wetterdienst



#### Table 11 Calibrated parameters for the nine models of the Dhünn River Basin

| Parameter                              | Unit | Rar                 | nge                          |  |
|--|------|---------------------|------------------------------|--|
|  |      | From                | То                           |  |
| Max. interception                      | mm   | 2 - for urban areas | 8 - for coniferous<br>forest |  |
| Retention constant<br>(surface runoff) | h    | 7                   | 10                           |  |
| Retention constant<br>(interflow)      | h    | 25                  | 30                           |  |
| Retention constant<br>(baseflow)       | h    | 700                 | 1000                         |  |
| Max. infiltration                      | mm/h | 100                 | 600                          |  |
| Vertical hydraulic<br>conductivity     | mm/h | 0,08                | 0,13                         |  |

Table 12 shows the calibration points and Table 13 presents the calibration period of each model as well as the areas of each sub-basin. The hydrological year in Germany starts on 1<sup>st</sup> November. Sub-basin delineation was carried out according to the drainage area of each hydrometric station (i.e., each calibration point).

#### Table 12 Calibration points of the nine models

| Model / Sub-basin    | Hydrometric station used for calibration / validation           |
|----------------------|---|
| Upper Große Dhünn    | Neumühle (SNEM)   |
| Dhünn reservoir      | Unterpilghausen (SPIL), Loosenau (SLOS), and GDT storage volume |
| Eifgenbach           | Finkenholl (SFIN) and Markusmühle (SMAR)                        |
| Linnefe              | Linnefe (SLIF)  |
| Scherbach            | Scherfbach (SSHE)   |
| Mutzbach             | Dünnwald (SDUE)   |
| Katterbach           | Paffrath (SPAF)   |
| Ophovener Mühlenbach | n/a   |
| Lower Große Dhünn    | Hummelsheim (SHUM), Manfort (SMAN), and Schlebusch (SSLB)       |

Since some hydrometric stations are no longer in operation (see Table 8), the models Linnefe, Scherfbach, Mutzbach, Eifgenbach, and Katterbach could not be validated for recent years (i.e., 2009 to 2015). The Linnefe model could not be validated for any time period since the whole available time series of Linnefe hydrometric station was used entirely for calibration. The Ophovener Mühlenbach model could not be neither calibrated nor validated due to lack of data (there are no hydrometric stations in this sub-basin).



| Model / Sub basin    | Area    | Calibration | Calibration | Validation | Validation |
|----------------------|---------|-------------|-------------|------------|------------|
| Model / Sub-basin    | [km²]   | begin       | end         | begin      | end        |
| Upper Große Dhünn    | <u></u> | 06.11.2001  | 01.11.2009  | 01.11.1973 | 06.11.2001 |
|                      | 23,3    |             |             | 01.11.2009 | 01.11.2015 |
| Dhüpp reservoir      | 38.25   | 25.05.2006  | 05.04.2009  | 06.11.2002 | 15.03.2006 |
| Dhuhin reservoir     | 30,23   | 25.05.2000  | 05.04.2009  | 05.04.2009 | 01.11.2015 |
| Eifgenbach           | 22.01   | 08.03.2006  | 05.04.2009  | 06.11.2002 | 15.03.2006 |
|                      | 32,91   |             |             | 01.11.2009 | 01.11.2015 |
| Linnefe              | 17,59   | 12.12.2003  | 30.09.2004  | n/a        | n/a        |
| Scherfbach           | 5,25    | 10.12.2006  | 19.03.2009  | 20.03.2001 | 07.06.2006 |
| Mutzbach             | 19,89   | 01.11.1992  | 31.10.1993  | 28.06.1996 | 10.10.1999 |
| Katterbach           | 5,46    | 01.11.1997  | 31.10.1999  | 28.08.1996 | 26.07.2000 |
| Ophovener Mühlenbach | 5,04    | n/a         | n/a         | n/a        | n/a        |
| Lower Große Dhünn    | 66.07   | 29.03.2006  | 08 04 2009  | 18.03.2001 | 02.03.2005 |
|                      | 00,07   | 23.03.2000  | 00.04.2009  | 08.04.2009 | 19.01.2015 |

# Table 13 Calibration and validation periods



# 4.2.5. Results

# 4.2.5.1. Upper Große Dhünn

The simulation results for the Upper Dhünn from 1973 to 2015 are presented in Figure 49. Figure 50 shows some simulated events.



Figure 49 Upper Große Dhünn - observed and simulated discharge at Neumühle hydrometric station (blue: observed; red: simulated




Figure 50 Upper Große Dhünn - event simulations (blue: observed; red: simulated)

## 4.2.5.2. Dhünn reservoir

The Dhünn reservoir model presents a good agreement at the stations Unterpilghausen (Figure 51) and Loosenau (see Figure 52). Figure 53 presents observed and simulated discharge at the GDT, including the guiding function for storage volume (OL)<sup>15</sup>, for the calibration and validation periods. The simulation considers the operational rules of the GDT, which are summarized as follows:

- Drinking water supply: 1205 l/ s as constant release
- Minimum discharge downstream: 1 m<sup>3</sup>/s at Manfort hydrometric station
- Reservoir filling: retention if volume is smaller than the guiding function for storage volume
- Flood protection: retention if discharge at Manfort hydrometric station is greater than 30 m<sup>3</sup>/s

<sup>&</sup>lt;sup>15</sup> In German: Orientierungslinie





Figure 51 Dhünn reservoir – observed and simulated discharge at Unterpilghausen hydrometric station (blue: observed; red: simulated)



Figure 52 Dhünn reservoir – observed and simulated discharge at Loosenau hydrometric station (blue: observed; red: simulated)





Figure 53 Observed and simulated volume at the GDT, including the guiding function for storage volume (OL) (blue: observed; red: simulated)

## 4.2.5.3. Eifgenbach

Figure 54 and Figure 55 show observed and simulated discharge of single events at Markusmühle and Finkenholl hydrometric stations. The Eifgenbach model shows a good agreement between observed and simulated discharge (see Figure 56 and Figure 57). There is a better model fit at the Markusmühle station (most downstream station) than at Finkenholl station.



Figure 54 Eifgenbach - event simulation Markusmühle (blue: observed; red: simulated)





Figure 55 Eifgenbach – event simulation Finkenholl (blue: observed; red: simulated)



Figure 56 Eifgenbach - observed and simulated discharge at Finkenholl hydrometric station (blue: observed; red: simulated)





Figure 57 Eifgenbach - observed and simulated discharge at Markusmühle hydrometric station (blue: observed; red: simulated)

## 4.2.5.4. Linnefe

The Linnefe model shows a good agreement between observed and simulated discharge (see Figure 58). As mentioned earlier, this model could not be validated for any time period since the whole available time series of Linnefe hydrometric station was used entirely for calibration. This station is no longer in operation.





Figure 58 Linnefe - observed and simulated discharge at Linnefe hydrometric station (blue: observed; red: simulated)

### 4.2.5.5. Scherfbach

This model presented water balance implausibility: observed discharge is higher than the accumulated precipitation for the hydrological years 2003 to 2009 (see Figure 59). The rainfall-runoff relation seems to be plausible only for the hydrological years 2001 and 2002. In the case of the Scherfbach model, simulated discharge seems to be more plausible than observed discharge. Nevertheless, observed discharge was used to calibrate the wave rise and its course as well as for model validation. The yearly water balance implausibility could be due to an inappropriate rating curve, so observed discharge at this station is not reliable. Figure 60 and Figure 61 show observed and simulated discharge of some events at the Scherfbach station, where the shape of observed and simulated hydrographs seems to coincide.



Figure 59 Scherfbach - observed and simulated discharge at Scherfbach hydrometric station (blue: observed; red: simulated; grey: accumulated precipitation) (source: Hydrotec, 2011)





Figure 60 Scherfbach - observed and simulated discharge at Scherfbach hydrometric station (blue: observed; red: simulated)



Figure 61 Scherfbach - observed and simulated discharge at Scherfbach hydrometric station (blue: observed; red: simulated)

#### 4.2.5.6. Mutzbach

The Mutzbach model had been previously calibrated (see Table 13). Figure 62 shows observed and simulated discharge (hydrographs), and Figure 63 presents accumulated observed and simulated discharge (i.e., accumulated volume), at the Dünnwald hydrometric station. Cumulative discharge for the hydrological years 1997 to 1999 (i.e., validation period, see Figure 63) presents a better fit than for the calibration period.





Figure 62 Mutzbach - observed and simulated discharge at Dünnwald hydrometric station (blue: observed; red: simulated)



Figure 63 Mutzbach – accumulated observed and simulated discharge at Dünnwald hydrometric station (blue: observed; red: simulated)



## 4.2.5.7. Katterbach

Figure 64 shows observed and simulated discharge for the hydrological year 1999. This model only presents satisfactory results for some events within the validation period (see, e.g. Figure 65). For other events, there is a discrepancy between observed and simulated discharge, not only in terms of peak discharge magnitude but also in terms of baseflow (see Figure 66).



Figure 64 Katterbach – observed and simulated discharge at Paffrath hydrometric station (blue: observed; red: simulated)



Figure 65 Katterbach – good model fit of observed and simulated discharge at Paffrath hydrometric station (blue: observed; red: simulated)







Figure 66 Katterbach – bad model fit observed and simulated discharge at Paffrath hydrometric station (blue: observed; red: simulated)

## 4.2.5.8. Lower Große Dhünn

Boundary conditions of this model include inflow from the Dhünn reservoir model. Figure 67 shows observed and simulated discharge at Hummelsheim for a single event. Figure 68 presents observed and simulated discharge at Manfort hydrometric station, for the calibration period. The model presents also a good agreement at the Schlebusch hydrometric station (see Figure 69).



Figure 67 Lower Große Dhünn – observed and simulated discharge at Hummelsheim hydrometric station (blue: observed; red: simulated)





Figure 68 Lower Große Dhünn – observed and simulated discharge at Manfort hydrometric station (blue: observed; red: simulated)



Figure 69 Lower Große Dhünn – observed and simulated discharge at Schlebusch hydrometric station (blue: observed; red: simulated)



## 4.2.5.9. TALSIM model for the Dhünn catchment

The model set-up is finished, calibrated, and validated. Figure 70 shows an example of a winter event in 2006. The model is being currently transferred by SYDRO to Wupperverband. Further simulations will be presented in the next report.



Figure 70 Winter event of TALSIM model - Dhünn catchment (blue: observed; red: simulated)

### 4.2.5.10. NASIM model for the Mirke catchment

The Mirke model was already calibrated and validated. It has been operational since 2008 and constantly improved. Figure 71 presents a calibrated event at different gauges within the catchment.



Figure 71 Calibrated event at different gauges - Mirke model (black: observed; red: simulated)



### 4.3. Model evaluation and discussion

The goodness-of-fit (GOF) or model fit of observed and computed discharge was obtained by comparing hydrographs and accumulated discharge. For each model, performance was evaluated with Nash-Sutcliffe Efficiency, correlation coefficient<sup>16</sup> (or Pearson correlation coefficient, *r*), coefficient of determination ( $R^{a}$ ), and volume-error for the calibration and validation periods.

Equations 4 to 7 present Nash-Sutcliffe Efficiency, correlation coefficient (Pearson coefficient, r), coefficient of determination  $R^2$ , and Relative Volume Error (RV<sub>E</sub>), respectively:

$$E = 1 - \frac{\sum_{i=1}^{n} (O_i - P_i)^2}{\sum_{i=1}^{n} (O_i - \bar{O})^2}$$
 eq. (4)

$$r = \frac{\sum_{i=1}^{n} (O_i - \bar{O})(P_i - \bar{P})}{\sqrt{\sum_{i=1}^{n} (O_i - \bar{O})^2} \sqrt{\sum_{i=1}^{n} (P_i - \bar{P})^2}}$$
eq. (5)

$$r^{2} = \left(\frac{\sum_{i=1}^{n} (O_{i} - \bar{O})(P_{i} - \bar{P})}{\sqrt{\sum_{i=1}^{n} (O_{i} - \bar{O})^{2}} \sqrt{\sum_{i=1}^{n} (P_{i} - \bar{P})^{2}}}\right)^{2} \qquad \text{eq. (6)}$$

$$RV_E = \left(\frac{\sum_{i=1}^{n} (P_i - O_i)}{\sum_{i=1}^{n} O_i}\right) * 100\%$$
 eq. (7)

Where O are observed and P are predicted (simulated) values, and  $r^2$  equals  $R^2$ .

The range of values of Nash-Sutcliffe Efficiency is 1.0 (for a perfect fit) and  $-\infty$ , with values lower than zero indicating that the mean value of the observed time series would have been a better predictor than the model. The correlation coefficient ranges from 0.0 (no correlation between observed and simulated values) and 1.0 (perfect fit) (Krause et al. 2005). The same applies to the coefficient of determination  $R^2$ .

Relative Volume Error (RV<sub>E</sub>) can vary between  $-\infty$  and  $\infty$ , with 0.0% as perfect fit (Janssen and Heuberger 1995, cited in Gumindoga 2010). Values ranging between -10% and -5% or +5% and +10% indicate reasonable performance; on the other hand, values between -5% and +5% indicate well model performance (Gumindoga 2010).

Table 14 to Table 24 present the goodness-of-fit for the nine models of the Dhünn catchment area. It is considered that the nine hydrological models of the Dhünn catchment area present an overall good agreement between observed and simulated discharge. For instance, all models present a good agreement for the hydrological year 2009 (01.11.2008 – 01.11.2009). Extreme events are also considered to be well represented by the models, e.g., Figure 50 shows the registered flood event of November 2010.

Some hydrological years show good agreement between all statistical parameters used for GOF analysis (see, e.g., hydrological year 2003 of Table 14 and hydrological year 2004 of Table 16). Other hydrological years present a good  $RV_E$  while showing poorer values of Nash-Sutcliffe Efficiency and coefficient of determination  $R^2$  (see, e.g., hydrological year 2012 of Table 24 and hydrological year 1998 of Table 20). On

<sup>&</sup>lt;sup>16</sup> The Pearson correlation coefficient is the square of the coefficient of determination,  $R^2$ , that is,  $r^2 = R^2$ .



the other hand, hydrological year 2015 of Table 15 illustrates a poor value of  $RV_E$  while showing better model fit in terms of Nash-Sutcliffe Efficiency and correlation coefficient *r*.

There are also some cases where correlation coefficient *r* and RV<sub>E</sub> show a good and reasonable agreement, respectively, while Nash-Sutcliffe Efficiency is poor (see, e.g., hydrological year 2011 of Table 22). Hydrological year 2003 of Table 16 presents good correlation coefficient *r* but poor Nash-Sutcliffe Efficiency and RV<sub>E</sub> values. Other cases show the same values of *r* (namely,  $R^2$ ) while presenting very different values of RV<sub>E</sub> (see Table 24, hydrological year 2011: poor RV<sub>E</sub> value and hydrological year 2012: good RV<sub>E</sub> value).

Pearson coefficient (also known as linear correlation) expresses the linear dependency between observed and simulated discharge. All *r* calculated values are greater than zero, indicating positive linear correlation. Nash-Sutcliffe Efficiency expresses the difference between observed and simulated discharge as squared values, overestimating the difference which is the main method's disadvantages (Krause et al., 2005).  $RV_E$ is considered to be more meaningful for reservoir management while the other statistical parameters might be more appropriate for high peak discharge occurrence. Since statistical parameters used for GOF analysis have different drawbacks, it is advisable to estimate several in order to describe model performance more accurately.

NASIM is a conceptual model, highly sensitive to variations in rainfall (input variable). Therefore, current efforts to improve the determination of spatial variability of precipitation with radar data and satellite-based products are being implemented in the frame of BINGO in order to enhance flood forecasting quality.

In addition, determination of soil moisture and evapotranspiration (state variables) and their effects on water balance and runoff generation processes are being investigated and supported by the installation of soil moisture sensors and lysimeters. Until now, both soil moisture and actual evapotranspiration are estimated by the models. Figure 72 shows potential and actual evapotranspiration (PET and AET) for the hydrological year 2007, and Figure 73 and Figure 74 present soil moisture, effective precipitation, and observed and simulated discharge at element 8193\_1 of the Upper Große Dhünn model (sub-catchment of Neumühle hydrometric station<sup>17</sup>) from January to July 2007.

The model also shows high sensitivity to antecedent soil moisture conditions: after a short period with no rain, soil begins to dry out (see Figure 73). Then, a high precipitation event takes place in May 2007, causing less significant discharge in comparison to January and February 2007 (see Figure 74). In June 2007, a lower rainfall amount (in comparison to May 2007) leads to higher discharge on account of antecedent soil moisture (see Figure 74).

Infiltration processes and surface runoff generation depend on antecedent soil moisture; thus, it is necessary to acquire deeper knowledge in soil wetness development over time in order to better calibrate the models. Discrepancies between observed and simulated discharge presented by the models are considered to be caused, among others, by little information on soil moisture, which will be overcome on BINGO framework.

<sup>&</sup>lt;sup>17</sup> Outflow from element 8193\_1 corresponds to observed discharge at Neumühle hydrometric station.





Figure 72 PET and AET at element 8193\_1 (Upper Große Dhünn model) – hydrological year 2007



Figure 73 Soil moisture and effective precipitation at element 8193\_1 (Upper Große Dhünn model)







Figure 74 Soil moisture and observed and simulated discharge at element 8193\_1 (Upper Große Dhünn model)



# Table 14 Upper Große Dhünn - goodness-of-fit

|                 | Model 1: Upper Große Dhünn                      |                                   |  |  |  |            |  |  |  |  |
|-----------------|---|-----------------------------------|--|--|--|------------|--|--|--|--|
|                 | Calibration point: Neumühle hydrometric station |                                   |  |  |  |            |  |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency                | Correlation<br>coefficient<br>(r) | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |  |
| 1974            | 0,56  | 0,75                              | 0,57   | 370321,31                                | 299432,41                                | -19,14     |  |  |  |  |
| 1975            | 0,77  | 0,89                              | 0,79   | 231673,89                                | 290039,34                                | 25,19      |  |  |  |  |
| 1976            | 0,79  | 0,92                              | 0,84   | 232142,25                                | 241329,59                                | 3,96       |  |  |  |  |
| 1977            | 0,55  | 0,76                              | 0,58   | 276815,19                                | 306783,22                                | 10,83      |  |  |  |  |
| 1978            | 0,70  | 0,87                              | 0,75   | 231492,95                                | 327882,97                                | 41,64      |  |  |  |  |
| 1979            | 0,59  | 0,77                              | 0,59   | 283996,84                                | 269086,13                                | -5,25      |  |  |  |  |
| 1980            | 0,82  | 0,92                              | 0,86   | 372555,47                                | 418805,34                                | 12,41      |  |  |  |  |
| 1981            | 0,68  | 0,83                              | 0,70   | 457294,56                                | 429200,94                                | -6,14      |  |  |  |  |
| 1982            | undeterm.                                       | undeterm.                         | undeterm.                                      | 251250,63                                | 342598,44                                | 36,36      |  |  |  |  |
| 1983            | 0,72  | 0,86                              | 0,73   | 321964,59                                | 373046,22                                | 15,87      |  |  |  |  |
| 1984            | 0,17  | 0,92                              | 0,84   | 299884,50                                | 425797,31                                | 41,99      |  |  |  |  |
| 1985            | 0,19  | 0,78                              | 0,61   | 256485,66                                | 354160,28                                | 38,08      |  |  |  |  |
| 1986            | -3,66   | 0,62                              | 0,38   | 204358,61                                | 345804,38                                | 69,21      |  |  |  |  |
| 1987            | 0,71  | 0,90                              | 0,81   | 347832,81                                | 482065,91                                | 38,59      |  |  |  |  |
| 1988            | 0,40  | 0,86                              | 0,74   | 284107,13                                | 381158,50                                | 34,16      |  |  |  |  |
| 1989            | 0,66  | 0,82                              | 0,68   | 215048,22                                | 259180,11                                | 20,52      |  |  |  |  |
| 1990            | 0,77  | 0,90                              | 0,80   | 217019,64                                | 244768,58                                | 12,79      |  |  |  |  |
| 1991            | 0,75  | 0,89                              | 0,79   | 224756,63                                | 218521,14                                | -2,77      |  |  |  |  |
| 1992            | 0,87  | 0,93                              | 0,87   | 318799,03                                | 319074,41                                | 0,09       |  |  |  |  |
| 1993            | 0,81  | 0,93                              | 0,87   | 283218,78                                | 399831,72                                | 41,17      |  |  |  |  |
| 1994            | 0,81  | 0,90                              | 0,82   | 362715,47                                | 349633,28                                | -3,61      |  |  |  |  |
| 1995            | 0,82  | 0,92                              | 0,84   | 348783,13                                | 434984,06                                | 24,71      |  |  |  |  |
| 1996            | 0,57  | 0,88                              | 0,77   | 232136,81                                | 182907,00                                | -21,21     |  |  |  |  |
| 1997            | 0,50  | 0,75                              | 0,56   | 202225,19                                | 240683,11                                | 19,02      |  |  |  |  |
| 1998            | 0,86  | 0,95                              | 0,90   | 410588,53                                | 403386,25                                | -1,75      |  |  |  |  |
| 1999            | 0,85  | 0,93                              | 0,87   | 309281,63                                | 337601,31                                | 9,16       |  |  |  |  |
| 2000            | 0,71  | 0,89                              | 0,79   | 365664,38                                | 381134,53                                | 4,23       |  |  |  |  |
| 2001            | 0,82  | 0,92                              | 0,85   | 367083,63                                | 388792,38                                | 5,91       |  |  |  |  |
| 2002            | 0,62  | 0,82                              | 0,67   | 374432,72                                | 415068,09                                | 10,85      |  |  |  |  |
| 2003            | 0,83  | 0,92                              | 0,84   | 284632,03                                | 311327,16                                | 9,38       |  |  |  |  |
| 2004            | 0,86  | 0,93                              | 0,87   | 375029,53                                | 338747,50                                | -9,67      |  |  |  |  |
| 2005            | 0,91  | 0,95                              | 0,91   | 337770,75                                | 348856,13                                | 3,28       |  |  |  |  |
| 2006            | 0,82  | 0,91                              | 0,83   | 339624,44                                | 304236,19                                | -10,42     |  |  |  |  |
| 2007            | 0,75  | 0,90                              | 0,81   | 433498,97                                | 432339,41                                | -0,27      |  |  |  |  |
| 2008            | 0,78  | 0,91                              | 0,84   | 343596,66                                | 400620,63                                | 16,60      |  |  |  |  |
| 2009            | 0,82  | 0,91                              | 0,82   | 277961,53                                | 280301,78                                | 0,84       |  |  |  |  |
| 2010            | 0,36  | 0,65                              | 0,43   | 319552,31                                | 231217,38                                | -27,64     |  |  |  |  |
| 2011            | 0,04  | 0,76                              | 0,58   | 289135,16                                | 359669,53                                | 24,39      |  |  |  |  |
| 2012            | 0,57  | 0,87                              | 0,75   | 323024,44                                | 355693,28                                | 10,11      |  |  |  |  |
| 2013            | 0,19  | 0,79                              | 0,62   | 258598,42                                | 310690,81                                | 20,14      |  |  |  |  |
| 2014            | -0,02   | 0,74                              | 0,55   | 251656,67                                | 307312,69                                | 22,12      |  |  |  |  |
| 2015            | 0,67  | 0,85                              | 0,72   | 213339,86                                | 298487,88                                | 39,91      |  |  |  |  |



|                 | Model 2: Dhünn reservoir<br>Calibration point: Unterpilghausen hydrometric station |      |  |  |  |            |  |  |
|-----------------|--|------|--|--|--|------------|--|--|
|                 |  |      |  |  |  |            |  |  |
| Hydrol.<br>year | ol.<br>r Sutcliffe Correlation<br>Efficiency (r)                                   |      | Coefficient of<br>determination<br>(R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |
| 2003            | 0,83   | 0,91 | 0,83   | 117455,93                                | 108400,31                                | -7,71      |  |  |
| 2004            | 0,82   | 0,91 | 0,83   | 125802,64                                | 107210,49                                | -14,78     |  |  |
| 2005            | 0,85   | 0,95 | 0,89   | 129890,76                                | 118033,37                                | -9,13      |  |  |
| 2006            | 0,71   | 0,87 | 0,76   | 122300,03                                | 100723,11                                | -17,64     |  |  |
| 2007            | 0,73   | 0,86 | 0,73   | 167317,34                                | 149958,05                                | -10,38     |  |  |
| 2008            | 0,87   | 0,93 | 0,87   | 142763,03                                | 139146,86                                | -2,53      |  |  |
| 2009            | 0,81   | 0,91 | 0,83   | 110485,34                                | 94700,65                                 | -14,29     |  |  |
| 2010            | 0,63   | 0,84 | 0,71   | 124321,45                                | 82862,02                                 | -33,35     |  |  |
| 2011            | 0,31   | 0,66 | 0,43   | 167902,84                                | 127946,90                                | -23,80     |  |  |
| 2012            | 0,67   | 0,86 | 0,74   | 155357,36                                | 115466,27                                | -25,68     |  |  |
| 2013            | 0,50   | 0,79 | 0,63   | 137495,52                                | 108228,76                                | -21,29     |  |  |
| 2014            | 0,35   | 0,73 | 0,54   | 129238,16                                | 90334,31                                 | -30,10     |  |  |
| 2015            | 0,56   | 0,81 | 0,66   | 140643,55                                | 90790,23                                 | -35,45     |  |  |

#### Table 15 Dhünn reservoir - goodness-of-fit

#### Table 16 Eifgenbach - goodness-of-fit

|                 | Calibration point: Markusmühle hydrometric station           |           |  |  |  |            |  |  |
|-----------------|--|-----------|--|--|--|------------|--|--|
| Hydrol.<br>year | Nash- Correlation<br>Sutcliffe coefficient<br>Efficiency (r) |           | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |
| 2003            | -0,47  | 0,91      | 0,82   | 198221,55                                | 275023,75                                | 38,75      |  |  |
| 2004            | 0,75   | 0,88      | 0,78   | 283984,88                                | 261193,11                                | -8,03      |  |  |
| 2005            | 0,81   | 0,91      | 0,82   | 338489,09                                | 290986,44                                | -14,03     |  |  |
| 2006            | 0,73   | 0,86      | 0,74   | 234692,64                                | 249873,30                                | 6,47       |  |  |
| 2007            | 0,79   | 0,89      | 0,80   | 355761,13                                | 348148,41                                | -2,14      |  |  |
| 2008            | 0,85   | 0,94      | 0,88   | 318622,06                                | 320139,19                                | 0,48       |  |  |
| 2009            | 0,84   | 0,93      | 0,86   | 278145,03                                | 233632,67                                | -16,00     |  |  |
| 2010            | 0,39   | 0,72      | 0,52   | 302213,50                                | 199590,25                                | -33,96     |  |  |
| 2011            | 0,61   | 0,78      | 0,61   | 320055,69                                | 303102,56                                | -5,30      |  |  |
| 2012            | undeterm.  | undeterm. | undeterm.                                      | 230574,55                                | 263834,09                                | 14,42      |  |  |
| 2013            | 0,62   | 0,88      | 0,77   | 214823,03                                | 261875,61                                | 21,90      |  |  |
| 2014            | 0,38   | 0,66      | 0,44   | 249722,80                                | 217982,73                                | -12,71     |  |  |
| 2015            | 0,68   | 0,84      | 0,71   | 244289,05                                | 210083,20                                | -14,00     |  |  |



#### Table 17 Eifgenbach - goodness-of-fit

|                 |   | Model 3: Eifgenbach               |  |  |  |            |  |  |  |  |
|-----------------|---|-----------------------------------|--|--|--|------------|--|--|--|--|
|                 | Calibration point: Finkenholl hydrometric station |                                   |  |  |  |            |  |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency                  | Correlation<br>coefficient<br>(r) | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |  |
| 2003            | undeterm.   | undeterm.                         | undeterm.                                      | 136318,50                                | 136883,41                                | 0,41       |  |  |  |  |
| 2004            | 0,74  | 0,88                              | 0,78   | 146910,78                                | 136641,42                                | -6,99      |  |  |  |  |
| 2005            | 0,86  | 0,93                              | 0,86   | 150882,78                                | 139976,06                                | -7,23      |  |  |  |  |
| 2006            | 0,62  | 0,79                              | 0,63   | 126639,41                                | 125866,98                                | -0,61      |  |  |  |  |
| 2007            | 0,72  | 0,85                              | 0,73   | 192589,69                                | 177184,36                                | -8,00      |  |  |  |  |
| 2008            | 0,82  | 0,93                              | 0,87   | 166856,53                                | 179560,00                                | 7,61       |  |  |  |  |
| 2009            | 0,84  | 0,92                              | 0,84   | 130665,84                                | 127310,16                                | -2,57      |  |  |  |  |
| 2010            | 0,25  | 0,63                              | 0,40   | 97750,71                                 | 75038,98                                 | -23,23     |  |  |  |  |

#### Table 18 Linnefe - goodness-of-fit

|                 |                                  | Model 4: Linnefe<br>Calibration point: Linnefe hydrometric station |  |  |  |            |  |  |  |
|-----------------|----------------------------------|--|--|--|--|------------|--|--|--|
|                 |                                  |  |  |  |  |            |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency | Correlation<br>coefficient<br>(r)                                  | Coefficient of<br>determination<br>(R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |
| 2004            | undeterm.                        | undeterm.  | undeterm.  | 42218,98                                 | 29945,94                                 | -29,07     |  |  |  |

#### Table 19 Scherfbach - goodness-of-fit

|                 | Model 5: Scherfbach              |   |  |  |  |            |  |  |
|-----------------|----------------------------------|---|--|--|--|------------|--|--|
|                 |                                  | Calibration point: Scherfbach hydrometric station |  |  |  |            |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency | Correlation<br>coefficient<br>(r)                 | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |
| 2001            | 0,25                             | 0,69  | 0,48   | 157392,48                                | 142357,22                                | -9,55      |  |  |
| 2002            | 0,69                             | 0,89  | 0,78   | 166789,39                                | 161892,63                                | -2,94      |  |  |

#### Table 20 Mutzbach - goodness-of-fit

|                 | Model 6: Mutzbach                |   |  |  |  |            |  |  |  |
|-----------------|----------------------------------|---|--|--|--|------------|--|--|--|
|                 |                                  | Calibration point: Dünnwald hydrometric station |  |  |  |            |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency | Correlation<br>coefficient<br>(r)               | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |
| 1996            | -1,71                            | 0,35  | 0,12   | 25044,05                                 | 38741,77                                 | 54,69      |  |  |  |
| 1997            | 0,52                             | 0,75  | 0,56   | 24493,88                                 | 35988,24                                 | 46,93      |  |  |  |
| 1998            | 0,16                             | 0,16 0,50 0,25 41308,95                         |  |  |  |            |  |  |  |
| 1999            | undeterm.                        | undeterm.                                       | undeterm.                                      | 67416,98                                 | 66015,82                                 | -2,08      |  |  |  |



#### Table 21 Katterbach - goodness-of-fit

|                 |   | Model 7: Katterbach                                |      |  |  |            |  |  |  |
|-----------------|---|--|------|--|--|------------|--|--|--|
|                 | Calibration point: Paffrath hydrometric station |  |      |  |  |            |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe<br>Efficiency                | Nash-<br>Sutcliffe Correlation d<br>Efficiency (r) |      | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |
| 1997            | -1,60   | 0,16   | 0,03 | 21597,91                                 | 14214,57                                 | -34,19     |  |  |  |
| 1998            | 0,23  | 0,53   | 0,28 | 20237,06                                 | 18234,99                                 | -9,89      |  |  |  |
| 1999            | 0,44  | 0,68   | 0,46 | 23822,60                                 | 20532,52                                 | -13,81     |  |  |  |
| 2000            | -1,02   | 0,41   | 0,17 | 23000,06                                 | 19704,56                                 | -14,33     |  |  |  |

## Table 22 Lower Große Dhünn - goodness-of-fit

|                 | Model 9: Lower Große Dhünn<br>Calibration point: Hummelsheim   |      |  |  |  |            |  |  |  |
|-----------------|--|------|--|--|--|------------|--|--|--|
|                 |  |      |  |  |  |            |  |  |  |
| Hydrol.<br>year | Nash- Correlation<br>Sutcliffe coefficient c<br>Efficiency (r) |      | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |
| 2001            | 0,70   | 0,84 | 0,71   | 881660,06                                | 946265,94                                | 7,33       |  |  |  |
| 2002            | 0,60   | 0,87 | 0,76   | 1240515,30                               | 1547301,80                               | 24,73      |  |  |  |
| 2003            | 0,80   | 0,95 | 0,90   | 912519,31                                | 1199986,40                               | 31,50      |  |  |  |
| 2004            | 0,67   | 0,87 | 0,75   | 585426,94                                | 747643,44                                | 27,71      |  |  |  |
| 2005            | 0,62   | 0,81 | 0,66   | 1128323,90                               | 1129897,90                               | 0,14       |  |  |  |
| 2006            | 0,66   | 0,86 | 0,75   | 1174866,30                               | 934133,50                                | -20,49     |  |  |  |
| 2007            | 0,29   | 0,68 | 0,46   | 2171166,50                               | 1244623,60                               | -42,67     |  |  |  |
| 2008            | 0,67   | 0,82 | 0,68   | 1382027,00                               | 1325601,50                               | -4,08      |  |  |  |
| 2009            | 0,65   | 0,89 | 0,79   | 819010,63                                | 988854,38                                | 20,74      |  |  |  |
| 2010            | 0,70   | 0,85 | 0,71   | 691429,00                                | 700458,31                                | 1,31       |  |  |  |
| 2011            | -0,01  | 0,78 | 0,61   | 1033586,80                               | 1174271,60                               | 13,61      |  |  |  |
| 2012            | 0,14   | 0,78 | 0,61   | 1034316,30                               | 841631,63                                | -18,63     |  |  |  |
| 2013            | -0,37  | 0,83 | 0,69   | 707470,63                                | 864861,56                                | 22,25      |  |  |  |
| 2014            | 0,65   | 0,81 | 0,66   | 578688,75                                | 623512,88                                | 7,75       |  |  |  |



|                                      | Model 9: Lower Große Dhünn |                                   |  |  |  |            |  |  |  |  |  |
|--------------------------------------|----------------------------|-----------------------------------|--|--|--|------------|--|--|--|--|--|
|                                      | Calibration point: Manfort |                                   |  |  |  |            |  |  |  |  |  |
| Hydrol.<br>year<br>Efficiency<br>(r) |                            | Correlation<br>coefficient<br>(r) | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |  |  |
| 2001                                 | 0,68                       | 0,87                              | 0,76   | 1438145,10                               | 1134187,75                               | -21,14     |  |  |  |  |  |
| 2002                                 | 0,74                       | 0,88                              | 0,78   | 1804021,90                               | 1724359,00                               | -4,42      |  |  |  |  |  |
| 2003                                 | 0,90                       | 0,95                              | 0,90   | 1369247,00                               | 1358492,40                               | -0,79      |  |  |  |  |  |
| 2004                                 | 0,62                       | 0,81                              | 0,66   | 1008519,31                               | 877851,00                                | -12,96     |  |  |  |  |  |
| 2005                                 | 0,62                       | 0,86                              | 0,73   | 1326200,90                               | 1294756,40                               | -2,37      |  |  |  |  |  |
| 2006                                 | 0,78                       | 0,89                              | 0,79   | 1147012,00                               | 1069845,00                               | -6,73      |  |  |  |  |  |
| 2007                                 | 0,51                       | 0,83                              | 0,68   | 1403875,40                               | 1380349,50                               | -1,68      |  |  |  |  |  |
| 2008                                 | 0,49                       | 0,77                              | 0,59   | 1450408,40                               | 1490613,40                               | 2,77       |  |  |  |  |  |
| 2009                                 | 0,55                       | 0,87                              | 0,76   | 1145763,10                               | 1139766,80                               | -0,52      |  |  |  |  |  |
| 2010                                 | 0,60                       | 0,84                              | 0,71   | 1075924,75                               | 845421,00                                | -21,42     |  |  |  |  |  |
| 2011                                 | -0,46                      | 0,71                              | 0,51   | 1258460,60                               | 1370347,50                               | 8,89       |  |  |  |  |  |
| 2012                                 | 0,11                       | 0,80                              | 0,64   | 1184030,40                               | 973260,81                                | -17,80     |  |  |  |  |  |
| 2013                                 | 0,38                       | 0,84                              | 0,71   | 1089217,50                               | 995150,44                                | -8,64      |  |  |  |  |  |
| 2014                                 | 0,55                       | 0,83                              | 0,68   | 938515,69                                | 724303,13                                | -22,82     |  |  |  |  |  |

#### Table 23 Lower Große Dhünn - goodness-of-fit

#### Table 24 Lower Große Dhünn - goodness-of-fit

|                 | Model 9: Lower Große Dhünn                       |      |  |  |  |            |  |  |  |
|-----------------|--|------|--|--|--|------------|--|--|--|
|                 | Calibration point: Schlebusch                    |      |  |  |  |            |  |  |  |
| Hydrol.<br>year | Nash-<br>Sutcliffe coefficient<br>Efficiency (r) |      | Coefficient of determination (R <sup>2</sup> ) | Obs.<br>acccum.<br>yearly<br>volume [m³] | Sim.<br>acccum.<br>yearly<br>volume [m³] | RVE<br>[%] |  |  |  |
| 2007            | 0,42   | 0,82 | 0,66   | 1474280,60                               | 1335154,40                               | -9,44      |  |  |  |
| 2008            | 0,59   | 0,83 | 0,68   | 1448857,90                               | 1433589,30                               | -1,05      |  |  |  |
| 2009            | 0,57   | 0,89 | 0,79   | 1065826,00                               | 1088140,10                               | 2,09       |  |  |  |
| 2010            | 0,60   | 0,82 | 0,68   | 950311,31                                | 795792,38                                | -16,26     |  |  |  |
| 2011            | -0,18  | 0,78 | 0,61   | 1042585,40                               | 1299565,10                               | 24,65      |  |  |  |
| 2012            | -0,34  | 0,78 | 0,61   | 912307,75                                | 930875,44                                | 2,04       |  |  |  |
| 2013            | 0,01   | 0,84 | 0,70   | 896761,75                                | 954452,63                                | 6,43       |  |  |  |
| 2014            | 0,71   | 0,86 | 0,73   | 747443,50                                | 690717,75                                | -7,59      |  |  |  |
| 2015            | 0,70   | 0,85 | 0,73   | 837859,19                                | 727770,94                                | -13,14     |  |  |  |

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### 5. The Netherlands

#### 5.1. Model objectives in BINGO

The Veluwe area is a large ice-pushed moraine in the central part of the Netherlands. The elevated sandy area contains a strategic groundwater body that is important for Dutch drinking water production. The area is primarily a nature area with forest, heathland and drift-sands. The moraine complexes (cross section of 5- 50 km; height of 30-100 m) have a groundwater system that responds relatively slowly to hydrological interventions and weather. Large scale drainage systems or surface water bodies are absent in the central part of the Veluwe. Drainage occurs primarily at the borders of the Veluwe through brooks and streams and through upward seepage in agricultural areas surrounding the Veluwe. The Veluwe can be considered as one large hydrogeological system that is continuously searching for a hydrological equilibrium (see also Deliverable 3.1 for more site specific information).

Under a warming climate the water quality of large rivers in the Netherlands might degrade (Delpla et al. 2009), especially during dry periods with low river discharge (van Vliet and Zwolsman, 2008; Zwolsman and van Bokhoven 2007). Moreover, climate projections predict that precipitation shifts from summer to winter and that potential evapotranspiration in summer increases, meaning that dry spells will occur more frequently and intensively (KNMI 2014). Providing the water requirements of nature, agriculture and drinking water in the Netherlands is therefore under pressure. Dutch authorities are particularly concerned about the sandy southern and eastern regions of the Netherlands because sandy soils of these regions have a poor water holding capacity, which renders them more vulnerable to dry spells. Large groundwater bodies, such as the Veluwe, could therefore form an important source of water during dry periods. To ensure sufficient and high quality water at acceptable costs, it is likely that the dependency on groundwater of the Veluwe will increase. This implies, for instance, that drinking water companies intensify or relocate current groundwater extractions, or install new water production sites. For sustainable management of groundwater resources, groundwater extractions should be balanced with the amount of precipitation that percolates to the saturated zone, the groundwater recharge. A major part of precipitation water reaching the Veluwe does not lead to groundwater recharge as it turns into evaporation from the soil surface and transpiration by plants, i.e. into evapotranspiration. From the perspective of groundwater recharge, this water loss can be influenced by nature management. In general, more green leaf area leads to a larger water loss by transpiration and interception, and to a decrease in groundwater recharge. Interventions, however, also affect surrounding areas via the groundwater system.

Regionally, different and often contrasting interests prevail. More numerous or intensive groundwater extraction may lead to lower discharge in streams and canals, to drawdown of the groundwater head, and affect adjacent nature areas. Nature areas situated on infiltration areas (e.g. elevated sandy soils) supply topographically lower land (wet ecosystems or agricultural fields) with seepage water, hence changes in recharge may lead to water logging or drought and affect agricultural productivity and nature targets. Such conflicting interests call for an integrated approach, where all interests are tuned to each other.

Both land use and climate control the water balance of the Veluwe area. Because the Veluwe area responds slowly to hydrological changes, we will evaluate the effects of land use and climate change on water resources of the Veluwe area and related functions in a historical perspective within the BINGO project. The model objectives of this research are to quantify the effect of climate and land use changes for i) historical conditions (from the 17<sup>th</sup> century), ii) the recent past (1980-2015), iii) the near future (2015-2025) and iv) the



far future (2050 to 2100). For these different time periods we will focus on long term effects as well as effects of dry spells (series of dry years) on groundwater levels.

### 5.2. Model application

### 5.2.1. Model description

We will use the groundwater model AZURE to simulate groundwater levels and discharges of discharge of groundwater to surface water, drainage and overlandflow of the Veluwe. The AZURE model (de Lange & Borren 2014) is based on MODFLOW (Harbaugh et al. 2000) for the simulation of groundwater flow and MetaSWAP (van Walsum and Groenendijk 2008; Walsum et al. 2010) to simulate evapotranspiration and groundwater recharge (Figure 68). The model has been developed and calibrated prior to the BINGO project. Basic features and characteristics are explained here. For more detail descriptions we would like to refer to de Lange and Borren (2014).

We will use crop factors multiplied by the Makkink reference crop evapotranspiration (Makkink 1957), to simulate potential interception evaporation, potential transpiration and potential soil evaporation. Actual interception evaporation is simulated according to the Rutter et al. (1971) model, actual transpiration is simulated using a water stress reduction function of Feddes et al. (1974) which accounts for the closure of leaf stomata during water stressed periods, and actual soil evaporation is simulated using Boesten and Stroosnijder (1986). For a detailed description of MetaSWAP coupled to MODFLOW we refer to Walsum et al. (2010).



Figure 75 Model structure of MetaSWAP coupled to MODFLOW



## 5.2.2. Data

AZURE model consists of 9 hydrogeological layers that have a horizontal ( $K_h$ ) and vertical conductivity ( $K_v$ ). The national Dutch hydrogeological database REGIS 2.1 has been used to determine the initial conductivities before calibration (paragraph 5.2.3). Major soil textures are sandy gravel, sand and clay. Because the Veluwe is an ice pushed ridge, the geological setting is quite complex and difficult to determine from coring's. In parts of the Veluwe, the layering is tilted to an approximate angle of 30 to 60 degrees due to the pushing forces of the ice sheet (see Figure 76). These complex features are accounted for by using different horizontal anisotropy factors (the ratio's between  $K_h$  and  $K_v$ ) in row (x) and column (y) direction. Regions with the largest deformations received a low anisotropy value (i.e. a relatively low  $K_h$ ) in the direction perpendicular to an ice pushed ridge.

The model boundaries of AZURE are defined as constant head boundaries. The extent of the model is chosen in such a way that a constant head boundary coincides with large rivers ('IJssel' and 'Rijn') or lake 'IJsselmeer'. Large lakes, as the 'IJsselmeer', are simulated as constant head boundaries as well. For terrestrial parts, the upper boundary is controlled by recharge estimated by MetaSWAP. AZURE has a minimum spatial resolution of 25 m x 25 m. For the BINGO project we will run the model at 250 m x 250 m since we are interested in large scale patterns and to save memory and computing time. The AZURE-model uses daily meteorological data.

The Veluwe will be studied from the 18<sup>th</sup> century (1750 AD) onwards and forms an example of an area where large-scale changes in land cover and drainage of surrounding areas have occurred during the past centuries (Koster 1978). The historical changes in land cover will be translated to changes in groundwater recharge. The translation of land cover to groundwater recharge will be based on historical observations of precipitation and temperature, available since 1706 (Labrijn 1945) and translated by the Royal Dutch Meteorological Institute (HISKLIM project) to meteorological station De Bilt (centre of the Netherlands). Insufficient historical data are available to derive the Makkink reference crop evapotranspiration. Therefore, air temperature will be translated to Makkink reference crop evapotranspiration, using a relationship between these two quantities based on modern observations at De Bilt.

For the recent past (1980 to 2015) and near future (2015 to 2025) we will use the simulated climate data generated by WP2. For the far future (2050, 2100) we will use climate scenarios of the Royal Dutch Meteorological Institute (KNMI 2014).

The output of AZURE will be ascii rasters of long term average groundwater recharge and lowest, average and highest groundwater head. For dry years en the consecutive 3 years we will produce quarter of a year average groundwater head and compare sequences of years to assess the response time of the system to dry years.





Figure 76 Different regions (1 to 8) with tilted geological layers

## 5.2.3. Calibration and validation strategies

The AZURE model was calibrated in 4 different stages on data of groundwater observation wells and on outflow data from 1995 to 2005 of different polder areas. First, the model was calibrated in a stationary run, optimizing  $K_h$  and  $K_v$ . Second, the model was calibrated in a stationary run optimizing anisotropy factors. Third, a non-stationary simulation was performed to optimize  $K_h$ ,  $K_v$  and the unconfined storativity for elevated regions. Forth, a non-stationary run was used to optimize  $K_h$ ,  $K_v$ , the unconfined storativity, the riverbed conductance and the drainage conductance. Basically, the latter two calibration steps focused on two different regions: elevated sandy soils and the polder area.

The representer-technique (Valstar 2007; Valstar et al. 2004) was used during stationary calibration. The representer approach is especially advantageous in groundwater problems where the total number of measurements is often small. The representer approach uses mathematical function to describe variability of model parameters in respect to individual measurements. It determines how far away a parameters is still adjusted and how large the adjustment can be.

For non-stationary runs the PEST module in IMOD (Vermeulen et al. 2016) was used for calibration. The module uses the Levenberg-Marquardt algorithm to minimize the model error by adjusting the model parameters. The further the model errors are from there minimum, the larger the adjustment of model parameters is

After each calibration step, experts (modelers and field experts) assessed whether the model results and parameter estimations fall within realistic limits and decided if parameter limits should be adjusted before continuing to the next calibration step.



## 5.2.4. Results

Groundwater levels of the calibrated AZURE model are on average 20 cm lower than observed. Deviations are larger for elevated areas, and lower at the ridges of the ice-pushed moraine (see Figure 77). The simulated dynamic range (the variation between the minimum and maximum groundwater heads) is on average 8 cm larger than the observed dynamic range. For the greater part, the residuals after calibration show a normal distribution. However, there are also clustered deviations, especially for elevated areas where simulated groundwater heads are too low.

Calibration based on observed in- and outflow of polder areas did not lead to much improvement of the model. These fluxes seem to be insensitive for parameter adjustments that were made during calibration.





Figure 77 Model area of the Veluwe research site, the difference between modelled and measured median (p50) of the 1995 – 2005 timeseries of groundwater heads and the average simulated groundwater head in meters above mean sea level (m+msl)

In figure 78 the calculated average discharge of groundwater to surface water, drainage and overlandflow is shown. On the higher part of the Veluwe there is no discharge. The groundwater levels are far below surface level that the don't reach the surface water. More on the edges of the Veluwe, where groundwater levels are closer to surface, discharge starts to take place





Figure 78: Combined average discharge from groundwater to surface water, drainage and overlandflow

## 5.3. Model evaluation and discussion

The AZURE model is a physically based model that is able to simulate groundwater heads and evapotranspiration for large regions. The state of the art coupled simulation of evapotranspiration and groundwater flow is well suited for climate and land use change analyses. Considering the complex hydrogeological system, we are satisfied with the model performance. With the exception of observation wells in elevated areas, most observation wells show a good agreement with the observed groundwater head.

In general, the AZURE model is drier than observed in elevated areas, which implies that it has difficulties in capturing observed steep gradients in hydraulic head at the fringe of the Veluwe. In general, the Veluwe

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system is a very complex hydrogeological structure. Due to not enough knowledge about these structures, this is simplified in de model. So probably some conceptual parts of the model do not describe the processes very well, or important geological structures are merely absent in the model. Tuning parameters to much with these simplified concepts does not necessarily lead to a better model, on the contrary. A model that is optimized on observations without considering realistic parameter values may lead to good results based on wrong reasons (Guisan & Zimmermann 2000) obviously, such a model is unsuitable for simulating effects of climate change. Therefore, the judgement of the experts about realistic parameter limits and field experts knowing the details of existing geological structures was a key feature during the calibration process.

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#### 6. Norway

#### 6.1. Model objectives in BINGO

The water resources modelling at the Bergen research sites create a platform for analyzing the impact of future changes in extreme weather on the drinking-water resources and stormwater systems. Thus, the overall goals of the BINGO project in relation to the Bergen research site are the development of methods/tools for forecasting the available water resources for drinking water supply, and assessment of the impact of climate change and increased frequency of weather extremes on CSO discharge. The specific objectives of the modelling works of BINGO at the Bergen research site in Norway are two. These are, analyzing the risk of drought and assessing the availability of future water resource for drinking water supply, and assessing climate change induced impacts on the stormwater and the recipients. The respective activities being undertaken are hydrological simulation of the inflows into the reservoirs that provide the city's drinking water supply, and modelling of the hydrology and stormwater drainage system of the Damsgård area. As described in the Description of Action (DOA), Bergen County has an operational Sewer and Stormwater model (i.e. as separate sewer or combined sewer system) covering most of Bergen City, including parts of the Damsgård area. Research efforts in the BINGO project has also delved into separating the stormwater and sewer system models, in line with the Bergen County's desire to separate the two systems as a means to overcome the present threat the extensive flash floods from the hillsides posed to the residential areas in Damsgård.

Jordalsvannet, Svartediket, Sædalen and Espeland are the main lakes that provide Bergen's drinking water supply. The area contributing the drinking water resource has a size of 31.9 km<sup>2</sup> and the elevation above mean sea lever ranges between 76 m to 983 m. The inflows into the reservoirs are low – in the winter, moderate – between April and September, and high in autumn (i.e. October to the end of December). The Damsgård area, the site of stormwater system modelling, has a size of around 8.3 km<sup>2</sup> and the elevation ranges from sea level to 468 m above mean sea level. According to <a href="http://www.nevina.no">http://www.nevina.no</a>, the mean specific runoff of the catchments contributing to the drinking-water reservoirs and of the Damsgård area are 115 l/s/km<sup>2</sup> and 70 l/s/km<sup>2</sup>, respectively. For detailed description of the research sites, please refer to D3,1.

The models selected are the HBV-model (Hydrologiska Byråns Vattenavdelning; Bergström (1976)) – for modelling the drinking water resources of Bergen City, and the SWMM (Storm Water Management Model; Rossman (2015)) – for modelling the stormwater system of the Damsgård area.

The HBV-model is a hydrological model widely applied in the Northern European countries. It performs well in capturing the hydrological process in cold climate regions and renders the benefit that it can adequately reproduce observations without being too complex. A regional HBV-model for the Bergen region developed and set up during the pre-study phase of the BINGO project is in use. Through description of the model, its calibration and validation, and discussion of the results are documented as part of a thesis work by Kristvik & Riisnes (2015).

SWMM has been selected to model the stormwater system of the Damsgård area mainly because it is an open source software and the availability of its source code, which gives the flexibility to improve the runoff routine of the model. Furthermore, since the Bergen County plans to improve the water quality of the Puddefjord, the model setup can later be used for water quality simulation as simulation of quantity and quality problems associated with runoff from urban areas can be made using SWMM. This was found necessary because the system in some part of the study area is not included in the existing Mike Urban-based model. There are no flow records in the section of the system left out of the model and efforts made



to transfer model parameters revealed the need for vigorous calibration of the existing model. The results are documented in a bachelor thesis project of Scheibler (2016).

### 6.1.1. Model application

#### 6.1.1.1. Model description

#### **HBV-model**

The HBV-model was first developed in the 1970's and is a widely applied tool for hydrological runoff simulations in Nordic countries (Bergström, 1992). The HBV model is a conceptually-based, deterministic rainfall-runoff model. Over the years, it has been developed and improved – moving from a lumped model to a distributed model (Bergström, 1998). The HBV-model set up for the Bergen research site is a semi-distributed version and the schematics of the model structure are presented in Figure 79. It is semi-distributed in the sense that it takes into account the hypsographic distribution of the catchment by splitting the area into ten elevation zones. The model uses temperature, precipitation and evapotranspiration as input. Firstly, this input enters the *snow routine*, which includes the estimation of dry snow storage, wet snow storage, snow melt, and snow refreezing. Based on this, it calculates the water that is further transferred to the *soil moisture routine*. In the soil moisture routine, the amount of water that stays in the unsaturated zone, how much is evaporated and transpired, and the amount of water that continues to the *response routine* is estimated. The response routine is divided into quick runoff (upper zone) and slow runoff (lower zone). The calculated responses in the response routine comprise the simulated runoff from the catchment.



# Figure 79 Schematic description of the HBV model structure where P is precipitation, T is temperature, EPOT is potential evapotranspiration, and R is generated runoff

Each routine has its own set of model specific parameters and the regional HBV-model that exist for Bergen comprise of values for these parameters. The regional model is divided into of five sub-models representing the five catchments surrounding the city's drinking water reservoirs. The analyses presented in this report are performed for the one of these catchments, Svartediket, which was chosen due its location close to the city center and relevant weather stations.

#### Stormwater model

The Environmental Protection Agency's (EPA) SWMM is a dynamic rainfall-runoff simulation model used for simulating quantity and quality of flows mainly from urban areas (Rossman 2015). Since its development by the EPA in 1971, SWMM has gone through several major upgrades, and is widely used for planning, designing and analyzing urban stormwater and drainage systems. Some of SWMM's typical applications

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include, sizing of detention facilities, flood plain mapping of natural channel systems, designing control strategies for minimizing combined sewer overflows, evaluating the impact of inflow and infiltration on sanitary sewer overflows, and generating non-point source pollutant loadings for waste load allocation studies. As illustrated in Figure 80 SWMM represents the water/material cycle between major environmental compartments through a series of flows. It can simulate all aspects of urban hydrologic and water quality cycles (e.g. rainfall, snowmelt, surface runoff, etc.).

There are several scientific publications, which applied SWMM for different purposes. Rai (2016) assessed application of SWMM for simulation of floods in river systems in the Brahmani river delta (India). They calibrated their model using a Monte Carlo based optimization procedure and sampling the parameter space using Latin-hypercube sampling method and concluded that the performance of SWMM for modelling river systems is very good. Li et al. (2015) successfully used the SWMM model to investigate the merits of implementing low impact development facilities (i.e. gravel infiltration retention system and depressive green field) in Guangzhou, China. Likewise, Rosa (2015) simulated the runoff and nutrient export from a low impact development watershed in USA and examined sensitivity of some of the model parameters.



#### Figure 80 Schematic illustrations of the processes modeled by SEMM. (Source: Rossman 2015)

The runoff routine of SWMM generates runoff from precipitation and can be employed for event-based or continuous simulation. It comprises a hydraulics component that mainly transports the runoff generated from a collection of sub-catchments to the desired outlet through a series of conduits and storage elements. In this project, SWMM 5.1 (version 5) is being used. This SWMM version is a complete re-write of the previous release and a C++ version of the source codes is available. The two main inputs model SWMM requires are physiographic information of the study area and detailed information of the stormwater system.



## 6.1.1.2. Data

## HBV-model

The HBV-model is run with a daily temporal resolution and uses data for precipitation, temperature, and evapotranspiration as input to simulate runoff. In addition, some catchment specific characteristics must be defined. Information on the model input and data sources used in the work presented in this report are found in Table 25 and Table 26.

#### Table 25 Input data to the HBV-model for Svartediket

|     | Variable                        | Format | Spatial resolution | Temporal resolution | Time span | Source  |
|-----|---------------------------------|--------|--------------------|---------------------|-----------|---|
| (1) | Precipitation(obs)              | .xlsx  | Station            | Daily               | 1980-2014 | 50540 Florida Weather   |
| (2) | Temperature(obs)                | .xlsx  | Station            | Daily               | 1980-2014 | www.eklima.no)  |
| (4) | Precipitation(sim)              | .csv   | Station            | Daily               | 1980-2014 | Downscaled data bias  |
| (5) | Temperature(sim)                | .csv   | Station            | Daily               | 1980-2014 | corrected to 50540<br>Florida Weather Station<br>(https://freva.met.fu-<br>berling.de)  |
| (3) | Potential<br>evapotranspiration | .xlsx  | Station            | Daily               | 1980-2014 | Derived from temperature<br>data (2) and (4) by the<br>Thornthwaite method <sup>2</sup> |

# Table 26 Catchment specific parameters for Svartediket generated by NVE's (The Nowegian Water Resources and Energy Directory) catchment generator NEVINA (NVE, 2016)

| Parameter  | Value   |
|--|---|
| Hypsographic distribution:   |   |
| $(H_{min}, H_{10}, H_{20}, H_{30}, H_{40}, H_{50}, H_{60}, H_{70}, H_{80}, H_{90}, H_{100})$ | (60, 147, 260, 338, 406, 459, 495, 532, 570) m.a.s.l. |
| Catchment area   | 12.3 Km <sup>2</sup>                                  |
| Lake percentage  | 4.1 %   |
| Specific runoff  | 105 l/Km <sup>2</sup>                                 |



#### Stormwater model

The stormwater system comprised 103 manholes (nodes) and 2068 m long circular conduits of varying sizes. Figure 81 illustrates the system and its elements. In system is a combined sewer system that collects sewerage from 35 sub-catchments of 26.1 ha size. Detail information of the system and all the features was obtained from the Norwegian Water and Wastewater systems' database (Gemini), and the existing model.



#### Figure 81 Sub-catchments and the stormwater network

Flow records are scarce in the project area. Installation of one or more flow meter in the stormwater pipe systems is underway in the BINGO project. The data available for calibrating the stormwater model were collected during flow measurement campaigns conducted in the period between 2001 and 2008. The data series have short length with a record at a given campaign station covering between 35 to 173 days. Thus, records from two of these stations (SSBMV and SSKROHN) were used. The former was used for evaluating the stormwater model calibration, even though the records quantify the combined flow. As can be seen in Table 27, the later station provided data series of the sewerage flow collected at a separate sewer system. The records from this station were used to calculate the per capita sewerage production at a 5 minutes interval observation time step. The mean per capita sewage production calculated on the dry days equaled 107.5 I (range from 82 to 162 l/p.d), which is within the range Ødegaard (2014) provided for Norway. As shown in Figure 82, the peak sewerage flow hours inferred from the records coincide with the hourly sub-daily variation. Using these two data series, a stormwater flow data series was generated at SSBMV station.




Figure 82 Boxplots of the sub-daily variability of the sewer flow over dry days. The red line and the blue dotes respectively designate the mean value for the Damsgård area and the mean value for Norway (Ødegaard, 2014)

Digital elevation model of 10 m resolution was obtained from Kartverk's website. Tabular summary of the data types, observation time steps and sources are given in Table 27.

| Station Code    |                 | Temporal      | Pe         | riod       | System/Data   | Data             |  |
|-----------------|-----------------|---------------|------------|------------|---------------|------------------|--|
|                 |                 | Resolution    | From To    |            | Туре          | Source           |  |
|                 | SSBMV           | 5 min.        | 29.04.2004 | 25.08.2004 | Combined      |                  |  |
| es              | SSLILLED        | 5 min.        | 26.05.2004 | 25.08.2004 | Combined      |                  |  |
| Time seri       | SSFROYA         | 5 min.        | 26.08.2004 | 09.12.2004 | Sewer         |                  |  |
|                 | SS61RG          | Real time     | 02.05.2004 | 24.08.2004 | Precipitation | Bergen<br>County |  |
|                 | SS73RG          | Real time     | 02.05.2004 | 24.08.2004 | Precipitation |                  |  |
| Geospatial data | Stormwater s    |               |            |            |               |                  |  |
|                 | Sub-catchme     |               |            |            |               |                  |  |
|                 | Land use and    | Kartverket    |            |            |               |                  |  |
|                 | Digital elevati | kartverket.no |            |            |               |                  |  |

#### Table 27 Type, source and description of data used for the stormwater model



# 6.1.1.3. Calibration and validation strategies

#### HBV-model

The regional HBV-model for Bergen was already calibrated and validated in previous studies. Detailed descriptions are found in Kristvik & Riisnes (2015). In this report, the model is validated for the time period 1980-2014. Validation is performed by comparing the simulated inflow (i.e. adopting parameterization of the regional HBV-model) to a reference inflow (i.e. obtained by areal scaling from an adjacent and gauged catchment (Kristvik & Riisnes, 2015)).

# Stormwater model

Stormwater data has not been available for calibrating the stormwater model. As highlighted in the previous subsection, attempt has been made to generate a stormwater flow series at SSBMV station by deducting the mean per capita sewage production from the combined flows. In addition to the data uncertainty, there is no evidence whether there are intakes/inlets into which the runoff generated from the mountainous part of the catchment lying along the western edge goes in. This was noted during attempts to auto-calibrate the model and measure the performance of the model with the commonly used efficiency metric (e.g. the sum of squares error). It was noted that the estimated flows at the catchment outlet were much higher than flows observed in the system. Efforts made to force the estimated flows to match the observed ones led to unrealistic distribution of water in the water balance components of the system.

Hence, the model was calibrated in such a way that the simulated hydrograph somehow follows the response pattern of the observed flows. Considerable consideration was given to the runoff coefficient SWMM reported at the end of the computations. The aim has been to bring the runoff coefficients of the largely undeveloped forest covered mountainous sub-catchment up to 0.2 and that of the predominantly urbanized sub-catchments above or equal to 0.6.

In order to evaluate the performance of the model using data other than the one used for calibration an internal station was selected. The 5 minute interval flow series at the SSLILLED station has a slightly shorter length with a discontinuity of 23.2 % over the period shown in Table 27. The gauging station is located in the southern tip of the catchment contributing to the SSBMV (Figure 83). Its catch area equals 12.7 ha and consists a pipe network of 407.4 m length joined one another through 20 manholes.





Figure 83 Location of the flow stations proposed for calibration and validation of the model

# 6.1.1.4. Results

#### HBV-model

Table 28 shows the Nash Sutcliffe criterion (NSE) between simulated inflow and the reference inflow. For the validation, the NSE for daily runoff is lower than the calibrated results (0.776 vs. 0.801), but improves when looking to monthly values (0.882). The simulated monthly runoff for the period 1980-2014 is presented in Figure 76. Because low flows are of interest, the logarithmic NSE of monthly runoff was also calculated, yielding a value of 0.888. This implies that the model performs well with regards to the objectives related to water supply in Bergen, as monthly variations and low flows are of particular interest. Furthermore, the daily



runoff simulated for the winter 2009/2010 is presented in Figure 77. This illustrates the model's ability to reproduce the dry periods that may cause problems to the water supply.

| Table 28 Comparie | son of NSE from model | calibration (Kristvik & | Riisnes, 2015) a | nd model validation |
|-------------------|-----------------------|-------------------------|------------------|---------------------|
|-------------------|-----------------------|-------------------------|------------------|---------------------|

|   | NSE   |
|---|-------|
| Calibration (daily 1980-2009)<br>(Kristvik & Riisnes, 2015) | 0.801 |
| Validation (daily 1980-2014)                                | 0.776 |
| Validation (monthly 1980-2014)                              | 0.882 |



Figure 84 Monthly runoff simulated for Svartediket in the timeperiod 1980-2014





#### Figure 85 Simulated daily runoff winter 2009-2010

#### Stormwater model

One of the main results is the runoff coefficent SWMM computed for each calibration from the model calibration. This was set as an objective during the model calibration. The values for each sub-catchment are provided in Table 29.

| Catc. ID | С    |
|----------|------|----------|------|----------|------|----------|------|
| 2        | 0.66 | 11       | 0.16 | 20       | 0.21 | 29       | 0.11 |
| 3        | 0.67 | 12       | 0.12 | 21       | 0.62 | 30       | 0.62 |
| 4        | 0.75 | 13       | 0.16 | 22       | 0.20 | 31       | 0.64 |
| 5        | 0.18 | 14       | 0.79 | 23       | 0.19 | 32       | 0.63 |
| 6        | 0.15 | 15       | 0.74 | 24       | 0.09 | 33       | 0.07 |
| 7        | 0.11 | 16       | 0.70 | 25       | 0.20 | 34       | 0.15 |
| 8        | 0.19 | 17       | 0.74 | 26       | 0.19 | 35       | 0.15 |
| 9        | 0.14 | 18       | 0.65 | 27       | 0.71 | 36       | 0.14 |
| 10       | 0.14 | 19       | 0.64 | 28       | 0.63 |          |      |

#### Table 29 Runoff coefficients (C) obtained for each sub-catchment after model calibration

Figure 86 shows the simulated and observed flows at the SSBMV and SSLILLED stations. The former is used for calibration and the later for verifying performance of the model.







Figure 86 Simulated and observed flows at SSBMV (upper) and SSLILLED (lower) stations

# 6.1.2. Model evaluation and discussion

#### HBV-model

Validation of the HBV-model for the Bergen region has revealed that model performance, measured by the Nash-Sutcliffe criterion, is adequate. The model performance is good both for daily and monthly runoff simulations and it is concluded that the model is well suited for further investigations of water supply in Bergen. Thus, the specific model set-up outlined herein will be kept in future simulations of inflow using input data produced in WP2 of the BINGO project.

#### Stormwater model

Although the model has not been calibrated optimally due to the aforementioned challenges, the model has yielded minimal (< 1%) continuity errors for both the surface runoff and the conduit routing. The model parameterization has yielded runoff coefficients higher than 0.6 for the sub-catchments located in the more urbanized part of the city, which goes along the Norwegian guidelines for stormwater management (NVE 2015). Visual evaluation of the goodness of fit between the simulated and observed flow series revealed that the parameterization has helped to capture the hydrologic processes well. The model has responded to storm events rightly. The hydrographs at SSBMV and SSLILLED stations show that the volumetric error is enormous. At SSBMV, the station used for calibration, the model overestimated the observations. One possible reason, as highlighted early on in this report, could be that the runoff generated from the entire catchment might not enter the combined system unlike how it is set up in the present model. On the contrary, comparison of the simulated and observed hydrographs at SSLILLED shows that the model underestimates



the runoff recorded at the gauging station. In both cases, however, the simulated peak flows are higher than the observed ones.

The main short coming of the present model set up is the limitation in verifying the information obtained from various data sources; for example, what percentage of the runoff generated from the catchments are collected and transferred through the combined system. On the other hand, the results obtained from the present modelling work, in which a simple calibration technique was implemented, suggests that the model performance can be enhanced by implementing auto-calibration algorithms.

In order to improve performance of the model, the following tasks are online.

- i. Field inspections and verifications whether the sub-catchments are connected to the combined sewer system as designated in the model;
- ii. Auto-calibration of the model to reduce the discordance between the simulated and observed flow series using the Shuffled Complex Evolution algorithm; and
- iii. Validation of the model using an internal gauging station.

#### 6.1.3. Bibliography

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# 7. Portugal

The Portuguese contribution to the BINGO objectives of testing different systems' vulnerabilities to climate change is fourfold:

- 1. Test climate change impacts in *estuary* bordering lands where expected sea level rise associated with more frequent storm surges and salt water intrusion are the driving forces;
- 2. Determine how potential reduction in *groundwater* recharge combined with salt water intrusion can reduce the availability of water to agricultural uses in coastal lowland areas;
- Estimate how vulnerable communities will become to increase *floods* frequency and magnitude in urban areas;
- 4. Assess how the potential changes in *surface water* flow regimes will jeopardize the current and future planned water uses.

Figure 87 locates geographically these issues and its associated systems. In order to get a better legibility of the aquifer systems' position, their configuration is to be found in D3.1.



Figure 87 Different systems modeled in the Tagus basin



The developments already achieved in the modelling component of the scientific approach to these questions will be developed throughout the following sub-sections.

# 7.1. Tagus estuary

# 7.1.1. Model objectives in BINGO

The upper Tagus estuary is bordered by rich agricultural lands. Climate change can affect these lands in two different ways. First, extreme storm events combined with high tides can lead to the overflowing of dikes and the inundation of these fertile lands. This phenomenon represents a potential threat to both people and infrastructures. The sea level rise associated to climate change will exacerbate these risks by increasing the frequency and the extent of the inundations. Secondly, salt water intrusion in the upper estuary occasionally reaches the major water intake that provides fresh water for irrigation of these lands. Both sea level rise and a decline in river flow can promote salt water intrusion, thereby reducing water availability and agricultural productivity. Hence, the goals of the Tagus estuarine models developed within BINGO are twofold: 1) to simulate extreme storm events and the resulting inundation of agricultural land; and 2) to simulate salt water intrusion in the upper estuary.

With a surface area of about 320 km<sup>2</sup>, the Tagus estuary is one of Europe's largest estuaries. A detailed description of this estuary and its margins is presented in a previous report (Deliverable 3.1, Alphen et al., 2016) and will be omitted here for brevity. In order to properly represent all the relevant physical processes, the model domain includes the whole estuary, from the river to the ocean. However, our focus is in the upper estuary (Figure 88).



Figure 88 Tagus estuary: general perspective (left) and detail (right) of the study area in the upper estuary (source: background image from ESRI basemap)



The dynamics of the Tagus estuary were simulated with the community model SCHISM (Zhang et al. 2016a). SCHISM simulates shallow water flows in river to ocean domains, in two and three dimensions, including salt and heat transport and the propagation of wind waves (Roland et al. 2012). The use of unstructured grids makes it particularly adapted to the problem at hand, since it allows a detailed representation of small-scale features that exist in the Tagus estuary (e.g., dikes, narrow channels). SCHISM runs in parallel mode using MPI, which provides the computation efficiency required in BINGO. LNEC participates in the development of this modeling system (e.g., Rodrigues et al. 2009, 2011; Pinto et al. 2012; Azevedo et al. 2014), and has a long experience in applying SCHISM and its predecessor SELFE (Zhang & Baptista 2008) to simulate estuarine and coastal circulation and water quality. Finally, SCHISM is used in LNEC's Tagus forecast systems (Fortunato et al. in press). These reasons have dictated the adoption of SCHISM to simulate the Tagus estuary dynamics.

# 7.1.2. Model application

# 7.1.2.1. Model description

Simulations are performed with the modeling system SCHISM (Zhang et al. 2016a; http://ccrm.vims.edu/schism/), version 5.3.1. This system evolved from SELFE (Zhang & Baptista 2008) and has been developed in several institutions, led by the Virginia Institute of Marine Sciences. SCHISM (*Semi-implicit Cross-scale Hydroscience Integrated System Model*) is a community modeling system based on unstructured grids, which aims at the cross-scale simulation of surface water processes from the river to the ocean. SCHISM uses semi-implicit finite elements and finite volume methods, combined with Eulerian-Lagrangian methods, to solve the shallow water equations. SCHISM is also fully parallelized. The version of SCHISM used herein includes a module for surface waves (Roland et al. 2012).



# SCHISM MODELING SYSTEM

Open-released Ready-to-be-released In-development Free-from-the-web

Figure 89 Structure of the SCHISM modeling system (source: CCRM-VIMS)



SCHISM and its predecessor SELFE have been extensively used to simulate the processes relevant to the present application:

- Applications to extreme events and coastal inundation: e.g., Bertin et al. (2012, 2014), Fortunato et al. (2013, 2016, submitted), Breilh et al. (2014), Zhang et al. (2016b, c). Comparisons with other models are presented in Chen et al. (2013) and Kerr et al. (2013).
- Applications to 3D estuarine circulation: e.g. Rodrigues et al. (2009, 2013, 2015), David et al. (2015), Ye et al. (2016). Detailed assessments of the ability of SELFE to model stratified tidal flows are described in Karna et al. (2015) and Karna & Baptista (2016).

The grid was generated with the softwares xmgredit (Turner & Baptista 1993) and nicegrid (Fortunato et al. 2011), based on grids from previous applications (e.g., Guerreiro et al. 2015; Fortunato et al. in press). The domain is over 110 km long and has a surface area of 1442 km<sup>2</sup>. It extends from the ocean, 27 km away from the estuary mouth, to the river, and includes the agricultural flood-prone areas in the northeastern part of the estuary (Figure 90 a). The primary generation criterion was the detailed representation of the geometry. The channels have a finer resolution, to allow the proper propagation of the tide. Nodes and element sides were placed along the crests of the dykes, to optimize the representation of the elevation of these sub-grid-scale features (Bilskie et al. 2015). The resulting grid has 140 000 nodes and a typical resolution of 15-25 m Figure 90 c).

For simulations in which the inundation of dry areas is not expected, a smaller grid was generated by eliminating the elements in all the areas that are protected by dikes. Reducing the number of nodes decreases the computational costs, in particular for the simulation of salinity intrusion during droughts. This reduced grid has 83 000 nodes.



Figure 90 Model grid and bathymetry: a) global view; b) detail of the inundation-prone areas of the upper estuary; c) distribution of the elements' equivalent diameter. Adapted from Fortunato et al. (submitted)

For the 3D simulations, the vertical grid was setup based on preliminary simulations in which three hybrid vertical grids were tested: 20 SZ levels grid (15 S levels and 5 Z levels), 39 SZ levels grid (30 S levels and 9 Z levels) and 54 SZ levels grid (45 S levels and 9 Z levels). Taking into account both preliminary data-



model comparisons and the computational times, the vertical domain is discretized in a hybrid grid with 39 SZ levels: 30 S levels between 0 and 100 m deep, and 9 Z levels between 100 m and the maximum depth. Further results regarding the vertical grids tested are presented in section 7.1.2.4.

SCHISM is forced by surface water elevations at the ocean boundary and river flow at the river boundaries. For the simulation of the storm events, wind and atmospheric pressure are imposed at the surface. Depthaveraged velocities are also imposed at the ocean boundary for a better representation of the incoming energy. In addition, the wave module is switched on, and waves are also imposed at the ocean boundary. For the salinity intrusion simulations, the model is run in 3D baroclinic mode. In this case, salinity and temperature are imposed at the open boundaries. The heat exchange with the atmosphere is also computed using the model of Zheng et al. (1998) and the model is forced by wind, atmospheric pressure, air temperature, humidity, downwelling shortwave radiation and downwelling longwave radiation at the surface.

Boundary conditions are imposed mostly from large-scale models (Figure 91). Surface water elevations and (when required) depth-averaged velocities at the ocean boundary are imposed from an application of SCHISM to the NE Atlantic Ocean (Fortunato et al., 2016). Surface water waves are determined from an application of the wave model WaveWatch III (Toolman 2009) to the North Atlantic, with a nested grid on the Portuguese coast (Fortunato et al. in press). Large-scale winds and atmospheric pressure fields, required to force these regional models, are obtained from the ERA-Interim reanalysis (Berrisford et al. 2011). Local-scale atmospheric forcings, used to force the estuarine model, are obtained by from the NCEP-NCAR Reanalysis (provided by the NOAA/OAR/ESRL PSD, http://www.esrl.noaa.gov/) or by the BINGO database developed in WP2.



Figure 91 Forcing of SCHISM with results from regional scale models (adapted from Fortunato et al. (in press)



# 7.1.2.2. Data

#### Bathymetry and topography

The generation of a digital terrain model was a very sensitive task, in particular because the extreme water levels in an estuary and the extent of an inundation largely depend on both the topography and the bathymetry. The lack of a comprehensive, detailed and updated dataset led to the use of multiple sources of data. These sources are briefly described below. The digital terrain model interpolated onto the finite element grid described above (Figure 88) is shown in Figure 92.

The only comprehensive bathymetry of the estuary dates from 1964/67. This dataset was used as a baseline bathymetry and updated wherever possible. We realize that the baseline bathymetry that we use, from 1964/67, is old and can be outdated. Therefore, this bathymetry constitutes a potential source of model errors. However, we don't believe this is a major problem for the following reasons:

1. In large areas of the estuary, in particular in the navigation channels, the baseline bathymetry has been replaced by recent measurements. The areas where we still use the 1960's data are mostly intertidal flats, that are not subject to dredging.

2. Sedimentation rates have been estimated through both chart comparisons and isotopic analyses of vertical cores (see Guerreiro et al., 2015 for a review). Results indicate sedimentation rates of about 0.3 mm/year in the intertidal areas, and between 0.7 and 2.2 cm/year close to the margins. So, the sedimentation of the intertidal areas in the past 50 years should be of the order of 15 cm. This value is of the same order of magnitude of the accuracy of the bathymetric measurements themselves.

3. The 1960's data constitute, to the best of our knowledge, the only existing data set in many areas. In the past several decades, there have been probably over a hundred scientific papers, Ph.D. theses and engineering studies applying hydrodynamic models to the Tagus estuary. All of them used the same baseline bathymetry, for lack of an alternative.

4. For the past two decades, we have applied many hydrodynamic models of the Tagus estuary (e.g., Fortunato et al., 1997, 1999, 2017a,b, Guerreiro et al., 2015, Rodrigues and Fortunato, 2017), and validated them with tide gauge data from 1972 (again, the only existing dataset covering the whole estuary). Over the years we have used increasingly sophisticated models, improved the grid resolution and forced the model with more accurate boundary conditions. Simultaneously, we have updated the bathymetry wherever possible, in particular in the main channels and some bays. Since the models are systematically validated with data from 1972, updating the bathymetry could actually increase the model errors, by increasing the discrepancy between bathymetry used in the model and the one that corresponded to the tide gauge data. Still, the inverse has always occurred, with each model application displaying smaller errors than its predecessor. We conclude that the changes in bathymetry do not constitute a significant source of errors, or at least they have not proved to be a dominant source of errors until now.

In summary, the use of old bathymetric data in some areas of the estuary is inevitable and not a major source of concern.

In most of the navigated areas (channels and docks) and in the tidal inlet, recent bathymetries were made available by the Lisbon Harbor authority. Topographic data along the margins of the estuary and LIDAR data along the coastal margins, both from 2011, were provided by the Agência Portuguesa do Ambiente and the Direção Geral do Território, respectively. Upstream of Vila Franca, only cross-channel survey data were available. Since the distance between transects (2.5 km) prevents the direct interpolation of the bathymetry, the mean cross-sectional area was evaluated and a linear regression was determined along the axis of the estuary. This regression was then used to interpolate the depths along the riverine part of the domain. Similarly, cross-sectional data are available with a spacing of 200 to 1500 m along the upper Sorraia River.



Given the narrow width of this part of the river it was represented with a rectangular cross-section and the bathymetry was interpolated using available cross-sections. The depth along the Risco River was estimated based of the few data points available.



Figure 92 Model bathymetry and station names: global view (top) and detail of the inundation-prone areas of the upper estuary (bottom). The stations correspond to the major water intake of the Lezíria (Conchoso) and the tidal stations used for valid

The topography of the margins was mostly based on a 2008 dataset provided by the Direção-Geral do Território and obtained by aerial photogrammetry. Although this dataset was provided on a 2 m resolution grid, there are some doubts on the accuracy of the dikes' crest heights. The height of the crest of the dikes protecting the Lezíria (island bounded by the Tagus estuary and river, the Sorraia and the Risco rivers) was provided by the Associação dos Beneficiários da Lezíria Grande de Vila Franca de Xira and verified in a few places by a dedicated topographic survey. The dikes covered by this dataset extend from the margins of the Sorraia River to the Tagus River. These data were measured after the partial reconstruction of the dikes that



followed their overtopping and damage during the 2010 Xynthia storm, one of the events used for model validation. The dikes represented in the model are therefore higher than the ones that existed during the Xynthia storm.



Figure 93 Determination of cross-sectional-averaged depth along the estuary axis upstream of Vila Franca

#### Water levels

Water levels measured at 13 tidal gauges installed along the estuary in 1972 were used to validate the model (Figure 92). These time series were harmonically analyzed to extract the amplitudes and phases of the tidal constituents (Fortunato et al. 1999). Time series for model validation are obtained by harmonically synthetizing the tidal elevations using the same 23 constituents included in the model.

Water levels measured at the Cascais tidal gauge (Figure 92) in 1983, 1988 and 2010 were also used to validate the 2D and 3D applications.

#### **River flow**

The Tagus river flow was established based on the data available at SNIRH (<u>http://snirh.pt</u>), namely at the Ómnias and Almourol stations. For lack of data, the river flow in the Sorraia River is taken as 5% of the river flow in the Tagus River, based on the ratios between annual averages in the two rivers.

#### Inundation during the 2010 Xynthia storm

The Xynthia (2010) storm was used to assess the ability of the model to reproduce extreme events. The data available for this validation consisted in water levels measured at the Cascais tide gauge, and the inundation limits in downtown Seixal. The former were obtained by interviewing witnesses and analyzing photographs taken at the time of the event (Freire et al. 2016).

#### Salinity and water temperature

Salinity and water temperature were validated with data from previous field campaigns in the Tagus estuary, namely INTAGUS (Neves 2010) and "Estudo Ambiental do Estuário do Tejo" (Silva et al. 1986) data. INTAGUS data included salinity and water temperature observations obtained on February 1988. Vertical profiles of salinity and water temperature were collected along three longitudinal profiles (BC: Barra-Corredor, CN: Cala do Norte, CS: Cala de Samora) during high-tide and low-tide. Monthly observations of



salinity acquired in 1983 ("Estudo Ambiental do Estuário do Tejo" data) were also used to validate the model. These data were collected along six sampling stations covering the downstream and upstream areas of the estuary (Figure 94).



Figure 94 Location of the sampling stations regarding salinity and water temperature data. The red lines represent the INTAGUS campaigns longitudinal profiles (BC: Barra-Corredor, CN: Cala do Norte, CS: Cala de Samora) and the dots represent the 1983 suvey stations

# 7.1.2.3. Calibration and validation strategies

The model applications presented herein evolved from LNEC's experience of two decades in modeling circulation in the Tagus estuary (e.g., Fortunato et al. 1997; Rodrigues et al. 2013). Based on that experience, parameters determined in previous applications were taken as a starting point for both the 2D depth-averaged model and the 3D baroclinic model.

#### 2D depth-averaged model

For the 2D model, the only calibration parameter is the friction coefficient and minor tuning from previous applications focused only in the upper estuary and the riverine part of the domain. Friction was parameterized using a Manning coefficient (Figure 95). This coefficient was selected based on a previous application (Guerreiro et al. 2015), the nature of the estuary bottom and the land cover. In particular, the Manning coefficient was taken as 0.015 m<sup>1/3</sup>/s in the upper Tagus estuary and in the Sorraia and Risco rivers, which have muddy bottoms. In contrast, values of up to 0.027 m<sup>1/3</sup>/s were used in the Tagus River, where the bottom sediments are mostly sandy. In the dry areas, the Manning coefficient was determined based on the land cover from Chen et al. (2015).





Figure 95 Manning friction coefficient (m1/3.s-1) used in the 2D model

The 2D model was validated in two steps. First, the model's ability to simulate tidal propagation was validated. A one-year simulation was performed, forced by tides from the regional model of Fortunato et al. (2016) at the ocean boundaries and by river flows of 517 m<sup>3</sup>/s and 20 m<sup>3</sup>/s at the Tagus and Sorraia boundaries, respectively. While the present bathymetry was used, the mean sea level was set to its 1972 value (2.15 m relative to Chart Datum – Fortunato et al. 1999), i.e., 11 cm below its present value. Root mean square errors were computed at 13 stations and compared with those obtained in previous applications. Both biased and unbiased root mean square errors were computed. The latter were computed by removing the mean sea level from both observed and modeled time series. While the biased errors should in theory better reflect reality, there are often doubts on the correct vertical positioning of the reference level. Hence, biased errors may partially reflect positioning errors. Root mean square errors at high tide were also evaluated, since the major concern is the ability of the model to reproduce high water levels. In a second step, a recent extreme event (the 2010 Xynthia storm) was simulated. The model was validated by comparison of measured and modeled water levels at a coastal tide gauge (Cascais) and by comparing the modeled and measured inundation extent in the mid-estuary (Seixal).

#### 3D baroclinic model

In the 3D baroclinic model the bottom stress was parameterized using a drag coefficient. Two approaches were evaluated to establish the drag coefficient: i) drag coefficient based on the previous applications of SELFE 3D in the Tagus estuary (Costa et al. 2012; Rodrigues et al. 2013, 2016); ii) and drag coefficient determined from the Manning coefficient adopted for the 2D depth-averaged model application (previous sub-section). To evaluate the influence of the drag coefficient in the model results, preliminary simulations were performed for a period of 20 days. These simulations were forced by tides from the regional model of Fortunato et al. (2016) at the ocean boundaries and by river flows of 517 m<sup>3</sup>/s and 20 m<sup>3</sup>/s at the Tagus and Sorraia boundaries, respectively. The time step was set to 30 s and the Generic Length Scale KKL turbulence closure scheme, with the Kantha and Clayson's stability function, was used. Data and model results were both harmonically analyzed and synthesized for eleven tidal constituents (Z0, MSF, O1, K1, N2, M2, S2, M4, MS4, M6, 2MS6). Root mean square errors were computed at 13 stations and compared



with those obtained in previous applications. The comparison of the model results with the 1972 water levels data showed that the second approach led to the smallest root mean square errors. The friction coefficient in the 3D baroclinic model was thus parameterized using the second approach.

The 3D model was then validated for salinity and water temperature by comparison with: i) 1988 data, which include both longitudinal and vertical profiles (Figure 94); and ii) 1983 data, which cover a broader area of the estuary. The 1988 simulation was performed for 30 days, between January 15, 1988 and February 15, 1988. Regarding the 1983 data two simulations of about 30 days were performed for two distinct seasons: January 1, 1983-January 31, 1983 and July 1, 1983-July 31, 1983. Simulated water levels were also compared with observations from the Cascais tidal gauge.

To simulate the advection of salinity and temperature two numerical schemes, upwind and TVD-Total Variation Diminishing (both available in SCHISM), were evaluated. The influence of these numerical schemes was analyzed for the 1988 simulation and, in particular, for the salinity results, since those are of major relevance for the modelling objectives in BINGO.

The 1988 simulation was also forced with two different atmospheric datasets, NCEP-NCAR Reanalysis (~250km/6h resolution) and BINGO WP2 historical data (12km/3h resolution). For the simulated period local atmospheric observations were not available. The comparison between the two datasets aimed at evaluating their influence on the salinity and water temperature results. The 1983 simulations were forced only with BINGO WP2 historical data (12km/3h resolution).

In all validation simulations the oceanic boundary was forced with 23 tidal constituents from the regional model of Fortunato et al. (2016). River flow at the Tagus river boundary was established with daily mean data measured at the Ómnias station from the SNIRH database. At the Sorraia River, the river flow was estimated as explained above. Initial conditions of salinity were set to decrease from 36 in the coastal area to 0 in the river boundaries. Salinity was set constant at all boundaries (36 at the oceanic boundary and 0 at the riverine boundaries). Initial conditions for temperature were set constant. For the 1988 simulation, water temperature at the boundaries was set constant: 15 °C at the oceanic boundary, based on daily data measured at the Cascais tidal gauge, and 13 °C at the riverine boundaries, estimated from monthly data measured at the SNIRH stations. For the 1983 simulations, since no data were available for the periods simulated, water temperature was forced with climatological time varying series. The water temperature was estimated with the Atlantic – Iberian Biscay Irish – Ocean Physics Reanalysis (http://marine.copernicus.eu/) and with SNIRH data at, respectively, the oceanic boundary and the riverine boundaries.

# 7.1.2.4. Results

#### 2D depth-averaged model

For the 1972 simulation, root mean square errors were computed at the 13 stations shown in Figure 92. Figure 96 shows both the biased and the unbiased error, computed for the whole time series and only at high tide.





Figure 96 2D Tagus model validation for the 1972 data. In the evaluation of unbiased errors, the mean sea level was removed from both observed and modeled time series. HT stands for high tide

The model's ability to reproduce extreme events was verified by simulating the Xynthia (2010) storm with an older version of the model (Fortunato et al. submitted). The RMS error at Cascais during the event was 7.5 cm. The inundation in downtown Seixal was correctly reproduced within the limits of grid resolution (Figure 97).



Figure 97 Validation of the modeled inundation of the Seixal downtown area during Xynthia. The red circles represent the inundation limits determined in the field and the blue shape is an interpretation of the inundated area based on those limits a



#### 3D baroclinic model

Similarly to the 2D model simulation, for the 1972 3D simulation, root mean square errors were computed at the 13 stations shown in Figure 92. Table 30 presents a comparison between the errors obtained for the present application and the previous 3D baroclinic application of SELFE in the Tagus estuary by Costa et al. (2012). For the 1983 and 1988 simulations the water levels errors at the Cascais tidal gauge are of the same order of magnitude as those obtained with the 1972 data.

# Table 30 3D Tagus estuary baroclinic model: Root mean square errors (RMSE) of the water levels for the 1972 data for the present application (SCHISM 3D) and the previous application by Costa et al. (2012) (SELFE 3D)

|               | RMSE (cm)<br>SCHISM3D | RMSE (cm)<br>SELFE 3D |
|---------------|-----------------------|-----------------------|
| Cascais       | 3.6                   | 1.8                   |
| P. de Arcos   | 4.6                   | 5.8                   |
| Trafaria      | 10.7                  | 9.6                   |
| Lisboa        | 7.6                   | 10.8                  |
| Pedrouços     | 5.4                   | 5.9                   |
| Cacilhas      | 3.9                   | 6.4                   |
| Seixal        | 5.7                   | 9.4                   |
| Montijo       | 8.8                   | 12.2                  |
| Cabo Ruivo    | 9.7                   | 13.5                  |
| Alcochete     | 7.3                   | 12.3                  |
| P. Sta. Iria  | 8.6                   | 17.3                  |
| Ponta da Erva | 8.8                   | 20.2                  |
| V. F. de Xira | 15.1                  | 21.5                  |

Figure 98 to Figure 103 present the comparison between the observations and the simulations regarding the 1988 field campaigns. Results include both vertical profiles (Figure 98 to Figure 101), which show the model's ability to represent the stratification during high river flow periods, and the mean salinity at each station, which provides information about the horizontal gradient along the estuary (Figure 102 and 95). The results presented in these figures were simulated with the 39 SZ levels vertical grid. However, since the vertical grid is of major relevance in the 3D simulations. The figures below show an example of how the setup of the vertical grid influences the salinity distribution.





# Figure 98 3D Tagus estuary baroclinic model: observed and simulated vertical profiles of salinity measured during high-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2) atmospheric forcings, and upwind and TVD numerical schemes

Error measures between the observations and the model results were also computed for the mean salinity and mean water temperature considering the data from all stations. Salinity root mean square errors are 2.0 for both simulations with NCEP atmospheric data and 2.1 for the simulation with BINGO WP2 atmospheric data. Regarding the water temperature, root mean square errors are 0.3 °C and 0.6 °C for the simulations with NCEP and BINGO WP2 data, respectively.

Figure 105 presents the comparison between the salinity observations in January 1983 and July 1983 and the model results. These sets of data were also used to validate the 3D baroclinic model since they have a broader coverage of the estuarine area, in particular in the upstream estuary. Salinity root mean square errors are 1.5 in January 1983 and 2.9 in July 1983. The increase of the error in July 1983 is a mainly due



to the model results at station #3.9, where the salinity is significantly underestimated. It should be noted that there is some uncertainty relative to the time of the samplings, which may contribute to the difference observed. In the station further upstream (station #1.0) the differences between the salinity observations and the model results are smaller than 1.



Figure 99 3D Tagus estuary baroclinic model: observed and simulated vertical profiles of salinity measured during low-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2) atmospheric forcings, and for upwind and TVD numerical schemes





Figure 100 3D Tagus estuary baroclinic model: observed and simulated vertical profiles of water temperature measured during high-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2) atmospheric





Figure 101 3D Tagus estuary baroclinic model: observed and simulated vertical profiles of water temperature measured during low-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2) atmospheric forcings





Figure 102 3D Tagus estuary baroclinic model: observed and simulated mean salinity along the longitudinal profiles during high-tide and low-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2)





Figure 103 3D Tagus estuary baroclinic model: observed and simulated mean water temperature along the longitudinal profiles during high-tide and low-tide (February 11-13, 1988). Model results are presented for both NCEP-NCAR Reanalysis (NCEP) and BINGO WP2 (WP2) atmospheric forcings





Figure 104 Influence of the vertical grid on the model results: vertical profiles of salinity and datamodel comparison of the mean salinity along the Cala do Norte longitudinal profile during low-tide





#### 7.1.3. Model evaluation and discussion

The elevation errors obtained in the 2D 1972 simulation are smaller those obtained in recent applications with the same dataset (Guerreiro et al. 2015; Fortunato et al. in press), showing that the model is accurate relative to other applications for the same estuary. In the upper estuary, unbiased high tide root mean square errors are smaller than 10 cm. Since the tidal range at the coast varies between 0.55 and 3.86 m (Guerreiro et al. 2015), the error represents about 4% of the tidal range, which can be considered excellent. Also, it is unlikely that the accuracy of the topography is better than 10 cm. Hence, the accuracy of the model is not a limiting factor in the evaluation of the inundation extent.

#### D3.3 - Calibrated water resources models for past conditions



The results for the Xynthia storm indicate that the model adequately reproduces extreme events. In the coastal area, the errors are only slightly larger than those obtained for tides alone. In the mid-estuary, the model proved able to reproduce marginal inundation. We expect that the accuracy will further improve when this test is run with the new version of the model, similarly to the 1972 test.

Early in the project, extensive sensitivity tests were performed for both physical and numerical parameters. These tests, described in detail in Fortunato et al. (submitted), were carried out with an older version of the model SCHISM and thus are not reproduced herein. Still, they provide further confidence in the model by showing that the extreme water levels are not very sensitive to further grid resolution and wave-related parameters (wave breaking coefficient and wave friction coefficient). The sensitivity of the model the river flow is restricted to the riverine part of the system (in the Vila Franca area and further upstream).

Regarding the 3D baroclinic model, results also show that the model represents adequately the tidal propagation. At most stations, particularly the ones located upstream, the errors obtained are lower than the ones from previous applications. The improvements obtained may result from the higher horizontal grid resolution, namely upstream, and the updates in the bathymetry (Fortunato et al., submitted), both contributing for a better propagation of the tide, and from the model itself.

Salinity and water temperature results show that the 3D model represents adequately the main patterns observed, namely the horizontal gradients and the stratification of the water column. Salinity horizontal gradients, in particular, which are of utmost relevance for the modelling objectives in BINGO, are in good agreement with the data. The model, however, tends to overestimate the vertical mixing of salt, in comparison with the observations that present a more pronounced stratification. A reduced stratification had already been observed in the previous applications (Costa et al. 2012), but significant improvements were achieved in the present application. A satisfactory agreement was also observed for temperature, with significant improvements compared to the application by Costa et al. (2012), in particular regarding the thermal stratification. Evaluation simulations showed that the setup of the vertical grid and the numerical method used to solve the advection of the tracers were particularly relevant in the simulation of the vertical patterns. The increase of the vertical grid resolution, from 20 SZ to 39 SZ levels, contributed to reduce the simulated mixing in the water column (Figure 104). The use of the TVD numerical scheme, which is less diffusive than the upwind, significantly improved the simulated stratification along the estuary, in particular during low-tide (Figure 98, Figure 99, Figure 102). However, the TVD method is more computationally demanding the upwind method, with the simulations lasting twice the time comparatively to the use of the upwind method. This computational cost prevents the use of the TVD method for very long simulations or for the simulation of a large number of scenarios. Thus, since i) both methods represent adequately the horizontal gradients and the mean salinity along the estuary, and ii) the most important events to evaluate the upstream propagation of the salt water will occur during low river flows, for future simulations the upwind method will be used for computational efficiency and the TVD will only be used when required. Regarding the influence of the atmospheric forcing on the 3D simulations, both forcings (NCEP Reanalysis and BINGO WP2) lead to similar results regarding the salinity distribution. The water temperature, however, tends to be underestimated when simulated with the BINGO WP2 dataset.

Overall, both the 2D depth-averaged and the 3D baroclinic models implemented for the Tagus estuary are suitable for the modelling objectives in BINGO and, therefore, for the simulation of the future and extreme events scenarios.



# 7.2. Groundwater

# 7.2.1. Model objectives in BINGO

Tagus river basin aquifer systems are intensively used for water supply and agriculture. Besides, a vulnerable zone is in law, thus conditioning agriculture and groundwater abstraction. The area under simulation encompasses the following aquifers: "Aluviões do Tejo", "Margem Direita", "Margem Esquerda" and "Ota - Alenquer" represented in Figure 106. Two numerical models are applied: BALSEQ\_MOD and FEFLOW. While the first one has been previously applied, the second one is being developed for the first time in the framework of BINGO project.

"Aluviões do Tejo", "Margem Direita", and "Margem Esquerda" aquifer systems contact each other and are assumed to be in connection with the Tagus river network, Tagus estuary and the ocean, which means that under climate change not only quantity but also quality issues might arise in these aquifers. The model to be used must then be able to simulate groundwater flow under confined/unconfined situations, recharge variations, and use/pollution changes and saltwater intrusion due to sea-level rise. Having this in mind, FEFLOW was the model chosen due to the fact that it is a widely used groundwater model able to simulate piezometric evolution and groundwater flow in saturated/unsaturated conditions, the influence of multilayer wells, interactions with surface water bodies, saltwater intrusion and contaminant transport.

In order to compute groundwater availability and the recharge boundary condition in the FEFLOW model, the BALSEQ\_MOD model is used. Besides applying this model to the above mentioned aquifers, this model is also applied to the "Ota-Alenquer" aquifer system.



Figure 106 Selected Tagus aquifer systems and their relation with the estuary and selected surface water basins



# 7.2.2. Model application

# 7.2.2.1. Model description

The BALSEQ\_MOD is a daily sequential water balance numerical model developed at LNEC (Oliveira 2004) that requires as input data: daily precipitation, monthly reference evapotranspiration, relative humidity and wind speed, root depths (time dependent), extent of land occupation, crop coefficients, development stage periods of plants, soil wilting point, soil field capacity, soil porosity, soil hydraulic conductivity, soil top horizon material and soil initial water content.

Recharge is estimated as the deep soil infiltration and is given by Deep infiltration = Precipitation – Surface runoff – Actual evapotranspiration – Soil water variation. The whole steps of the program run are shown in Figure 107. Surface runoff is computed using daily precipitation, material of the soil top horizon and soil water content. Actual evapotranspiration is computed by a soil water balance using top soil infiltration (given by precipitation – surface runoff), soil water content, reference evapotranspiration, relative humidity, wind speed and a combination of parameters that allow the computation of crop coefficients and the development stage of plants. Finally deep infiltration is computed based on the soil hydraulic conductivity, porosity, and soil water content.





#### Figure 107 Execution steps of the BALSEQ\_MOD model for the soil daily sequential water balance

FEFLOW is a finite elements model developed by DHI, which allows the modelling of variably saturated flow, unconfined flow, contaminant transport, density/viscosity-affected flow, multilayer wells, flexible meshing and situations of activation/deactivation of areas inside the modelling area.



FEFLOW is described in FEFLOW 7.0 Introductory Manual (DHI 2015a), FEFLOW 7.0 User Guide (DHI 2015b), Hesch (2009) and <u>http://nitrat.dk/xpdf/clausnitzer introduction to feflow.pdf</u>, <u>http://www.scisoftware.com/products/feflow\_details/ feflow\_details.html</u>. A schematics of workflow in FEFLOW is presented in Figure 108.



Figure 108 Schematics of workflow in FEFLOW

Source: http://novametrixgm.com/pdfs/brochures/FEFLOW\_Conference\_HydroGeoBuilder\_final.pdf

# 7.2.2.2. Data

For groundwater recharge assessment eleven rain gauge stations with daily data, belonging to the National Information System of Water Resources (SNIRH – http://snirh.pt/) and five meteorological stations with monthly data were used to run the BALSEQ\_MOD model. General data for these stations is shown in Table 31. The distribution of these stations can be seen in Figure 109. The observed data from these stations was used to generate modelled data for the evaluation period from 1979-03-01 to 2015-07-31 and to generate decadal predictions for the period 2015-01-01 to 2024-12-31 at each station. This data comes from the Tagus River Basin Management Plan of 2011 (Lobo Ferreira et al. 2011).



| Station Name                           | Aquifers | time period<br>(begin) | time period<br>(end) | completeness | Variables      | time<br>frequency | Latitude<br>(°) | Longitude<br>(°) | Altitude<br>(m) | Anemometer<br>height above<br>ground (m) |
|--|----------|------------------------|----------------------|--------------|----------------|-------------------|-----------------|------------------|-----------------|--|
| Vila Nogueira de<br>Azeitão (22C/02ug) | ME       | 01-10-1980             | 30-09-2009           | 97.33%       | Precipitation  | Daily             | 38.516          | -9.013           | 126             | -  |
| Moinhola (22F/03c)                     | ME       | 01-10-1980             | 30-09-2009           | 96.03%       | Precipitation  | Daily             | 38.584          | -8.616           | 41              | -  |
| Canha (21F/01ug)                       | ME       | 01-10-1980             | 30-09-2009           | 93.78%       | Precipitation  | Daily             | 38.77           | -8.627           | 52              | -  |
| Marianos (18G/03uc)                    | ME       | 01-10-1980             | 30-09-2009           | 94.06%       | Precipitation  | Daily             | 39.181          | -8.476           | 48              | -  |
| Bemposta (17I/02ug)                    | ME       | 01-10-1980             | 30-09-2009           | 94.01%       | Precipitation  | Daily             | 39.349          | -8.141           | 96              | -  |
| Tojeiras de Cima<br>(18H/04ug)         | ME       | 01-10-1980             | 30-09-2009           | 92.59%       | Precipitation  | Daily             | 39.279          | -8.236           | 153             | -  |
| Barragem de<br>Montargil (19H/02c)     | ME       | 01-10-1980             | 30-09-2009           | 92.72%       | Precipitation  | Daily             | 39.051          | -8.173           | 95              | -  |
| Barragem de Magos<br>(20E/01c)         | AT       | 01-10-1979             | 30-09-2009           | 95.47%       | Precipitation  | Daily             | 38.99           | -8.694           | 43              | -  |
| Santarém (ESA)<br>(18E/01c)            | MD       | 01-10-1979             | 30-09-2009           | 77.59%       | Precipitation  | Daily             | 39.252          | -8.702           | 61              | -  |
| Pernes (17F/01ug)                      | MD       | 01-10-1979             | 30-09-2009           | 99.64%       | Precipitation  | Daily             | 39.391          | -8.663           | 81              | -  |
| Ota (19D/02)                           | OA       | 01-10-1980             | 30-09-2002           | 100.0 %      | Precipitation  | Daily             | 39.111          | -8.989           | 39              | -  |
| Santarém (18E/01)                      | MD/ME    | 10-1959                | 09-1988              | 100.0 %      | Meteorological | Monthly           | 39.252          | -8.702           | 61              | 6  |
| Alvega (171/01)                        | ME       | 10-1959                | 09-1988              | 100.0 %      | Meteorological | Monthly           | 39.467          | -8.05            | 51              | 4  |
| Lisboa/Tapada da<br>Ajuda (21B/07)     | ME       | 10-1959                | 09-1988              | 100.0 %      | Meteorological | Monthly           | 38.7            | -9.183           | 37              | 6  |
| Salvaterra de Magos<br>(19E/02)        | ME/AT    | 10-1959                | 09-1988              | 100.0 %      | Meteorological | Monthly           | 39.033          | -8.733           | 5               | 4  |
| Dois Portos (19B/02)                   | ÓA       | 10-1959                | 09-1988              | 100.0 %      | Meteorological | Monthly           | 39.033          | -9.183           | 110             | 4  |

#### Table 31 Rain gauge and meteorological stations used to assess groundwater recharge

Notes:

Aquifers: AT - Aluviões do Tejo, ME - Margem Esquerda, MD - Margem Direita, OA - Ota - Alenquer.

Meteorological variables comprise: Average maximum (Tmax, °C) and minimum (Tmin, °C) temperature, Average relative humidity (RHmed, %), Wind speed (km/h), Insolation (hour/month), Penman-Monteith computed reference evapotranspiration (ETo, mm/month)



Figure 109 Rain gauge and meteorological stations used for groundwater recharge assessment



Required parameters to run BALSEQ\_MOD were derived from the soil map (Figure 110, data used in Lobo Ferreira et al. 2011) and the Corine land cover map 2006 (Figure 111, Caetano et al. 2009).



Figure 110 Soil map used to characterise soil dependent parameters in BALSEQ\_MOD



Figure 111 Corine Land Cover map used to characterise land cover dependent parameters in BALSEQ\_MOD



BALSEQ\_MOD outputs daily values of the water balance processes that it models. These can be recomputed for any desired time period.

FEFLOW uses the following input data:

- Digital elevation model and depths of the geological layers, in shapefile. Depths of geological layers are assumed constant throughout the simulation time. The MDT is updated to 2011. The source of these data is the Tagus River Basin Management Plan of 2011, geological maps of Portugal (from *Instituto Geológico e Mineiro*), and borehole data existing in *Agência Portuguesa do Ambiente* (APA) archives that were loaded by LNEC to well's databases since several years. See Figure 104 for the geometry of the model.
- Porosities, Permeabilities, Specific Storage, in shapefile. Assumed constant throughout the simulation time. The source of these data is the Tagus River Basin Management Plan of 2011 (Lobo Ferreira et al. 2011) and bibliography (e.g. http://www.aqtesolv.com/aquifer-tests/aquifer\_properties.htm).
- Recharge values, in shapefile. The annual values resulting from the application of the BALSEQ\_MOD model previously described is one of the boundary conditions of FEFLOW.
- Piezometry (for calibration), in shapefile. Annual to monthly data, depending of the performance history of the monitoring network. The data come from the Tagus River Basin Management Plan of 2011, SNIRH, wells licensed by CCDR-LVT and from monitoring under the study for the location of the new Lisbon's airport (Lobo Ferreira et al. 2008).
- River network, used to define head boundaries.

As output FEFLOW provides data on:

- Piezometric levels, at chosen years during and at the end of the simulation period.
- Chloride concentrations, at chosen years during and at the end of the simulation period.





Figure 112 Geometry of the Tagus modelled aquifer systems

# 7.2.2.3. Calibration and validation strategies

The BALSEQ\_MOD model is not calibrated. It uses the best estimates of the required parameters and its results are used as they are. There are no control data, for instance soil moisture determinations that could be used to confront the obtained results. The only way available to validate its results is precisely by calibrating the flow model. During this process it may be required to change the recharge values in order to obtain better results. If this happens, if wished, it is possible to come back to the BALSEQ\_MOD model and correct its input parameters.

FEFLOW calibration is, at this stage of development, being done manually, by comparing the results with the piezometry monitoring data. Later on PEST (under the FEPEST tool) shall be used. Part of the network of observation points being used for calibration is presented in Figure 113 and some preliminary results in Figure 114.






Figure 113 Set of observation points used in the calibration of the FEFLOW model



Figure 114 Behavior of heads in the observation points (steady state)



The temporal data used for FEFLOW depends on piezometry time series. These are available from 1980 to 2010. As the model is being used at a regional scale, data is agglutinated in the period 1980 to 2000 for calibration and then from 2001 to 2010 for validation.

Still a great amount of work is to me made concerning aquifer exploitation for these periods.

At the moment it is expected that the model will be considered calibrated when the absolute deviation of the modelled piezometry is not more than 10 % beyond the range of observed values. For instance if the piezometry values oscillate between 30 m height and 40 m height, a modelled piezometry between 29 m height and 41 m height would be acceptable (the range is 40 - 30 = 10 m, and 10 % of this is 1 m). Besides, it may be acceptable that up to 20 % of the calibration wells are outside of this range.

Besides these calibration rules, another rule for the acceptance of the results of the FEFLOW run is that the error balance is 1% of the whole water budget or lower.

## 7.2.2.4. **Results**

Results of the application of BALSEQ\_MOD to the four aquifer systems, using measured meteorological data and parameters derived from soil and land cover (year 2006) maps as described in previous section can be seen in Figure 115.

As mentioned in the beginning of the groundwater section (7.2.1) the FEFLOW model did not have any antecedents, so the model is still in the development / calibration process, and no results are available. Uncalibrated results for the piezometric heads of the top aquifer are presented in Figure 116 for layer 1 of the model (the most superficial layer).



Figure 115 Average groundwater recharge computed using real meteorological data (periods ranging between 01-10-1979 and 30-09-2009, depending on the aquifer system – see Table 31)





Figure 116 Hydraulic heads for Layer 1 of the FEFLOW model for Tagus aquifers

# 7.2.3. Model evaluation and discussion

BALSEQ\_MOD model has been applied using existing meteorological data for the soil conditions and assuming the land cover as in 2006 was kept constant for the whole period of run. Efforts continue to develop and calibrate the FEFLOW model. For now it is not still possible to draw conclusions concerning groundwater modelling.

# 7.3. Floods (Trancão basin)

# 7.3.1. Model objectives in BINGO

The Trancão basin is located at the northern limits of Lisbon, covering an area of 292 km<sup>2</sup>. The Trancão River and its tributaries are prone to rapid floods due to intense rainfalls, strong slopes of the river basin headwaters and the existence of heavily urbanized areas. As referred in a previous report (Deliverable 3.1, Alphen et al., 2016), past flood events have led to extensive inundations which caused human casualties and severe social and economic impacts in most of the basin, particularly in Póvoa river and Loures lowlands. The importance of the flood risks in the Trancão river basin was recognized in the context of the EU Floods Directive (Directive 2007/60/EC) and the flood risk management plan was developed. Despite these efforts, floods in the Trancão river basin need to be estimated considering land-use changes in a context of climate change to support appropriate flood management measures.

The rainfall-runoff model HEC-HMS (Hydrologic Engineering Center, Hydrologic Modeling System) developed by the US Army Corps of Engineers was chosen for flood hydrographs estimations on Trancão river basin. The HEC-HMS is a validated tool for simulating the runoff response to a precipitation event and is suitable for general geographical conditions. It is widely used in research and application practice due to



its free availability. It was used in the Tagus River Basin Management Plan (PGBHT 2012) for flood simulations.

# 7.3.2. Model application

## 7.3.2.1. Model description

The Hydrologic Modeling System (HEC-HMS) was designed to simulate the rainfall-runoff processes in a wide variety of basin types considering the soil and land use characteristics. Various atmospheric and land surface components of the hydrologic cycle are included in HEC-HMS, namely, precipitation, evapotranspiration, snowmelt, solar radiation, canopy interception, surface depression storage, infiltration, surface runoff, and baseflow. The HEC-HMS model also include hydraulic components such as: source inflows, channel routing, channel losses, diversion structures, and reservoirs. The referred components allow to simulate the processes of runoff such as loss, direct runoff, channel routing and baseflow, based on the selection of different models available in HEC-HMS (HEC 2000).

HEC-HMS model setup consists of four main components: the basin model, the meteorological model, control specifications, and input data (time series, paired data, and gridded data). The basin model represents the hydrologic elements (subbasins, reaches, reservoirs, etc.) and their connectivity. In the basin model it is also defined the method for the evaluation of infiltration losses, the method to transform precipitation into runoff hydrographs, and the routing model of the hydrographs through the river reaches. The meteorological model defines the spatially and time distribution of precipitation over the river basin.

## 7.3.2.2. Data

### Digital elevation model

A Digital elevation model (DEM) with a horizontal resolution of 25 m × 25 m was generated from data provided by The Portuguese Environment Agency (APA – Agência Portuguesa do Ambiente) with ArcGis software. The DEM (Figure 117) was used to delineate Trancão river basin and sub-basins and to obtain its physical characteristics through ArcGis tools.





Figure 117 Digital elevation model of Trancão river basin

#### Soil, Geology and Land-use

The Tagus River Basin Management Plan (PGBHT 2012) database served as basis of soil and geology maps for the Trancão river basin. The maps of land cover and uses in Trancão basin were obtained from the database CORINE Land Cover (CLC) for the available dates: 1990, 2000, 2006 and 2012.

### Precipitation

Recorded daily precipitation values were extracted from SNIRH database (<u>http://snirh.pt</u>), provided by The Portuguese Environment Agency (APA). Hyetographs of selected rainfall events were also made available by APA for S. Julião do Tojal meteorological station, located in Trancão basin.

#### Discharges

Maximum flood discharges observed on the three hydrometric stations (Ponte Canas, Ponte Pinhal and Ponte Resinga) located in Trancão river basin were extracted from SNIRH database. Available flood hydrographs were also provided by The Portuguese Environment Agency (APA). The data available in the mentioned hydrometric stations are scarce, comprising only the period from 1977 to 1991. Ponte Canas and Ponte Resinga stations were extinct in 1991 and 1990, respectively.

# 7.3.2.3. Calibration and validation strategies

The HEC-HMS model was applied to Trancão river basin. The defined basin model, namely its subbasins and hydrologic connectivity, is presented in Figure 118. The Trancão basin was divided into several subbasins to increase the modelling performance and to use HEC-HMS as semi- distributed model.





Figure 118 HEC-HMS basin model for the Trancão basin

The selection between available models that represents the components of the runoff process was made based on previous experience and current practice in Portugal. The SCS curve number method was selected for the simulation of infiltration losses. For the transformation of excess precipitation into runoff the SCS unit hydrograph method was selected which requires the lag time input values. The routing model was the Muskingum-Cunge method, which requires the definition of the river cross-section geometry, river slope and Manning's roughness coefficient. The above mentioned methods require the input of the values of the parameters to obtain simulated runoff hydrographs. These values were estimated based on stream and basin characteristics, although some of the parameters cannot be precisely estimated (for instance, the SCS curve number).

Calibration of HEC-HMS model was performed using historical flood events with observed hyetographs and hydrographs. Due to the reduced period of discharges measurements available in the Trancão river basin, only a few events were used in the calibration of the model, namely, floods of December 1978, January 1985 and February 1985. For the model validation the flood event of December 1981 was considered.

The calibration process was made in two steps. After systematic search of the best fit lag time values, an automatic trial and error method available in HEC-HMS was applied to calibrate the SCS number values, that have been initially been set based on Corine Land Cover map (1990) and soils characteristics. To minimize a specific objective function, the peak weighted root mean square error was used.

# 7.3.2.4. Results

The HEC-HMS model was validated using the flood event of December 1981, which is one of the largest recorded floods in Trancão river basin with available hydrographs. In Figure 119, the observed and simulated hydrographs are presented for Ponte Pinhal and Ponte Canas hydrometric stations. The results show relatively good agreement between observed and simulated flood hydrographs. The HEC-HMS is able to simulate the peak discharge values and the general hydrograph configuration, although a slight delay in time is observed on the peak discharge occurrence.







Figure 119 Comparison between observed (red line) and simulated (blue line) discharges for the flood event of December 1981 in Trancão basin

# 7.3.3. Model evaluation and discussion

The HEC HMS hydrological model has been calibrated for the Trancão river basin using three flood events and validated with one flood event. Although the model is able to reproduce the flood event of December 1981, the limited available hydrographs used in the model calibration is the principal source of uncertainty in the application of the HEC-HMS model to the Trancão river basin. Indeed, the considerable heterogeneity on the basin land use and the rapid geomorphic response to rainfall events, make it advisable that other precipitation events be included in the model calibration. This effort will be pursued in order to extend the database used in the calibration of the model.

# 7.4. Water Resources (Zêzere and Sorraia Basins)

### 7.4.1. Model objectives in BINGO

The two Tagus sub-basins selected as case-studies for studying the climate change impacts on water resources are the Zêzere River basin on the right-hand side margin and the Sorraia River basin on the left-hand side margin of the Tagus River as described in BINGO Deliverable 3.1 (Alphen et al., 2016). Both basins have high capacity reservoirs whose management operation changed the natural flow regimes through regulation and reduction in flows. In the Zêzere basin, the main water use is for power generation,

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which is not consumptive, whereas in the Sorraia basin the two main reservoirs in the headwaters have a demand-driven irrigation water use which is highly consumptive. Cumulatively with the power generation objective the last reservoir in the Zêzere cascade (Castelo do Bode) has also the diversion of water to supply Lisbon's metropolitan area, but the total annual amounts diverted are not significant when considering the Zêzere natural flow (as described in BINGO Deliverable 3.1).

In both case-studies the objective of task 7.4 will be to detect vulnerabilities in the current water uses due to water scarcity induced by new rainfall regimes. In this framework the simulation of the inflows to the headwaters' reservoirs is mandatory. Also significant will be to assess the potential Zêzere basin contribution to the Tagus main stream in drought climatic settings, when the water coming from the upstream country (Spain) will accordingly be reduce. Although the water convention between Portugal and Spain (the Albufeira Convention) specify water minimums for the main shared rivers (including the Tagus River) the yielding of the agreed amounts may be exempted when exceptional dry periods occur (Santafé-Martínez 2003).

This constrain is important for the simulation of both water quantity minima in the water intake (at Valada) downstream the Zêzere confluence as well as its quality characteristics (section 7.4.5).

As for the study of flow maxima extremes, that topic is tackled in a different spatial zone (the Trancão River sub-basin – see section 7.4.3).

# 7.4.2. Model application

## 7.4.2.1. Model description

The flow simulation in each case-study basin is performed with the MIKE BASIN software that incorporates DHI's Nedbør-Afrstrømnings-Model (NAM) as a lumped conceptual model. As NAM is a lumped model, assuming each sub-catchment as one unit, the parameters and variables considered represent therefore average values for the entire sub-catchments.

Below is given a description of the model based on Agrawal and Desmukh (2016).

The NAM model as a default considers 9 parameters in its automatic calibration, although it can be set for more model parameters, while accounting for the surface zone storage, root zone storage and ground water storage. During calibration the model parameters are adjusted in such a way as to get the best possible relationship between the simulated and the observed discharges.

A minimum of 3 years data is required by the model to get the more reliable model results. The 9 default parameters of NAM are as follows:

- Maximum water content in surface Storage (U<sub>max</sub>): This parameter represents the cumulative water content of interception storage (i.e. on vegetation and depression storage), and storage in the upper layers of soil.
- Maximum water content in root zone storage (L<sub>max</sub>): This parameter represents the maximum moisture content in soil in the root zone, available for transpiration by vegetation.
- 3. Overland flow runoff coefficient (CQOF): This parameter determines the division of excess rainfall between overland flow and infiltration.
- 4. **Time constant for routing interflow (CKIF):** This parameter determines the amount of interflow which decreases with larger time constants.



- 5. Time constants for routing overland flow (CK<sub>1</sub>, CK<sub>2</sub>): These two parameters determine the shape of hydrograph peaks. The routing takes place through the two linear reservoirs which are serially connected with different time constants in hours. High sharp peaks are linked with shorter time durations and vice-versa.
- 6. Root zone threshold value for overland flow (TOF): This parameter determines the relative value of the moisture content in root zone above which the overland flow is generated. The effect of this value can be mainly seen in rainy season where an increase in parameter value will delay the start of runoff.
- 7. Root zone threshold value for interflow (TIF): This parameter determines the relative value of the moisture content in root zone above which interflow is generated.
- 8. Time constant for routing base flow (CKBF): This parameter can be determined from the hydrograph recession in dry periods. In rare cases, the shape of the measured recession changes to a slower recession after some time. To simulate this, a second groundwater reservoir may be required.
  - Root zone threshold value for groundwater recharge (TG): This parameter determines the relative value of moisture content in root zone above which ground water recharge is generated. By increasing the value of this parameter, recharge to the ground water storage reduces.



Figure 120 Structure of NAM Model



The default ranges for these 9 parameters are given below:

| Parameter                         | Lower bound | Upper bound | Units |
|-----------------------------------|-------------|-------------|-------|
| U <sub>max</sub>                  | 10          | 20          | mm    |
| L                                 | 100         | 300         | mm    |
| C <sub>QOF</sub>                  | 0.1         | 1.0         | -     |
| C <sub>KIF</sub>                  | 200         | 1000        | hours |
| CK <sub>1</sub> , CK <sub>2</sub> | 10          | 50          | hours |
| T <sub>OF</sub>                   | 0           | 0.99        | -     |
| T <sub>IF</sub>                   | 0           | 0.99        | -     |
| C                                 | 1000        | 4000        | hours |
| T <sub>G</sub>                    | 0           | 0.99        | -     |

#### Table 32 Parameters Range Values of NAM Model

#### Basic Modeling Components

The elements of NAM model representing the various phases of hydrological cycle are represented mathematically by the following functions:

Evaporation: The evaporation E<sub>a</sub> demands are initially met at the potential rate from the surface capacity. On the off chance that the moisture content U in the surface capacity is less than these requirements (U < Ep), the remaining part is thought to be pulled back by root movement from the lower zone capacity at an actual rate E<sub>a</sub>. E<sub>a</sub> is corresponding to the potential evapotranspiration and fluctuates directly with the relative soil dampness content as.

$$Ea = \begin{cases} L & U \ge E \\ U + \frac{L}{L_{max}}(E - U) & otherwise \end{cases}$$

2. Net rainfall and infiltration: Net rainfall P<sub>N</sub> is not clearly defined by the MIKE NAM but implies the mathematical relation expressed below

$$P_{N} = \max(0, P - E_{a} - QIF - (U_{max} - U)))$$

This leaves infiltration to the lower zone capacity defined as

3. **Overland flow:** At the point when the surface storage spills, i.e. at the point when  $U > U_{max}$ , the water surplus  $P_n$  becomes available to overland stream and also to infiltration. QOF indicates the part of  $P_n$  that gets allocated to overland flow. It is thought to be corresponding to  $P_n$  and to differ directly with the relative moisture content in the root zone,  $L/L_{max}$ , of the lower zone storage. It only happens when the saturated fraction of the lower zone exceeds the threshold value.

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$$QOF = \begin{cases} CQOF P_n \left[ \frac{L/L_{max} - TOF}{1 - TOF} \right] & L/L_{max} > TOF \\ 0 & otherwise \end{cases}$$

4. Interflow: The interflow share, QIF, is thought to be relative to U and to shift directly with the relative dampness of the lower zone storage. It occurs only when a critical saturation fraction of the lower zone exceeds the threshold value. This interflow should be subjected to restraint for take place only when there is enough availability of water to the upper zone storage to keep this interflow.

$$QIF = \begin{cases} CQIF & \left[ \frac{L/L_{max} - TIF}{1 - TIF} \right] & L/L_{max} > TIF \\ 0 & otherwise \end{cases}$$

5. Interflow and overland flow routing: The interflow is directed through two straight reservoirs in arrangement with the same time consistent CK<sub>1,2</sub>. The overland stream routing is also based on the linear reservoir idea but with a consistent variable time. To hold a linear response for near surface flows and a kinematic response for above surface flows at higher discharges, the time constants will be modified as

$$\label{eq:CK} CK = \begin{cases} CK_{12} & OF < OF_{min} \\ CK_{12} \left[ \frac{OF}{OF_{min}} \right]^{-\beta} & otherwise \end{cases}$$

Where OF is the overland flow (mm/hour) and OF<sub>min</sub> is the upper limit for linear routing (=0.4 mm/hour) and  $\beta$ =0.4.

6. **Groundwater recharge:** The quantity of infiltrating water *G* recharging the groundwater capacity depends upon the soil moisture content in the root zone capacity. It is related to the infiltration entering the lower zone. It happens when the saturated fraction exceeds the threshold value.

$$G = \begin{cases} I \left[ \frac{L/L_{max} - TG}{1 - TG} \right] & L/L_{max} > TG \\ 0 & otherwise \end{cases}$$

7. **Groundwater storage and base flow:** the groundwater capacity allows the infiltrated water to become base flow in two ways. The simple one is that it uses a linear reservoir concept such that base flow is

$$q_g = \begin{cases} CKBF \, S_g S_g > 0 \\ 0 & otherwise \end{cases}$$

The second one makes use of the concept of a shallow reservoir typical of depression catchments with little topographic variations and have possibility for water logging. In this case base flow is directly related to water table depth above the maximum drawdown of groundwater zone and is given by

$$q_{g} = \begin{cases} CK_{BF} S_{Y} (D_{g}^{max} - D_{g}) D_{g}^{max} \ge D_{g} \\ otherwise \end{cases}$$

Where,

5 = water in groundwater storage above zero reference (negative values are possible)



 $D_{g}$ 

= depth of water table below zero datum

 $D_g^{max}$  = depth of water table attaining a maximum value

8. **Capillary flux:** Water can transfer upwards from the ground water to the lower zone storage by capillary action. The capillary flux, C, is related to the square root of the shortage in the lower zone and inversely related to the drawdown in the groundwater reservoir.

$$C = \left(1 - \frac{L}{L_{max}}\right) \left(\frac{D_g}{D_g^1}\right)^{-\alpha}$$

If C has units of mm/day then the parameter  $\boldsymbol{\alpha}$  is given by

$$\alpha = 1.50 + 0.45 D_g^1$$

Where  $D_{\sigma}^{1}$  is the depth of the groundwater table at which the capillary flux is 1 mm/day when L=0.

### Soil Moisture Accounting 4-parameter model

As depicted in Figure 120 the NAM model has a soil-moisture-accounting based algorithm with a detailed description of the soil water content profile and the interflow process. This account for 9 parameters in the calibration process that, once optimized, can lead to aggregated magnitudes of field capacity that do not fully reproduce the lithology and geology of the soils in the entire basin, especially for large basin areas.

In order to generate upper and lower bounds for the MIKE BASIN's set-up file that are site-specific from each case-study while reproducing the overall geological framework, a second and simpler soil-moistureaccounting model was used to bind these parameters bounds. The model is based on a simple 4-paramater Thornthwaite-Mather algorithm modified to be applicable to heterogeneous areas like volcanic geological environments (Rodrigues, 1994).

The parameters coming out from this preprocessing model whose magnitudes were bounds for input as their equivalents in the NAM model were  $L_{max}$ ,  $C_{QOF}$ , and  $C_{KBF}$ . These variables can be locked during the NAM auto calibration process leaving the algorithm to perform over the remaining 6 variables.

# 7.4.2.2. Data

### The Portuguese National Water Resources Information System - SNIRH

NAM model requires various input data which includes the parameters to define the catchment, model parameters, initial conditions, hydro-meteorological data and stream flow data. The basic meteorological data requirements are precipitation, potential evapotranspiration and temperature. Using these, the model produces the output in the form of catchment runoff, subsurface flow contributions to the channel, and information about other elements of the land phase of the hydrological cycle, such as soil moisture content and groundwater recharge. All the input and output for the model is in time series format.

All the raw data used in the sub-sequential studies was retrieved from the Portuguese National Water Resources Information System – SNIRH (<u>http://snirh.pt</u>).

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As the Portuguese water data collecting agency suffered periodic structural changes and budgetary constraints during the time period subject to the calibration and validation of the models (1976 to 2016), particularly in the early 90's and from 2008 onwards, impacts in both data availability and reliability were observed in the data processing activity. Special care was thus directed to data validation with both homogeneity tests and reconstructing methodologies, both statistical (e.g. correlation and spatial interpolation) and hydraulic (for the rating curves from flow gages).

All the rainfall and atmometer evaporation data were weighted for each sub-basin in order to match the input requirements to the flow model.

The flow data analysis detected frequent extrapolations of rating curves, with huge implications in maxima determination. This fact was further complicated by the long periods of validity of the rating curves without updates from field surveys, a fact particularly sensitive in moving bed sections.

In view of this situation it was found necessary to resort to flow data indirectly determined from dams' exploitation.

In the Zêzere case study (with its sampling points depicted in Figure 121) the flow data ended up coming from the hydropower company EDP. Although there are still some validation issues (the flow through the turbines is derived indirectly from power production; the spillway discharges determinations are biased; there is lack of sensibility for low flow values), these data proved more reliable than the flow gauge data and displayed less gaps in the time-series. Flow gauge data were nevertheless used to calibrate the inflows from the tributary of the Zêzere close to its mouth, the Nabão River (Figure 121).

In the Sorraia case study (with its sampling points depicted in Figure 122) the inference of the natural inflow from upstream was even more problematic since it had to be reconstructed not only from the small-head power production – characteristic from these smaller dams – but also from the several consumptive uses, such as irrigation, agro-industry and municipal consumptions, also characteristic from these agricultural-based operation management dams. In these cases the lack of sensibility in the low flows data is even greater, which becomes more problematic in face of the endemic dryness of the southern Tagus margin lands.

Despite all these setbacks it was possible to reconstruct flow data for both case-studies enabling the calibration process described below.





Figure 121 Zêzere river basin with major sub-basins for flow simulation



Figure 122 Sorraia river basin with major sub-basins for flow simulation



# 7.4.2.3. Calibration and validation strategies

### Soil Moisture Accounting 4-parameter model

As mentioned earlier a simpler 4-parameter soil-moisture-accounting model was used to make a first screening on the bounds of three NAM soil and groundwater recession parameters in order to use them subsequently in the NAM model.

The full modelling period from the hydrologic year 1976/77 to 2008/09 was separated in 3 periods of 11 years each. The objective was to use different calibrations for the predominantly wet period of 1987/88-1997/98 and for the predominantly dry period of 1998/99-2008/09 and then select the best combination that would give fair recession curves in dry years while preserving peak adjustments. The first 11-year period (1976/77-1986-87) was used as an additional weighting loop although its base hydrometeorological information is, in general terms, less reliable.

Three sub basins in the Zêzere system and two sub-basins in the Sorraia system were subject to the screening process.

#### Zêzere System

The Zêzere river basin is characterized by steep slopes, forested areas prone to frequent wild fires and top soils with little water holding capacity. The drainage basin's configuration denotes in its development from the headwater to the mouth (Figure 121) an elongated shape, allowing for spiky flow peaks in winter and much reduced low flows in summer. Downstream of Cabril dam the river bends almost 90° assuming a N-S path. The Nabão sub-basin displays much of the general characteristics of the Zêzere main basin while being less steep and with less mountain highs.

The sub-basins whose monthly flows were modeled in this system were (Figure 121):

- Nabão at Zêzere confluence;
- Zêzere at Cabril dam;
- Zêzere at Castelo do Bode dam.

The result of the calibration is displayed for each sub-basin and for each 11-yr period inFigure 123 to Figure 125Figure 123.





Figure 123 Calibration results for the Nabão sub-basin





Figure 124 Calibration results for the Zêzere at Cabril sub-basin





Figure 125 Calibration results for the Zêzere at Castelo do Bode sub-basin

The calibrated values of the correspondent  $L_{max}$  were very low, in tune with the lithological information. The correspondent  $C_{QOF}$  values were higher than 50 %, meaning that the higher percentage of net rainfall goes to the river as overland flow, again in accordance with expected behavior. As for the correspondent  $C_{KBF}$  parameter the recessions found are just mildly abrupt.

#### Sorraia System

The Sorraia headwaters are hilly shaped and not steep with bare soils.



As can be seen in Figure 122Figure 120, two sub-basins converge to the artificial lake created by the Maranhão Dam (in the Raia River), while the Sor is dammed in Montargil. Is from the joining of these two rivers that the Sorraia is born.

Due to the uniformity of climate, soils and vegetation cover only the sub-basin draining to Montargil was subject to modeling and calibration.

Q (mm) Escoamento mensal no Rio Sor (1976/77 - 1986/87) Qcak Qob 250.0 200.0 150.0 100.0 50.0 0.0 tempo (meses) Q (mm) Escoamento mensal no Rio Sor (1987/88 - 1997/98) Qobs 250.0 200.0 150.0 100.0 50.0 0.0 tempo (meses Q (mm) Escoamento mensal no Rio Sor (1998/99 - 2008/09) Qcalc Qobs 250.0 200.0 150.0 100.0 50.0 0.0 tempo (meses)

Figure 126 depicts the final calibration for the basin flows.

Figure 126 Calibration results for the Zêzere at Castelo do Bode sub-basin

The calibration results were very similar to the Zêzere sub-basins in what relates to the soil water holding capacity (very thin top soils) but the percentage of water reaching the river as overland flow is equivalent to the percentage of water infiltrated in the soil. The recessions are milder although the data suggests long zero value flows. The poor quality of the flow data for this particular area turn the calibration exercise in a difficult problem.



### MIKE BASIN NAM model

### Rainfall and evaporation

The rainfall regime of the Nabão river basin was characterized based on the identification of the influential meteorological stations in the area of the basin. Recorded monthly rainfall data of eight rain-gauge stations (Ansião, Cernache do Bonjardim, Ferreira do Zezere, Rego da Murta, Caxarias, Tomar, Pedrógão and Sardoal) for the period of 35 years, i.e. from October 1976 to April 2010, was used for the modeling. The real precipitation of the area was computed from point precipitation by using Thiessen Polygon Method. Rainfall values were extracted from SNIRH database (<u>http://snirh.pt</u>), provided by The Portuguese Environment Agency (APA).

The evaporation data from Cernache do Bonjardim climatological station for the same time period was selected.

### Runoff

Monthly discharges observed, for the time period between 1976 and 2010, on the hydrometric station of Tomar, located practically on the mouth of the Nabão River was also extracted from SNIRH database, to calibrate the model.

### Digital elevation model

A Digital elevation model (DEM) of Nabão river basin with a horizontal resolution of 25 m × 25 m was generated from data provided by The Portuguese Environment Agency (APA – Agência Portuguesa do Ambiente) with ArcGis software (Figure 127).



Figure 127 Digital elevation model of Nabão river basin

An auto-calibration of the model was done to estimate the final NAM parameters, comparing observed and simulated monthly runoff. This auto-calibration is based optimizing two objective functions: (a) minimizing the water balance error (% WBL) to achieve agreement between the average simulated and observed runoff;



and (b) minimizing the overall Root Mean Square Error (RMSE) for the entire flow spectrum to achieve overall agreement of the shape of the hydrographs.

The MIKEBASIN model was set-up with all input information and calibrated for the period of 34 years from 1976 to 2010 to obtain the set of best fit model parameters which could simulate runoff with high degree of agreement with observed runoff. The set of model parameters were obtained during the model calibration and were found within their specified range as shown in Figure 128.

| i In     | clude autocalib  | ration              | iter j snow meit | ingation    | initial conditions | nutocalibration | Seasonal Variation                     | nine senes   |      |   |
|----------|------------------|---------------------|------------------|-------------|--------------------|-----------------|--|--------------|------|---|
| Cali     | bration paramete | ers                 |                  |             |                    |                 | Objective function                     |              |      |   |
|          | Parameter        | Fit                 | Initial Value    | Lower Bound | d Upper Bour       | nd              | <ul> <li>Overall water bala</li> </ul> | nce          |      |   |
| •        | Umax             | <ul><li>✓</li></ul> | 16,22957017      | 0,1         | 20                 |                 | Overall root mean                      | square error |      |   |
|          | Lmax             |                     | 100              | 10          | 300                |                 | Peak flow RMSE                         | Peak flow >  | 60   |   |
|          | CQOF             | ~                   | 0,954053672      | 0,1         | 1                  |                 | Low flow BMSE                          | Low flow <   | 20   |   |
|          | CKIF             | ✓                   | 308,6455088      | 200         | 1000               |                 | • con non ninoz                        | LOW NOW S    | 20   |   |
|          | CK1,2            | ✓                   | 28,40670028      | 0           | 50                 |                 | Maximun number of ev                   | aluation     | 2000 |   |
|          | CK2              | ~                   | 10               | 10          | 50                 |                 |  |              | 0    |   |
|          | TOF              | ~                   | 0,985997176      | 0           | 0,99               |                 | Initial number of days e               | xcluded      | U    |   |
|          | TIF              | ~                   | 0,982657717      | 0           | 0,99               |                 |  |              |      |   |
|          | TG               | ~                   | 0,142397954      | 0           | 0,99               |                 |  |              |      |   |
|          | CKBF             | ~                   | 2206,944017      | 1000        | 4000               |                 |  |              |      |   |
|          | CQLow            | <b>~</b>            | 0                | 0           | 100                |                 |  |              |      |   |
|          | CKLow            | -                   | 10000            | 1000        | 30000              |                 |  |              |      |   |
| _        |                  |                     |                  |             |                    |                 |  |              |      | > |
|          | <b>E</b> 11. 1   | <b>a</b>            |                  |             |                    |                 |  |              |      |   |
| <u>^</u> | Filtering:       | Show all c          | catonments       |             | ~                  |                 |  |              |      |   |
|          | Name             | Area                | Branch na        | ame C       | hainage            | RR-model        |  |              |      |   |
|          | Rio Nabã         | o 1063,             | 8                | 0           | I                  | NAM 🗸           |  |              |      |   |

Figure 128 Model parameter values of model calibration and their range

# 7.4.2.4. Results

Monthly discharges were predicted for the period of 1th September 2007 to 29th February 2008 to validate the MIKE Basin using the model parameters estimated during the calibration period.

A comparison between observed and simulated monthly runoff for the validation period is illustrated in Figure 129. This graph indicates a good match between the observed and simulated runoff. In fact, from the analysis of Figure 129, it is observed that the shapes of the observed and simulated peak and low flows are matching well for almost all the chosen time period. It can also be seen that the time of beginning and termination of observed and simulated runoff events are matching with moderate accuracy.

The comparison between observed and simulated accumulated monthly runoff volume (as shown in Figure 130) might seem far from optimal but this was the consequence of restraining the three parameter values to the magnitudes assumed by hydrogeological common knowledge that were corroborated by the 4-parameter model. By having the three main soil-moisture-accounting parameters locked during the calibration it was not possible for the auto calibration algorithm to diminish the calculated flow to attenuate the departure from the observed flows. The unlocked versions of the calibration assumed 300 mm for the soil capacity which does not hold.





Figure 129 Observed and simulated monthly runoff during model calibration



Figure 130 Observed and simulated accumulated runoff

# 7.4.3. Model evaluation and discussion

Both the 4-parameter Soil-Moisture-Accounting and the MIKE BASIN NAM rainfall runoff models performed well in describing the flow regimes from rainfall and evapotranspiration. While the former, simpler model, had a more robust performance in mimicking the flow regime, the latter has more potentialities in simulating hydrological response of the basin to rainfall for time steps lower than the month.

The NAM model performed well, in such criteria as timing (of flow initiation and peak occurrence), rate and shape of peak, and low flows magnitude. Some deviation still occurs between accumulated monthly observed and simulated runoff volumes that need to be sort out.



## 7.4.4. Bibliography

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# 8. Spain

# 8.1. Model objectives in BINGO

Badalona (Spain), which is included in the metropolitan area of Barcelona with an extension of 21.2 km<sup>2</sup> between a coastal mountain range ('Serralada de la Marina') and the Mediterranean Sea, is one of the BINGO Research sites. In this research site, a full risk assessment related to flooding problems and Combined Overflow (CSOs) impacts will be carried out for current scenarios. Moreover, the efficacy of a set of adaptation strategies to face with climate change will be evaluated for future scenarios

In D3.1 a detailed description about site characterization and the main objectives of the case studies can be found.

The 1D sewer model developed in 2012 for the Drainage Management Plan (DMP) of Badalona had become obsoleted and unable to adequately describe flooding in the urban area due to last civil works and infrastructures executed during the last years. Moreover recent advances in software and hardware developments indicate a clear possibility to use new tools and methodology to describe the hydraulic behavior of the urban drainage systems more adequately (Russo et al. 2015).

Due that one of the main objective of Badalona research site within the BINGO project is the flood risk assessment with special regards for pedestrian and vehicular circulation, the old 1D sewer model has been updated to a detailed 1D/2D model considering a 1D approach for the updated underground sewer network flow and a 2D approach for overland flow. This type of model allows us to take into account surface flows coming from upstream catchments and the interaction between the two drainage layers: streets, sidewalks (major system) and the sewer network (minor system). It is important to remark the hydraulic characterization of inlet systems among the sewer network that permit to reproduce realistic interaction between major (streets, sidewalks, parks, squares, etc.) and minor systems (sewer network) (concept of dual drainage).

The full availability of valuable information for the Badalona research site like detailed data about sewer elements (mainly from the DMP and Badalona city council), rainfall series data provided by 3 rain gauges, water depth series provided by 14 water level sensors encouraged the setup and calibration of a detailed coupled 1D/2D model in the framework of BINGO project.

1D sewer flow was simulated by full 1D Saint Venant equations, while surface flow concerning overland flow on streets, sidewalks and other impervious and pervious areas was simulated by full 2D Saint Venant equations. Flow exchanges occur between manholes and inlets accurately characterized using experimental formulas achieved by Technical University of Catalonia. The software used for the creation of the model was the version 7.5 of InfoWorks ICM by Innovyze.

The 1D/2D hydrological and hydraulic model will be the core of further models: the sediment transport module and the sea water quality model. The 1D/2D and the sea water quality model will be used for hazard and risk assessment in the following BINGO deliverables, whereas the sediment transport model will be used only for the analysis of the efficiency of a change in land use of upstream catchment as adaptation strategy to reduce sediment inflow in sewer network (different intervention measures for the forest/rural areas). The three different models are presented in the following 2 sections, the first section presents the 1D/2D drainage model and the sediment transport model (that is presented as a sub model of the 1D/2D model) and the second section presents the sea water quality model.



# 8.2. Model application – 1D/2D Drainage model

## 8.2.1. Model description

The 1D/2D model covers an extension of 21.7 km<sup>2</sup> covering the whole municipality, which has more than 215,000 inhabitants (third most-populated municipality in Catalonia with an average of 10,000 inhabit. /km<sup>2</sup>), involving 368 km of sewers and a 2D unstructured mesh with almost 80,000 cells was created on the basis of a detailed digital terrain model (DTM).

The model was calibrated and validated using a set of recent rainfall and water depth time series coming from a distributed sensors network of well-recorded flooding events that occurred between 2014 and 2016 and post event collected data (videos, photos, etc.).

### 8.2.1.1.1. Sediment transport model description

Sediments can affect the performance of sewer networks. Sediment deposits reduce the pipe flow capacity increasing the risk of flooding and CSOs. Sediments also carry contaminants that can affect the impact of CSOs on the receiving water bodies. Finally, sediments deposits in the drainage network are removed (usually couple of times a year) increasing sewer maintenance costs. The aim of the sediment transport model is to simulate the movement of sediment in the combined sewer network of Badalona.

Two different models have been set-up and calibrated under the BINGO framework:

 InfoWorks ICM model. A sediment transport module is added to the 1D/2D (drainage network/surface runoff) model (presented in previous sections). The added model simulates sediment generation from the urban surfaces and sediment transport inside the sewer. The sediment transport model runs simultaneously with the 1D/2D model.

• HEC-HMS model. An HEC-HMS model was set-up to quantify the amount of eroded sediments from the rural area of 'Serralada de la Marina' that enter into the sewer system. This area is located in the upstream part of the catchment of Riera Canyadò (see Figure 138). The upstream part is mainly rural area, it covers approximately 60% of the Riera Canyadò catchment and is considered as an important source of inorganic sediments. The downstream remaining part is dense urban area.

#### InfoWorks ICM sediment model

InfoWorks ICM allows the simulation of sediment erosion, deposition and transport of suspended sediment through the sewer system (only non-cohesive sediments were considered being the major part).

InfoWorks ICM calculates the sediment input from the urban surface to the combined sewer network using 2 models:

- Dry weather build up module. This module computes the dry weather accumulation of sediment both on the urban surface and on the manholes. The buildup equation is based on the hypothesis that, on a clean surface the rate of pollutant accumulation is linear but as surface mass increases the accumulation rate decays exponentially.
- Wet weather wash-off module. This module computes the time varying sediment discharge to the system networks depending on both the rainfall intensity and the accumulated sediment.

Once the sediment discharge is simulated, sediment transport in the drainage network is simulated using the Velikanov model (Ackers White and KUL model are also available in Infoworks ICM, but they were not

### D3.3 – Calibrated water resources models for past conditions



tested). The Velikanov model computes temporal and spatial variation of sediment deposition, erosion and suspended sediment concentration based on the flow velocity obtained from the hydraulic model.

The equation that describes the transport of suspended sediment is based on conservation of mass. It assumes that concentration of any suspended sediment is fully mixed across the section of the conduit and transported along the conduit with the local mean velocity of the flow. No dispersion is considered. Erosion and deposition of sediment depend on the settling velocity and the turbulence level that is related to the flow velocity in the pipe.

The Velikanov model was chosen because it is simple and with a limited number of parameters. Velikanov's energy equation allows the calculation of the capacity to transport materials in suspension for a flow with known hydraulic characteristics. The concentration of the transportable materials is in a range limited by two boundaries corresponding to the maximum and minimum concentration that can be transported:

$$\begin{array}{c|c} C_{\text{min}} & C_{\text{max}} \\ \hline & & C \\ \hline \text{erosion} & \text{no erosion / deposition} \\ \hline \\ C_{\text{min}} = \eta_{\text{min}} \cdot \frac{\rho_s \cdot \rho_m}{\rho_s - \rho} \cdot \frac{U}{w} \cdot J \\ \hline \\ \end{array} \begin{array}{c|c} C_{\text{max}} = \eta_{\text{max}} \cdot \frac{\rho_s \cdot \rho_m}{\rho_s - \rho} \cdot \frac{U}{w} \cdot J \\ \hline \\ \end{array}$$

 $\begin{array}{l} \eta_{\text{min}}, \eta_{\text{max}} \text{ efficiency coefficients} \\ p_m \text{ density of water (Kg/m^3)} \\ p=1 \\ p_s \text{ relative density of sediment to water (Kg/m^3)} \\ U \text{ mean flow velocity through pipe section (m/s)} \\ W \text{ sediment settling velocity (m/s)} \\ J \text{ pipe slope (m/m)} \end{array}$ 

The equation shows that:

- If the suspended solid concentration C is smaller than the minimum sediment concentration (Cmin), erosion occurs.
- If the suspended solid concentration C is larger than the maximum sediment concentration (Cmax), deposition occurs.
- If the suspended solid concentration C is between Cmin and Cmax there is no erosion/deposition (precisely, the rate of erosion equals the rate of deposition).

#### HEC-HMS sediment model

A HEC-HMS model was impermented in order to achieve a better estimation of the sediment tranpsort from rural areas that can be used as input to the InfoWorks ICM. This model was used to predict the generation of sediment in a rural subcatchemnt in the upstream part of Badalona drainage network. The results obtained in this section are not used as inputs to the InfoWorks ICM sediment model.

The software used for the creation of the model was the version 4.1 of HEC-HMS of the USACE army, which was designed to simulate the complete hydrologic processes of dendritic watershed systems.

| Area  | 2.0 Km <sup>2</sup> |
|-------|---------------------|
| Slope | 0.1%                |





Figure 131. Extent of the HEC-HMS basin model for Canyet sub-basin, a rural basin in the upstream part of the sewer network of Banalona.

The procedure adopted in HMS is the Modified Universal Soil Loss Equation (MUSLE) method. It is an estimate of sheet and rill soil movement down a uniform slope using rainfall energy as the erosive force acting on the soil and considering different soil characteristics (texture, structure, organic matter, and permeability).

The MUSLE method computes the sediment yield from a previous land segment for a storm event based on the following equation:

$$Sed = 11.8 \cdot (Q_{surf} \times q_{peak})^{0.56} \times K \times LS \times C \times P$$

Where:

| $Q_{surf}$ | Surface runoff volume (m <sup>3</sup> ) | event dependent |
|------------|---|-----------------|
| $q_{peak}$ | Peak runoff rate (m <sup>3</sup> /s)    | event dependent |
| K          | Soil erodibility factor                 | 0.6             |
| LS         | Topographic factor                      | 4.5             |
| С          | Cover and management factor             | 0.5             |
| Р          | Support practice factor                 | 0.01            |

## 8.2.2. Data

In order to ensure good and detailed results, it is not sufficient to have just a technologically advanced tool for the calculation concerning hydrological and hydraulic processes, several other aspects must be also carefully considered, such as the following:

Detailed topographic data able to reproduce complex urban morphologies. During a storm event in urban areas, generally, runoff produced in roofs and terraces is directly conveyed to the underground sewer networks, while runoff produced in streets and roadways circulates over the urban surfaces until it reaches the inlet structures of the drainage systems. Furthermore, stormwater runoff in is characterized by low flow depths (often less than 15-20 cm) and high flow velocities (up to 3-4 m/s due to the low roughness of the surfaces in urban areas. In this context, it is very important to have a DTM with high resolution. For this study, a specific 2 m<sup>2</sup> resolution DTM model has been used. This DTM was generated by a LIDAR (laser imaging detection and ranging) flight with a minimum density of 0.5 points/m<sup>2</sup> and a precision of 6 cm in terms of ground elevation provided by the Catalan Institute of Cartography.





Figure 132 Detailed terrain model of Badalona case study

- Definition of the Rainfall-Runoff transformation model (losses and routing models) and definition of pervious zones (2D Infiltration zones).
  - Hydrological initial losses: in this case, the totality of the runoff generated on roofs was considered directly connected to the sewer network (so it was neglected in the rainfall-runoff transformation of the 2D domain). Typically, roof drainage is designed to handle 1-5 minutes rainfall intensities with return periods larger than 10 (Real Decreto 314/2006). This means that during higher return period intensities roof drainage can fail. This is believed to have a small influence on the current flood analysis since only short duration intensities (<5min) can actually overtop the roof drainage. Nevertheless, roof drainage failure can create damages to buildings (Ruiz et al., 2018).</li>
  - Surface roughness coefficients: two different coefficients have been taken into account depending on the type of surface considered: streets and roads (0.016 s·m<sup>-1/3</sup>) and rural areas (0.025 s·m<sup>-1/3</sup>).
  - Hydrological losses (in pervious areas): Horton method has been implemented whose parameters have been calibrated in similar experiences (initial infiltration = 20 mm/h, residual infiltration = 7.2 mm/h, decay constant = 0.043 h<sup>-1</sup> and recovery constant = 0.108 h<sup>-1</sup>).
  - Routing parameters: cells characteristics and representation of the area excluded by 2D domain, which are building and other infrastructures (roads and train railways) that suppose an interruption of the surface flow and most of time have their own and independent drainage structures.





Figure 133 Defined 2D Infiltration zones in Badalona's 1D/2D coupled model

- Characterization of the surface drainage structures through ICM 'Gully 2D' nodes, which were hydraulically characterized on the basis of experimental data carried out in the UPC hydraulic laboratory.
- The hydraulic characterization of manholes and drain inlets between surface and underground systems allows the improved estimation of surface and pipe flows during a storm. This interaction between both systems (major and minor) is known as dual drainage. To achieve this objective the location and typology of all the inlets in the case study was determined. Indeed, the hydraulic performance of each inlet present in Badalona (more than 25,000 of more than 190 different types that were classified in groups represented by eight inlet types previously studied by Gómez & Russo 2011) was analyzed using experimental expressions proposed by Russo et al. (2013).



Figure 134 On the left Interaction of surface flow (Schmitt et al. 2004)5. On the right, head/discharge curve for a node with Gully 2D flood type to model the flow exchanges between the sewer system and the 2D meshed area

• Rainfall and flow-depths records for the adjustments of the model parameters in the calibration phase and for the results validation.



The existence of 3 rain gauges (spatial rainfall distribution was applied considering data coming from those three rain gauges) and 14 useful water level sensors (from AQUATEC) in the municipality administrative land (Figure 135) allows detailed calibration and validation processes.

Rainfall and water level time series were processed in order to obtain, respectively, five and one minutes time series.

- Sensor sewer network have been updated with new CSOs monitoring systems installed in Maria Auxiliadora and Riera Canyadó subcatchments outlet structures. In each point were installed:
  - o 1 radar level sensor and a temperature sensor upstream of the weir infrastructure.
  - o Another temperature sensor and a turbidity sensor downstream of the weir infrastructure.

When a CSO occurs both temperature sensors will indicate the same temperature so it is possible to detect the duration of the overflow and the frequency of this type of events along a period. Turbidity sensor was selected in order to provide information about water quality of the discharges reaching Badalona bathing waters. The schemes of the installation can be observed in the Figure 136.





Figure 135 Location of the rain gauges and water level sensors in Badalona



Figure 136 Installation scheme for the new installed sensors in Riera Canyadó and Maria Auxilidora outlet structures

Some photos of the new sensors are presented in the following figure.





Figure 137 Radar level sensor in Maria Auxiliadora subcatchment (right) and temperature level sensor in Riera Canyadó subcatchment (left)

# 8.2.2.1.1. Data for sediment transport

Data for the InfoWorks ICM sediment tranport model

Two different subcatchments in Badalona have been selected for monitoring purposes (Figure 138 left) and for in-sewer sediment characterization:

- Riera Canyadò. This subcatchment covers an extension of 5.4 km<sup>2</sup> and it is the catchment area of the stream Riera Canyadò that was channelized. The upstream part consists of rural area (60%), which is an important source of inorganic sediments into the network (according previous studies). The remaining dowstream part (40%) consist of dense urban area. Sediment traps were constructed upstream the urban area in order to limit the sediment input into the drainage network.

- Maria Auxiliadora: This subcatchment consists of dense urban area. It mainly produces wastewater with organic sediments.

Two different sets of data were collected at both sub-catchments:

- Sediment characterization: particle size distribution, density and content of organic matter. These data are relevant for model conceptualization and calibration and to improve our knowledge about the origins of the sediments present in the network.
- Turbidity and suspended solids concentrations measurements. These data are relevant for model calibration. The sensors of are located together with the temperature and water level sensors used to detect CSOs and described in the previous data sections. The observed concentrations of suspended solids will be shown together with the calibration results.

2 different sediment characterization campaigns were performed (the first one in January 2017 the second one the 24<sup>th</sup> of April 2018). The first campaign was performed in wet weather, whereas the second one after approximately 7 days of dry weather. Each campaign was performed during few hours by collecting sediment
samples at six different points in the Badalona network (see Figure 138 and Figure 140). An approximately 1dm<sup>3</sup> sample of deposited material was collected from the bottom of the sewer pipes using a shovel. The sediment samples were analyzed by the geotechnical laboratory of the Technical University of Catalonia.



Figure 138. First field campaign. Location of the 6 points where sediment samples were collected.



Figure 139. Second field campaign. Location of the 6 points where sediment samples were collected.

**Figure 140** and Figure 141 shows the results of the first campaign. The results show a d<sub>50</sub> (the diameter of the particle that 50% of the sample's mass is smaller) of approximately 1-2 mm; an organic matter content of 1.2-9.8 % and a specific density of 2.65-2.94 kg/dm<sup>3</sup>. These results suggest that the sediments deposited in the sewer network are mostly non-cohesive due to the low organic matter content and are made of coarse sand to fine gravel. Organic matter content is higher at Maria Auxiliadora (3.6-9.8%) compared to Riera Canyadò (1.2-1.4%). This is because the catchment of Maria Auxiliadora is almost entirely urbanized, whereas the Riera Canaydo one is partly rural. This low organic matter content allows the use of sediment transport models for non-cohesive sediments. Specific density is also slightly higher at Maria Auxiliadora.





Figure 140: First campaign results. Particle size distribution.

| Muestra | MO (%) | $ ho_{s}$ (Mg/m <sup>3</sup> ) |
|---------|--------|--------------------------------|
| MA-1    | 4.5    | 2.65                           |
| MA-2    | 3.6    | 2.73                           |
| MA-3    | 9.8    | 2.94                           |
| RC-1    | 2.3    | 2.65                           |
| RC-2    | 1.4    | 2.66                           |
| RC-3    | 1.2    | 2.66                           |

## Figure 141: First campaign results. Content of organic matter and specific density.

## Data for the HEC-HMS sediment tranport model

The data of this section come from previous studies and only the most relevant data are reported in the following. Three geomorphological sectors were identified:

- The Marina mountain range (south-east sector of Catalan coastal range) up to 466 meters. It is composed by granitic rocks.
- Between 50 and 100 meters there are detritic materials from tertiary age.
- In a lower height and until the coast there are detritic materials from quaternary age, like alluvial sediments of the bottom of natural streams. These sediments are composed by sand and gravel and are the ones that finally enter to the sewer network.

The particle size distribution that was used in the HEC-HMS model is shown in the following figure:



Figure 142. Particle size distribution used in HEC-HMS model. Data analyzed in the laboratory of the engineering departement of the Polytechnic University of Catalonia (UPC).

From Figure 142 it can be deduced that these sediments are made of coarse sand and fine gravel. These sediments accumulate into the sediment trap that is located upstream of the sewer network of Badalona (downstream the rural catchment modelled with HEC-HMS); when the sediment trap is full these sediments enter into the drainage network.

# 8.2.3. Calibration and validation strategies

Before calibration and validation processes were performed, several modifications of the Badalona 1D model have been undertaken due to the significant new infrastructures such as pipes and the stormwater tank built in the period 2012-2016.

Calibration is the procedure for ensuring an acceptable level of confidence in a model's ability to accurately represent the real system. It refers to the whole process of ensuring that a model behaves as similar to the real system as possible. For urban surface runoff models, it is recommended that at least three events are used (DHI 2002). In this case, 4 events were used for parameter estimation (calibration) and 1 event was used for model validation.

According to the results of a study developed by the Sheffield University (Shepherd et al. 2011) concerning the routing parameters in a 2D domain using InfoWorks CS, the extent of the flooded area is relatively insensitive to mesh density and surface roughness, while the modeling of building has however been shown to have a significant effect on flood depths.

From previous experiences, Manning roughness plays an important role in the flow velocity of the surface runoff, and its adjustments allows to reduce peaks time error. Discharge coefficients were also analyzed, but no significant change is detected for perimeters of the flooded areas.

Once sensitivity analysis was terminated, calibration and validation processes were carried out considering the following information of 5 rainfall events. The rainfall intensities have been associated to the corresponding return period T.



| Date event        | <b>P (mm)</b><br>Cumulative<br>rainfall | I (mm/h)<br>maximum rainfall<br>intensity in 20 minutes<br>(return period) | I (mm/h)<br><sup>5Min</sup><br>maximum rainfall<br>intensity in 5 minutes<br>(return period) | Event used<br>for |
|-------------------|---|--|--|-------------------|
| 22 August 2014    | 25.5                                    | 42.6 (T=0.4)   | 74.4 (T=0.6)   | Calibration       |
| 28 July 2014      | 46.5                                    | 56.4 (T=0.8)   | 91.2 (T=0.8)   | Calibration       |
| 03 October 2015   | 34.1                                    | 81 (T=2.1)   | 122.4 (T=1.1)  | Calibration       |
| 14 September 2016 | 21.7                                    | 64.5 (T=1.1)   | 142.8 (T=6.1)  | Calibration       |
| 18 June 2016      | 24.6                                    | 60.3 (T=0.8)   | 105.6 (T=0.9)  | Validation        |

## Table 33 Events selected for calibration and validation of the model

# 8.2.3.1.1. Calibration and validation of the sediment transport model

## InfoWorks ICM sediment model calibration

The calibrated parameters to simulate sediment transport in the sewer network were:

- Buildup parameters. In this case, the buildup sediment mass parameter has been taken as a characteristic value of each subcatchment, considering 18 Kg/ha·day for rural catchments (like Riera Canyadó) and 35 Kg/ha·day for urban catchments (as Maria Auxiliadora). Considering fixed this parameter, the decay factor was calibrated. This parameter regulates the quantity of mass that is generated in every subcatchment for each time step.
- Wash-off parameters. The most sensitive parameter in the wash-off formula is the exponential rainfall coefficient which was calibrated in order to represents the quantity of mass that flows into the sewer network.
- Sediment transport parameters. Velikanov model only requires the calibration of two parameters  $\eta_{min}$ ,  $\eta_{max}$ . These efficiency coefficients regulate the concentration of sediments eroded or deposited as a function of the flow velocity.
- The following table shows the three rain events with available suspended solid concentration data were used for the calibration. It is noted that the HEC-HMS model outputs were not used for the model calibration.

|            | Suspended sediment data |         | Rain         |        | lmax.  | lmax     | T (Imax) |
|------------|-------------------------|---------|--------------|--------|--------|----------|----------|
| Event      | Maria                   | Riera   | Gauge P (mm) | P (mm) | (mm/h) | (mm/h)   | (years)  |
|            | Auxiliadora             | Canyadó |              |        |        |          |          |
|            |                         |         | BA1          | 67.0   | 20.4   | 16.5     | 0.1      |
| 24/03/2017 | 3/2017 FAIL             | ОК      | BA2          | 67.4   | 87.6   | 39.6     | 0.4      |
|            |                         |         | BA3          | 54.4   | 24.0   | 22.5     | 0.2      |
|            |                         |         | BA1          | 3.6    | 6.0    | 3.6      | 0.1      |
| 30/05/2017 | 5/2017 OK               | ОК      | BA2          | 3.6    | 6.0    | 3.9      | 0.1      |
|            |                         |         | BA3          | 4.2    | 6.0    | 4.2      | 0.1      |
|            |                         |         | BA1          | 12.8   | 38.4   | 33.6     | 0.3      |
| 14/09/2017 | ОК                      | ОК      | BA2          | 13.0   | 57.6   | 37.8     | 0.4      |
|            |                         |         | BA3          |        | not av | vailable | •        |

Table 34. Events used for the calibration of the sediment transport model.

#### HEC-HMS sediment model calibration

For the calibration of the model two different precipitation events have been analyzed:

Table 35. Calibration events for the HEC-HMS model

| Precipitation<br>Event | Maximum 5 minutes<br>intensity (mm/h) | Estimated<br>volume of<br>sediments (m <sup>3</sup> )* |  |
|------------------------|---------------------------------------|--|--|
| 18/06/2016             | 106                                   | 1.2  |  |
| 16-17/09/2016          | 100                                   | 0.8  |  |
| ь.                     |                                       |  |  |

\* Volume of sediment estimated from visual inspections of the sediment trap at the entrance of the sewer at Canyet sub-basin.

It is important to remark that, no measurements of flow were available so the whole HMS model was only calibrated trying to match the calculated values with the sediment observed in the trap. Although the efficiency of the sediment trap is not known, it was assumed that all the sediment observed in the trap after the rain event was the same amount of eroded material in the basin.

For the first event, the practice factor was adjusted to get a sediment load similar to the volume of sediment observed during the visual inspection (it is remarked than the sediment trap was clean before the event).

### 8.2.4. Results

The results concerning several sensors located in Badalona are shown and summarized in the following figures. These results show great correspondence between simulated and observed flow depths in the Badalona sewers network.



Figure 143 Calibration results (for the four selected events) regarding a monitored manhole (BA21) in Torrent Vallmajor street. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths





Figure 144. Calibration results (for the four selected events) regarding a monitored manhole (BA2) in Riera Canyadó street. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths.



Figure 145 Calibration results (for the four selected events) regarding a monitored manhole (BA4) in Martí Pujol Avenue. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths



Figure 146 Calibration results (for the four selected events) regarding a monitored manhole (BA16) in Sant Ignasi de Loiola Avenue. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths



Figure 147 Calibration results (for the four selected events) regarding a monitored manhole (BA15) in Alcalde Xifré Square (oufall of the Estrella water tank). Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths





Figure 148 Calibration results (for the four selected events) regarding a monitored sluice (BA12) in the entrance to Estrella water tank in Sant Joan Avenue. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths



Figure 149 Calibration results (for the four selected events) regarding a monitored sluice (BA9) in the entrance to Estrella water tank in Torrent Batlloria Street. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths



Figure 150 Calibration results (for the four selected events) regarding a monitored manhole (BA19) in Ponent Street. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths



Figure 151 Calibration results (for the four selected events) regarding a monitored manhole (BA20) in Mar Jònica Street. Non-continuous lines show measured flow depths, while continuous ones show simulated flow depths

## 8.2.4.1.1. Results of the sediment transport model

#### Results of the InfoWorks ICM sediment model

Figure 152 shows the calibration results. The green lines show selected simulation results that were obtained by trial error calibration of the parameters, the red line is the selected best fit, the blue line is the simulated

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flow (secondary axes). Overall, the simulated green line show that there is a large uncertainty in TSS concentrations, this is typical in sewer sediment transport modelling; the simulated concentrations show similar pattern to the simulated flow, this is because the TSS concentrations are a function of the flow velocity in the selected Velikanov model; further, the model overestimates observed concentrations; finally, despite the order of magnitude of the simulated concentrations might look reasonable, observed patterns are not well reproduced.

There is a time-lag lack in representing the peak time in all the simulations. This peak timing cannot be reproduced by the sediment transport formulas because is related to the hydraulic behavior (where a peak time difference also exists).

The total amount of sediment that is discharged from the catchment is not kwon, and this implies a significant uncertainty. Moreover, from the sampling campaign just few values are obtained and usually in a very short period of time (around one hour data), and these data are not enough to compare the obtained results from the model.



Figure 152. Calibration results. Simulated vs observed Total Suspended Solid concentrations. The simulated flow is also shown.

#### Results of the HEC-HMS sediment model

Figure 153 shows an example of sediment graph obtained through model calibration. The figure shows the time variation of sediment load at the location of the sediment traps during a rainfall event. The total sediment load was calibrated to match the observed volumes of sediment (see the data section).



Figure 153. Calibration results of the HEC-HMS model

## 8.3. Model evaluation and discussion of the 1D/2D drainage model

The central step in the calibration processes is the iterative adjustment of model parameters in order to match model results with observations. The complexity of the processes depends on the number of calibration parameters being adjusted, and the character of the calibration parameter and the modeling concepts being applied determines its transparency. Calibration may be understood as an optimization process involving one or several simultaneous objective functions, which represent formalized measures of deviations between measurements and simulations. In manual calibration, these measures are used with visual observations in the evaluation of goodness-of-fit of simulations to observations. Common numerical performance measures were used in the calibration of the 1D/2D Badalona model.

Calibration with statistical analysis generally calculates root mean squared error as a statistical measure of deviation between measured error as a statistical measure of deviation between measured and simulated time series:

$$RMSE = \sqrt{\frac{\sum_{j=1}^{n} (Meas_j - Sim_j)^2}{n}}$$

where, RMSE is the root mean squared error;  $Meas_j$  is the measured (observed) flow depth at time j;  $Sim_j$  is the simulated flow depth at time j; n is the number of time steps in the calibration/validation period.

Another statistical measure of deviation used in the statistical analysis is the coefficient of determination as follows:

$$R^{2} = \frac{\left[\sum_{j=1}^{n} \left(Meas_{j} - \overline{Meas_{j}}\right) \cdot \left(Sim_{j} - \overline{Sim_{j}}\right)^{2}\right]^{2}}{\sum_{j=1}^{n} \left(Meas_{j} - \overline{Meas_{j}}\right)^{2} \cdot \sum_{j=1}^{n} \left(Sim_{j} - \overline{Sim_{j}}\right)^{2}}$$



where,  $R^2$  is the coefficient of determination;  $\overline{Meas_j}$  is the mean value of measured (observed) flow depths during the calibration/validation period;  $\overline{Sum_j}$  is the mean value of the simulated flow depths during the calibration/validation period.

Finally, peak error (PE) and time to peak error (TPE) are expressed as follow:

$$PE = |max(Meas_j) - max(Sim_j)|$$
$$TPE = |t\{max(Meas_j)\} - t\{max(Sim_j)\}|$$

Statistical analysis of the model was carried out calculating all these parameters previously described. A summary is presented in the following table.

|                               |                                 |                |           | Measured          | Simulated         | 、      | TPE       |  |
|-------------------------------|---------------------------------|----------------|-----------|-------------------|-------------------|--------|-----------|--|
| Code                          | Location                        | R <sup>2</sup> | RMSE      | peak<br>level (m) | peak level<br>(m) | PE (m) | (minutes) |  |
| Calibration event: 22/08/2014 |                                 |                |           |                   |                   |        |           |  |
| BA21                          | Torrent Vallmajor Street        | 0.94           | 0.04      | 0.58              | 0.33              | 0.25   | 2         |  |
| BA2                           | Riera Canyadó Street            | 0.94           | 0.04      | 0.35              | 0.13              | 0.22   | 7         |  |
| BA4                           | Martí Pujol Avenue              | 0.71           | 0.11      | 0.37              | 0.24              | 0.13   | 9         |  |
| BA16                          | Sant Ignasi de Loiola<br>Avenue | 0.84           | 0.18      | 0.99              | 0.55              | 0.44   | 15        |  |
| BA15                          | Andrés Xifré Square             | 0.61           | 0.20      | 1.42              | 0.52              | 0.90   | 10        |  |
| BA12                          | Sant Joan Avenue                | 0.76           | 0.12      | 1.07              | 0.94              | 0.13   | 11        |  |
| BA9                           | Torrent Batlloria Street        | 0.52           | 0.11      | 1.07              | 0.41              | 0.66   | 15        |  |
| BA19                          | Ponnent Street                  | 0.88           | 0.04      | 0.34              | 0.35              | 0.88   | 9         |  |
| BA20                          | Mar Jònica Street               | 0.93           | 0.17      | 0.49              | 0.42              | 0.07   | 7         |  |
| Calibra                       | tion event: 28/07/2014          |                |           | •                 | •                 |        | •         |  |
| BA21                          | Torrent Vallmajor Street        | 0.73           | 0.08      | 0.73              | 0.51              | 0.22   | 25        |  |
| BA2                           | Riera Canyadó Street            | 0.54           | 0.05      | 0.38              | 0.10              | 0.28   | 60        |  |
| BA4                           | Martí Pujol Avenue              | 0.52           | 0.05      | 0.48              | 0.26              | 0.22   | 28        |  |
| BA16                          | Sant Ignasi de Loiola<br>Avenue | 0.86           | 0.03      | 3.19              | 1.13              | 2.07   | 10        |  |
| BA15                          | Andrés Xifré Square             | 0.69           | 0.20      | 1.70              | 1.04              | 0.66   | 16        |  |
| BA12                          | Sant Joan Avenue                | 0.78           | 0.17      | 2.27              | 1.93              | 0.34   | 11        |  |
| BA9                           | Torrent Batlloria Street        | 0.56           | 0         | 1.08              | 0.76              | 0.32   | 28        |  |
| BA19                          | Ponnent Street                  | 0.93           | 0.1       | 1.38              | 0.76              | 0.62   | 8         |  |
| BA20                          | Mar Jònica Street               | 0.86           | 0.15      | 1.38              | 1.20              | 0.18   | 11        |  |
| Calibra                       | tion event: 03/10/2015          |                |           |                   |                   |        |           |  |
| BA21                          | Torrent Vallmajor Street        | 0.98           | 0.10      | 0.96              | 0.77              | 0.19   | 4         |  |
| BA2                           | Riera Canyadó Street            | 0.96           | 0.09      | 0.52              | 0.16              | 0.36   | 1         |  |
| BA4                           | Martí Pujol Avenue              | 0.63           | 0.15      | 1.82              | 0.40              | 1.42   | 14        |  |
| BA16                          | Sant Ignasi de Loiola<br>Avenue | 0.76           | 0.85      | 3.12              | 1.29              | 1.83   | 53        |  |
| BA15                          | Andrés Xifré Square             | 0.78           | 0.28      | 1.33              | 1.32              | 0.01   | 25        |  |
| BA12                          | Sant Joan Avenue                | 0.86           | 0.30      | 2.44              | 3.23              | 0.79   | 14        |  |
| BA9                           | Torrent Batlloria Street        | 0.65           | 0.01      | 0.82              | 1.55              | 0.73   | 9         |  |
| BA19                          | Ponnent Street                  | 0.96           | 0.21      | 1.54              | 0.85              | 0.69   | 6         |  |
| BA20                          | Mar Jònica Street               | 0.86           | 0.33      | 1.52              | 1.46              | 0.06   | 11        |  |
| Calibra                       | tion event: 14/09/2016          |                | 1         |                   | 1                 | 1      | I         |  |
| BA21                          | Torrent Vallmajor Street        | 0.86           | 0.11      | 0.91              | 0.75              | 0.17   | 8         |  |
| BA2                           | Riera Canyadó Street            | 0.86           | 0.05      | 0.53              | 0.11              | 0.42   | 9         |  |
| BA4                           | Martí Pujol Avenue              | 0.71           | 0.04      | 0.45              | 0.40              | 0.05   | 11        |  |
| BA16                          | Sant Ignasi de Loiola<br>Avenue | 0.73           | 0.15      | 1.61              | 1.24              | 0.37   | 10        |  |
| BA15                          | Andrés Xifré Square             | 0.76           | 0.13      | 1.73              | 1.28              | 0.45   | 9         |  |
| BA12                          | Sant Joan Avenue                | 0.80           | 0.14      | 1.98              | 2.07              | 0.09   | 4         |  |
| BA9                           | Torrent Batlloria Street        | 0.48           | 0         | 1.21              | 1.50              | 0.29   | 11        |  |
| BA19                          | Ponnent Street                  | -              | -         | -                 | -                 | -      | -         |  |
| BA20                          | Mar Jònica Street               | 0.92           | 0.12      | 1.12              | 0.77              | 0.35   | 3         |  |
|                               |                                 | Validatio      | on event: | 18/06/2016        | <b>_</b>          |        | -         |  |
| BA21                          | Torrent Vallmajor Street        | 0.94           | 0.03      | 0.90              | 0.70              | 0.20   | 7         |  |
| BA2                           | Riera Canyadó Street            | 0.98           | 0.05      | 0.54              | 0.16              | 0.38   | 4         |  |
| BA4                           | Martí Pujol Avenue              | 0.67           | 0.04      | 0.51              | 0.43              | 0.08   | 10        |  |
| BA16                          | Sant Ignasi de Loiola<br>Avenue | 0.65           | 0.18      | 1.64              | 1.20              | 0.44   | 11        |  |
| BA15                          | Andrés Xifré Square             | 0.62           | 0.15      | 1.61              | 1.15              | 0.46   | 10        |  |
| BA12                          | Sant Joan Avenue                | 0.71           | 0.15      | 1.62              | 1.90              | 0.28   | 11        |  |
| BA9                           | Torrent Batlloria Street        | 0.64           | 0.01      | 1.22              | 1.15              | 0.07   | 13        |  |
| BA19                          | Ponnent Street                  | -              | -         | -                 | -                 | -      | -         |  |
| BA20                          | Mar Jònica Street               | 0.97           | 0.15      | 1.19              | 0.73              | 0.46   | 5         |  |

Table 36 Statistical parameters related to calibration processes for the sewer facilities used in this study



In general, results show a good agreement between measured (observed) and simulated values. But some remarks can be done regarding the exposed values:

- In almost all cases, the temporal evolution is quite similar in the measured and simulated series and allows adequate matching of intermediate peaks.
- For less extreme events differences seems to decrease.
- Big differences in calibration event 2 can be explained due to the lack of information of the data for rain gauge PBA3 (installed after this event).

Surface flow gauges are not so common in urban drainage calibration and validation processes regarding overland flow. In this case, flooding parameters provided by the model were compared to observed data collected in the post-events videos and photos recorded during selected storms events. In this way, it is possible to compare overland flow depth to reference elevations of objects located on the surface. For example surcharged pipe flow condition (with an evident overflow producing the displacement of the gully covering the manhole), is well simulated by the model as shown in the following figure.



# Figure 154 Sewer network effects in the surfaces at Molí de la Torre Street for the 14 September 2016 event

Another example of the validation of the simulated surface flow can be observed in other historical event previous to the construction of the stormwater tank (April 28th, 2011). Flow depths provided by the model for this event were compared to a video recorded during this event. The model reflects the runoff recorded in the video images achieving water depths around 15 cm.



Figure 155 Calibration results for Pere Martell Street: flow depths provided by the model for the event of 28 April 2011 were compared to videos recorded during this event. Specifically, in the bottom of the figure, (inside the red square) it is possible to observe the runoff generated, which is arround 15 cm. In the upper part of the figure an image of an amateur video shows the runoff during the event.

# 8.3.1.1. Discussion of the sediment transport model

The sediment transport model will be used to evaluate the impacts of different restoration measures of the forest areas. This section showed a first calibration of the sediment transport model that was made with sparse data. The significant TSS concentration uncertainties are in line with similar studies of sediments in sewers (Freni et al., 2018; Ebtehaj et al., 2013). A better calibration could be obtained if more data were available, i.e. the collected turbidity data (that are much easier to collect compared to TSS) could be used. This requires local calibration between turbidity and TSS concentrations. Further, improved estimation of sediment inputs to the drainage network could be considered, i.e. by using the HEC-HMS model presented. The model could also be used to evaluate the impacts of including sediment transport in flood and CSO modeling, however this will not be part of BINGO.



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# 8.4.1. Model application. Sea water quality model

## 8.4.1.1. Model description

The sea water quality model aims at simulating the near shore (within few hundred meters from the shore line) E.Coli concentrations in the Badalona sea water after CSO events. Every summer several (generally less than 10) CSOs events occur and the municipality forbids bathing in the sea after the CSO spill due to insufficient sea water quality (R.D. 1341/2007 of the 11oct2007). During moderate rainfall events (usually larger than few mm) CSOs occur through the approximately 20 weirs located along the beaches of Badalona. Figure 156 shows an example of a sea water quality simulation after a CSO event. A through description of the Badalona urban and catchment hydrology was provided in D3.1.



# Figure 156. Sea water quality simulation after a CSO event in Badalona (Red=high; Blue=low E.Coli concentrations in sea water).

The sea water quality model was developed using the software MOHID Studio from Bentley Systems. MOHID is a three-dimensional water modelling system, developed by MARETEC (Marine and Environmental Technology Research Center) at Instituto Superior Técnico (IST). In the Badalona Research site, the model simulates both the hydrodynamics of the sea in the coastal region and the contaminant transport resulting from CSOs. The model was originally developed within the COWAMA (Coastal Water Management) project (Suñer, 2007, 2008) that provided a computational model operating since 2007 for real time simulations of bathing water quality of the Barcelona beaches. The marine model part of COWAMA was modified in order to include the area of Badalona and observations in Badalona were collected in order to validate the model. In the following, an overview of the main simulated processes and the modifications of the sea water quality model of the BINGO project is provided, while further model details can be found in (Suñer, 2007, 2008).

Simulation of near shore water quality requires spatial discretization scales in the order of tens of meters whereas coastal hydrodynamic processes occur at scales of hundreds of kilometers. Therefore, 3 nested model domains are used to simulate hydrodynamic processes from the large regional scale to the local near shore scale of Badalona. Figure 157 shows the three nesting model levels. Level 1 covers an area of approximately 20,000 km<sup>2</sup> with squared cells of approximately 1x1 km<sup>2</sup>. At this domain the hydrodynamic processes of astronomic tides and wind induced currents are simulated in 2D (barotropic) mode and the outputs are used as boundary conditions for the nested Level 2. Level 2 covers an area of approximately 1000 km<sup>2</sup> with rectangular cells of sides from 500 m to 200 m (finer cells close to the shore line). At this domain the hydrodynamic processes occurring at the continental shelf associated with wind action, wind generated waves and density driven circulations induced by fresh water river plumes are simulated in 3D mode and the outputs are used as boundary conditions of the nested Level 3. Level 3 covers an area of

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approximately 50 km<sup>2</sup> with rectangular cells of sides from 200 m to 40 m (finer cells close to the shore line). At the Level 3 domain both hydrodynamics and contaminant transport from CSO events is simulated. The simulated processes include the effects of currents and waves; density, temperature and salinity variations; near shore currents generated by CSOs discharges; solar radiation effects on mortality of E.Coli; advection and diffusion of E.Coli from CSOs. Level 3 is simulated in 3D mode and provide time series of E.Coli concentrations to be compared with observations. The Level 3 computational mesh was extended to the Badalona model area (the original model was developed for Barcelona).



Figure 157. The 3 nested model domains of the sea water qulaity model. Level 1, regional scale, O(200km); Level 2, sub-regional scale, O(50km); Level 3, city scale, O(10km).

Figure 157 shows the location of the CSOs discharges in Badalona. CSOs are simulated in the marine model as both water discharge and concentration inputs. Water discharge time series at the CSO points are obtained from the urban 1D/2D drainage model of Badalona. The input concentrations used for CSO discharges are assumed to be fixed. A value of 1e<sup>6</sup> CFU/100 ml (Colony Forming Units per 100 mL) was used and it was estimated based on field measurements shown in the following section. Output uncertainty bounds were also simulated by assuming lower (1e<sup>5</sup> CFU/100 ml) and higher (1e<sup>7</sup> CFU/100 ml) E.Coli inputs.

The Escherichia Coli mortality/decay is simulated assuming a first order exponential decay with a  $T_{90}$ , the time at which 90% of the bacterial population is no longer detectable, meaning a 1 log reduction of the number of pathogens.



Figure 158. CSO discharge locations (violet circles) and 'Pont del Petroli', a pedestrian bridge used for the sea water sampling campaigns carried out as part of the BINGO project.

# 8.4.1.2. Data

Field data are essential in order to calibrate and validate the model. Water quality data of both sea water and CSO water were collected.

The water quality of CSO water discharges was measured at the CSO structures of Riera Canyadò and Maria Auxiliadora during 2 different CSO events. **Fehler! Verweisquelle konnte nicht gefunden werden.** shows the location where the data were collected and a sketch of the sampling methodology. Several water samples were collected during the first hours of rain events, the first samples were used to estimate the first flush effect, and the rest of the samples were used to calculate the mean E.Coli concentration of the spill.







Table 37 shows the E. Coli concentration results of the field campaigns. Mean E. Coli concentrations were in the range of  $2.1 \cdot 10^5 - 3.4 \cdot 10^6$  ufc/100 ml. It is difficult to analyze and interpret the data due to the limited number of measurements. More than an order of magnitude of different between the minimum and the maximum E. Coli concentrations was observed. Moreover, the largest rainfall event (24/03/2017) had the lowest concentrations, likely due to more dilution due to the larger stormwater runoff volume.

| Date       | Maria Auxiliadora [ufc/100 ml] | Riera Canyadó [ufc/100 ml] | Rainfall             |
|------------|--------------------------------|----------------------------|----------------------|
| 24/03/2017 | Not available                  | 2.1·10 <sup>5</sup>        | 55-67 mm in 12 hours |
| 30/05/2017 | 7.4·10 <sup>5</sup>            | 3.4·10 <sup>6</sup>        | 3-4 mm in 3 hours    |
| 10/04/2018 | Not available                  | 2.5·10 <sup>6</sup>        | 8-10 mm in 3 hours   |

| Table 37. | Water quality | results of th | ne field | campaign. |
|-----------|---------------|---------------|----------|-----------|
|-----------|---------------|---------------|----------|-----------|

The sea water quality was measured by the municipality of Badalona. Sampling campaigns consisted in taking water samples in the sea after (sometimes also during) rainfall/CSO events. Water samples were taken at 'Pont del Petroli' (see Figure 160. Sea water quality sampling locations. The 'Pont del Petroli' cross section shows the three sampling locations.), a pedestrian bridge that allows taking measurements at different distances from the shore line. The samples were taken once a day (normally between 9AM and 2PM) at 3 different points along the 'Pont of Petroli' (1-close to the shoreline, 2-in the middle of the pedestrian bridge and 3-at the most offshore point) for the few days following rainfall events until bacterial concentrations would recover to values complying with water quality standards. Figure 161 shows an example of the water quality parameters measured at the 'Pont of Petroli' by the municipality of Badalona.

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Figure 160. Sea water quality sampling locations. The 'Pont del Petroli' cross section shows the three sampling locations.

| Núm. registre                                  | 29                  | 30                  | 31                  |
|--|---------------------|---------------------|---------------------|
| Punt/Paràmetre                                 | 1 Línia costa       | 2 Centre            | 3 Extrem            |
|  |                     | pont                | pont                |
| Escherichia coli (ufc/100 ml)                  | 4.9x10 <sup>3</sup> | 1.9x10 <sup>3</sup> | 7.8x10 <sup>2</sup> |
| (PNT-M-6, M-29, M-49)                          |                     |                     |                     |
| Enterococs intestinals (ufc/100 ml)            | 3.8x10 <sup>2</sup> | 2.3x10 <sup>2</sup> | 90                  |
| (PNT-M-6, M-20)                                |                     |                     |                     |
| Salinitat (%) (PNT-Q-68)                       | 37.6                | 37.9                | 37.9                |
| MES (mg/l) (PNT-Q-13)                          | 22                  | 15                  | 10                  |
| Oxidabilitat (mg O <sub>2</sub> /I) (PNT-Q-21) | 37                  | 46                  | 59                  |
| Terbolesa (UNT) (PNT-Q-71)                     | 11.0                | 4.3                 | 2.6                 |

Figure 161. Example of the water quality indicators measured at 'Pont of Petroli' by the municipality of Badalona.

# 8.4.1.3. Sensitivity analysis

Before calibrating and validating the model, a sensitivity analysis was performed. Different sensitivity scenarios have been simulated in order to improve the discussion about the model results, to qualitatively assess the model sensitivity to the different parameters/inputs and to improve model calibration. The sensitivity analysis was performed on the original model parameters (it is reminded that this model is an updated version of the original model available for Barcelona).

The sensitivity analysis showed that (a) wind speed and (b) E. Coli mortality rates are the most influential parameters for simulating the decay of sea water concentrations during the few days following a CSO event; moreover, (c) input E. Coli concentration (associated to the CSO discharge) is the most influential parameter to simulate the peak sea water concentrations during the CSO event. Classical sensitivity analysis (where parameters are ranked based on their influence on selected model outputs) was not performed because data are sparse and input parameters (i.e. wind speed, E. Coli mortality rates) can vary in space and time making the sensitivity analysis difficult to interpret.

The different sensitivity scenarios were:

- Baseline. It has 10<sup>6</sup> ufc/100 ml input E. Coli concentrations, the other model parameters are the same as the original model (Suñer, 2007, 2008).
- Without wind. It assumes no wind, therefore wind induced currents in the sea are zero.

- High grid resolution. It has a fourth level nesting model with cell sizes of approximately 20 m.
- Higher E. Coli mortality. It assumes a *T*<sub>90</sub> with 2.5 times faster mortality. The factor 2.5 is an average ratio between mortality rates calculated with the two different mortality models of Canteras (1995) and Chapra (1997). Both of the models have the same inputs: solar radiation, sea water temperature and salinity.
- No E. Coli mortality. It assumes that the mortality rate of E. Coli is zero.
- Turbulence model. A different turbulence model was chosen in the model interface.
- Wind from a station. Wind speed and wind direction are taken from a local station in the harbor of Barcelona. The baseline scenario took wind results from a model output at a cell of 10 km size covering the area of Badalona.



Figure 162. Sensitivity scenario results.

Figure 162 shows the results of the sensitivity analysis. All the scenarios show very similar peak concentrations (1.0·10<sup>5</sup>-5.0·10<sup>5</sup> ufc/100 ml), this supports the fact that peak concentrations are mostly sensitive to input E.Coli concentrations. The following discussion analyses the decay of E. Coli concentrations during the few days following the CSO event.

The 'without wind' scenario shows overall concentrations of an order of magnitude higher than the baseline. The 'wind from a station' scenario shows similar concentrations to the baseline scenario, despite for the second simulation day where concentrations are an order of magnitude lower. Wind is the main process generating near-shore sea water velocities in the order of 0.1-0.2 m/s during the selected events. Input wind speed taken from a station can overestimate hydrodynamic velocities (resulting in lower concentrations due to higher dispersion). Instead, the baseline scenario is based on grid cell inputs which is a likely better approximation of large scale wind patterns.

The 'High grid resolution' scenario shows that the 'baseline' model grid is able to capture the major processes correctly, nevertheless the baseline model overestimate concentrations in the first hours after the CSO event. The larger grid of the baseline scenario produces artificial numerical diffusion that fails to reproduce steep concentration gradients that can occur during the first few hours following the CSO event.

The 'Turbulence model' scenario shows slightly lower concentrations. This gives an idea of the output uncertainty generated by the choice of a different turbulence model.

The 'Higher E. Coli mortality' scenario shows overall concentrations almost an order of magnitude lower than the baseline. The 'No E. Coli mortality' scenario shows overall concentrations almost an order of magnitude higher than the baseline. This shows that the mortality rates of E. Coli are a very influential parameter.



# 8.4.1.4. Calibration and validation

The original marine model was developed for the area of Barcelona and it was modified in the framework of BINGO creating a new computational grid covering the model area of Badalona) in order to simulate the sea water quality of the beaches of the city. Therefore, model calibration and validation of the new model area of Badalona is necessary. There different periods with available sea water quality data after CSO events were simulated in order to calibrate and validate the model:

- From 26/01/2018 to 30/01/2018 (calibration event).
- From 13/09/2016 to 17/09/2016 (calibration event).
- From 12/10/2016 to 18/10/2016 (validation event).

The starting simulation time of the marine model was approximately 24 hours before the beginning of the CSOs (model warm-up/spin-up period) in order to make sure that the coastal hydrodynamics were properly simulated when the CSO event started. The calibrated model parameters were the wind drag coefficient and the E. Coli mortality rate  $T_{90}$ .  $T_{90}$  is theoretically a function of sea water temperature, salinity and solar radiation; nevertheless, similar studies on sea water quality modelling in the Adriatic Sea assumed fixed values of  $T_{90}=2.3 \text{ days}$  (Scroccaro et al., 2010; De Marchis, 2013). In this study the calibration considered an hourly varying  $T_{90}$  with lowest night values of 4 days and highest day values of 1 day. The wind drag coefficient controls the wind generated coastal currents and the calibrated value was 0.0008. This value is within the observed values (from 0.5 to 1.5) presented by Large and Pond (1980). A higher drag produces higher shear stresses and therefore higher sea water velocities and as a consequence a higher advection and dispersion of the contaminant plume.

The marine model inputs are: the time series of CSOs water discharge for every CSO structure in Badalona obtained from the 1D/2D urban drainage model by simulating rainfall events using observations from local pluviometers; rainfall time series from 3 different pluviometers in Badalona applied to the drainage model based on Thyssen polygons; the wind speed and direction taken from national scale circulation models (Puertos del Estado); air and water temperature and solar radiation based on local measurements.

# 8.4.1.5. Results

Figure 163 shows the comparison of the simulated E. Coli concentrations with the observed concentrations at three different locations along the pedestrian bridge 'Pont del Petroli'. The black continuous lines shows the simulation results obtained using an E. Coli input concentration of 10<sup>6</sup> cfu/100ml. The upper and lower uncertainty bounds were obtained with input concentrations of 10<sup>7</sup> and 10<sup>5</sup> cfu/100ml respectively.

Overall, the model can reproduce both the order of magnitude and the patterns of the observed E. Coli concentrations. Most of the observed 'peak' concentrations fall between the simulated uncertainty bounds. The main model uncertainty here considered is due to the input E. Coli concentration from the CSO discharges. Field observations showed mean E. Coli concentrations at CSO structures of  $2.1 \cdot 10^5 - 3.4 \cdot 10^6$  ufc/100 ml; previous projects (Suñer et al., 2008) used inputs of  $1 \cdot 10^7$ . The  $1 \cdot 10^7$  ufc/100 ml is considered to be a worst case concentration as it is a typical concentration of combined sewer without being diluted with storm water runoff.



Figure 163. E. Coli simulation results during 3 different CSO events at three different measuring points along the pedestrian bridge 'Pont del Petroli'. The 500 CFU/100 ml is the bathing water quality threshold.

Salinity was also simulated and compared with the observations (Figure 164). Overall the model seems to reproduce the observed salinity, however it is difficult to properly judge the model performance due to lack of more frequent measurements, particularly during low concentration periods (during the CSO event and the following hours). Observed salinity seems to recover to typical sea water salinity (approximately 37-38 ‰, according to http://www.pontdelpetroli.org) already one day after the CSO events when bacterial concentrations are still beyond water quality thresholds.

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Figure 164. Salinity simulation results during 3 different CSO events at three different measuring points along the pedestrian bridge 'Pont del Petroli'.

Finally, the depth variations of the main model outputs have been presented: E.Coli concentrations, salinity and sea water velocity (Figure 165). The model results are shown at the measuring point corresponding to 200 m offshore on the pedestrian bridge 'Point del Petroli' (the water depth here is approximately 8 m) at two different time steps: at the beginning of the CSO event and after 6 hours. The results show that at the beginning of the CSO event there are significant variations of E. Coli concentrations, salinity and velocity in the first meter below sea water surface. These variations are smoothen out already 6 hour later. This means that at the early stages of CSO events, stratification due to different salinity and water temperature between sea water and CSO discharge is significant. These results can be useful for planning/discussing future field campaigns.



Figure 165. E.Coli, salinity and velocity simulations at 200 m offshore on the pedestrian bridge 'Pont del Petroli'.

### 8.4.2. Evaluation and discussion of the sea water quality model

Overall the sea water quality model can reproduce observed concentrations with order of magnitude precision. Specifically the results of calibration and validation processes demonstrate the adequacy of the model for the hazard and risk assessment in the framework of WP3 and WP4. Significant spatial and temporal variation of the simulated indicators (E.Coli and salinity) was observed. Sea water salinity could also be simulated and it was shown to significantly fall after CSOs (CSO water has no salinity) and to recover typical sea water salinity already one day after the observed CSO events when bacterial concentrations were still beyond water quality thresholds.

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# 9. Conclusions and outlook

D3.3 is a comprehensive report, which describes the multiple model developments and applications for the six BINGO research sites for past conditions. The different model applications have been developed site-specific in order to provide tools and solutions for local water management problems related to climate variability and climate change. Therefore, it is important to show that the models are able to realistically represent past conditions. As different model objectives and geographical locations require different modelling strategies and types of models, e.g. groundwater in the Netherlands, reservoirs in Norway, and canals in Spain, BINGO does not aim at model intercomparison experiments. However, as shown in D3.1, there are of course multiple links between sites. Generally, BINGO researcher and stakeholder work hand in hand as a team to solve problems all together. Thus, D3.3 also serves as an overview for site researchers and stakeholders. Based on different modelling strategies, D3.3 helps to develop new ideas and the transfer of algorithms among sites within the remaining 30 months of BINGO.

D3.3 has shown that there often are obstacles for modelers, such as data availability and quality, model instabilities and failures, and data post-processing. However, D3.3 also shows that the model performance can generally be considered as satisfactory, given the wide variety of model developments and applications. For example, the Portuguese site features four different modelling domains with different objectives, which of course already provides a wide range of model performances, which are difficult to relate to each other.

The next step in WP3 is dedicated to scenario modelling and also to diminish model performance problems that came to light during the model performance evaluation, e.g. in Norway or Cyprus. Future scenario calculations will consider climate and water/land use impacts for the next decade. Climate ensembles have already been created by WP2 in 12 km and hourly resolution, which will especially help to analyze long term trends and droughts. As for floods, spatial and temporal resolution will be too coarse to realistically represent these events. Here, WP2 will provide extremal episodes with high resolution, which can be nested in the long-term time series modelling. Furthermore, water/land-use changes can have drastic effects on water availability and thus cause water scarcity. These effects can overlay climate change induced water scarcity and thus, needs to be considered as well. Here, D.3.2 provides input for the hydrological models in terms of modified land use maps and water uses. By switching new water/land-use data on and off, it will be possible to quantify effects of climate and water/land-use changes. Finally, data from BINGO field campaigns will help to further fill data gaps and improve model process algorithms and performances. For example, this holds true for soil moisture processes at the German site, transpiration processes at the Dutch site, precipitation patterns at the Norwegian site, ephemeral runoff at the Cypriote cite, and canal flow at the Spanish site.

The next Deliverable 3.4, which will be submitted by month 36, will focus on the application of decadal predictions (2015-2024), land and water use changes and extremal episodes. It will also include a detailed on the similarities and interactions among sites.