

A combined hydrologic and hydraulic modeling approach for testing efficiency of structural flood control measures

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Abstract The necessity of estimating the degree and spatial extent of positive impacts with regard to protecting communities and properties through potential flood control projects can be considered one of the main reasons for performing flood modeling. This paper presents an overall systematic approach based on the simulation of some extreme event conditions, using a hydrological model to generate the resulting river flows and then using a hydraulic modeling exercise to decide upon floodplain evolution in the case-study area, Bostanlı river basin, which has been under the threat of flooding for many years. The potential serviceability of the planned Bostanlı Dam in the study area was examined by using the HEC-HMS and HEC-RAS modeling tools, both integrated with GIS functions for spatial operations. The results indicate that the dam construction as planned would have a somewhat positive impact as a potential flood control measure, since it seems to decrease the flood peaks of 68.9 and 158.7 m³/s (that would potentially be generated by 100- and 500-year storm events under current conditions) to 65.5 and 150.7 m³/s (when the dam is in operation), respectively. However, this seems to contribute little to the overall flood mitigation performance in the basin.

Keywords Bostanlı · Hydrologic modeling · Hydraulic modeling · Floods · GIS

1 Introduction

Flood hydrographs, flood peak estimation and floodplain determination have always been significant factors in flood-related analyses in order to have accurate flood estimations that are required for many design- or management-related studies. Beyond its use in several hydrologic and hydraulic analyses, flood estimation is essential for saving human lives and protecting properties (Olivera and Maidment 2000). Most widespread uses of hydro-spatial

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modeling in river basins under flood risk have focused on developing flood hydrographs and corresponding floodplain areas for such specific purposes as determining the effects of urbanization changes on flow regimes, evaluating potential flood mitigation projects, and performing quasi-unsteady or full-unsteady flow analyses. Indeed, it has become possible to obtain full-discharge hydrographs by using watershed modeling and then to translate them through the watershed, paying special attention to the effects of timing, tributary inflows, reservoirs, channel modifications, and other flood mitigation components (Maidment and Djokic 2000).

Several attempts have been previously made to establish flood hydrographs and identify floodplain areas through the use of some relevant techniques, especially GIS-supported hydrologic and hydraulic modeling. Olivera and Maidment (1999) constructed flood hydrographs by using the HEC-HMS model; they then obtained corresponding flood maps from the HEC-RAS model to be used for the design of highway drainage facilities. Andrysiak and Maidment (2000) carried out an application project to determine floodplain areas in a basin in the United States by using some specific floodplain determination and visualization tools. Anderson (2000) carried out floodplain delineation analyses using ArcView GIS software and some modified scripts along with the HEC-HMS hydrologic model, the HEC-RAS hydraulic model, and HEC-GeoRAS software extension. Alamilla et al. (2001) used the HEC-1 and the HEC-RAS models in combination with GIS to analyze the direct effects of urbanization, allowing for the suitable planning and controlled development of a watershed in the United States. Lastra et al. (2008) compared a hydrological method, which was followed by hydraulic modeling for flood hazard mapping during a case-study in the northern Iberian Peninsula, a method quite similar to the one presented in this paper, to geological-geomorphological methods to identify the differences between the estimates of flooded surface areas, and thus to assess the consistencies of both approaches; they then suggested a combined use to delineate the extent of flooding for varying intervals.

Beside these efforts to evaluate flood maps with a comprehensive modeling approach, numerous studies have concentrated on different components that complement integrated approaches for estimating flood hydrographs. Usul and Yilmaz (2002) and Noorbakhsh et al. (2005) used Clark's instantaneous unit hydrograph (IUH; Clark 1945) technique integrated with GIS in order to develop flood hydrographs. Daniil and Laziridis (2005) estimated design discharges by using SCS-Type I design storms in the HEC-HMS model. Cleveland et al. (2007) focused on estimating unit hydrograph timing parameters (time of concentration and time to peak) from digital elevation models (DEM) that might later be utilized in any conventional hydrologic model. There are also studies that focused on the curve number estimation as a parameter, with the greatest uncertainty in hydrologic assessments (McCuen 2002; Schneider and McCuen 2005), and on the principles of sensitivity analysis approaches that may potentially be used in exploring the sensitivity of resulting river flows to this parameter (Saltelli et al. 2004; Saltelli 2006; Lastra et al. 2008).

As in some of the above examples, flood analyses become more effective by first transferring hydrologic model outputs into a hydraulic model and then final outputs into a GIS environment. While existing hydrologic models were mostly used earlier to provide unidirectional data flows to the model rather than a direct data exchange, today, there exist many examples of greatly improved integration between the model and GIS, making it possible to develop shared procedures for handling data, preparing inputs to the model and displaying outputs through the use of automated stepwise procedures within the GIS environment.

The objective of this study is to develop hydrologic and hydraulic models for the Bostanlı Basin in Izmir, Turkey, which has experienced several flood threats for many years. In order to cope with floods and decrease any further damage, local authorities have designed a set of measures which have aimed to improve stream conveyance capacities by straightening reaches, lining channels and building hydraulic structures, including a new dam. These measures in correspondence with other program objectives of the authorities seem to work well when finalized, yet they still need to be re-examined mainly because of the urban sprawl from the coastal region to the mountains of the basin. For assessing potential impacts of the foreseen measures on flood control and giving proper research-based notice to the authorities, a hydrologic modeling approach, through the use of the HEC-HMS hydrologic model (HEC 2001a), and a floodplain determination technique, achieved by employing the HEC-RAS hydraulic model, were selected in the study. This paper presents an overall systematic approach based on the approximation of selected extreme event conditions, hydrological modeling to generate the resulting river flows from these events, and hydraulic modeling to simulate the flow conditions in stream channels, as well as the modeling of floodplain evolution without further mention of the detailed analytical procedures that relate to individual research components.

2 Study area

Bostanlı Basin is roughly located between the coordinates of $38^{\circ}27'27''\text{N}$ – $27^{\circ}05'32''\text{E}$ and $38^{\circ}33'49''\text{N}$ – $27^{\circ}10'14''\text{E}$ inside the municipal borders of the city of Izmir in the west of Turkey (Fig. 1a, b). The total basin area is about 29.6 km^2 , and the overall region is partially urbanized, with more intensive settlements especially in its southern parts. The main stream which originates at an elevation of 997 m. and ends at the coast of Izmir Bay flows in the northwest–southeast direction along a total length of 14.28 km. The basin receives most of the yearly rainfall between November and March, while rainfall in summer is fairly rare (Tempo 2000). The most vigorous flood, which occurred on the 4th of November, 1995, caused 69 deaths in addition to physical damages in the basin area. The extreme precipitation of 108.0 mm observed at the Izmir meteorological station on this date was the maximum within the observation period 1986–1995, a period with the second largest periodical maximum when compared to the periods 1976–1985 and 1996–2005, which had the observed maximum rainfall depths of 134.1 and 92.0 mm, respectively (DMI 2006).

For supplying drinking water to its vicinity and controlling potential future floods in the basin downstream, the construction of a rock-fill dam is intended by the Izmir General Directorate of Water and Wastewater Management (IZSU; Fig. 1c). The planned dam with the crest elevation of 180 m and the crest width of 10 m will receive water drained from an area of 16 km^2 . The reservoir volume is estimated as 2.1 million m^3 at the normal water level, while the maximum allowable storage in the reservoir is 3.6 million m^3 . After it is commissioned, the dam is expected to supply 2.97 million m^3 ($0.094 \text{ m}^3/\text{s}$) of drinking water yearly on average to the Karsiyaka District of the city Izmir.

3 Methodology and data

For designing a hydraulic system from scratch or assessing the performance of an existing system as in this study, the selection of a design storm becomes a priority. Many recently

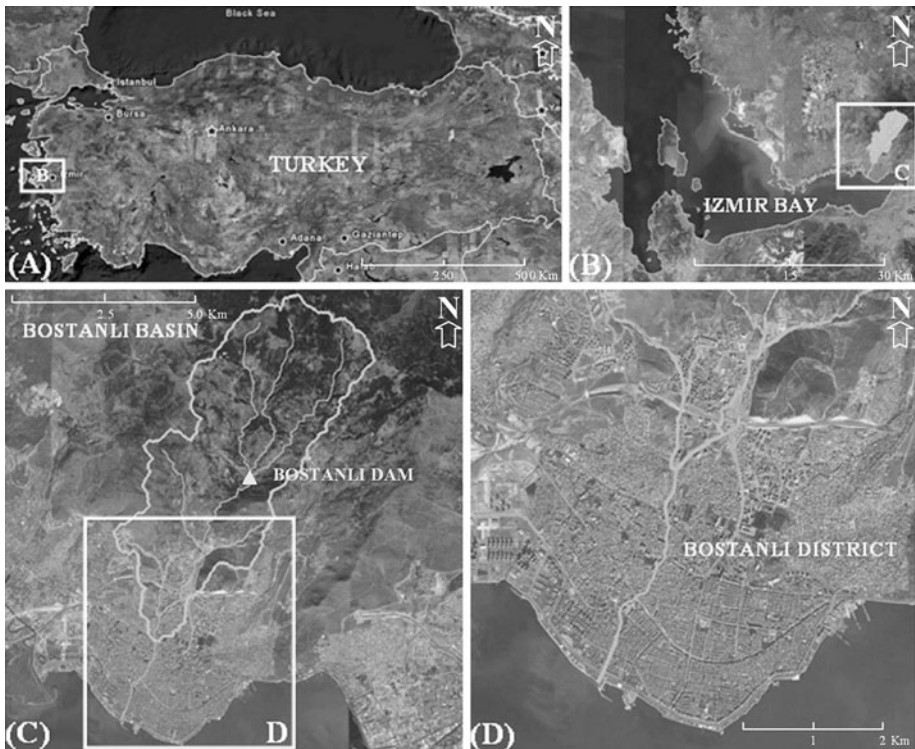


Fig. 1 **a** Location of the study area in Turkey and **b** along the coasts of Izmir Bay, **c** Bostanli River and the upper basin area considered for the hydrological study, and **d** populated basin downstream with the flood risk

developed methods with unsteady flow analysis require reliable estimates of a design hydrograph that is used as an input for rainfall-runoff modeling to represent a selected design storm with the recurrence interval “ T ”. The instantaneous intensity method (Chow et al. 1988) is one of the methods available for this purpose. In this method, a pair of curves which are defined by the instantaneous intensity functions, $f(t_a)$ and $f(t_b)$, are assumed to fit the precipitation intensities before and after the peak of the instantaneous intensity hyetograph. The precipitation depth value for a selected storm duration period “ T_d ” around the peak of the storm equals the value given by the intensity–duration–frequency (IDF) curve and the total amount of rainfall within time T_d is given by the area under the two curves defined by the intensity functions above. If IDF function is known, equations for computing the hyetograph of the selected design storm can be obtained for the variation in intensity with time. The computation of the hyetograph with a time interval of Δt needs the selection of a smaller (or the smallest possible) computation interval, Δt_i , to increase the accuracy and precision of the finally computed hyetograph.

Besides the design storm hyetograph, hydrological simulations in the current study also require the generation of unit hydrographs for a number of subbasins carefully identified over the entire basin area to be considered in the hydrological study. In this step of preparing inputs to the hydrologic simulation, the selection of a proper synthetic unit hydrograph (UH) technique becomes very important, especially when there is no way of utilizing an actual unit hydrograph generated from observations. Clark’s UH method

proves to be efficient here as a useful technique amongst others (e.g., Snyder 1938, Clark 1945, Nash 1957, and U.S. Soil Conservation Service (SCS) 1985) due to the convenience of computing its parameters through a series of much more rapid and accurate analyses with the help of a GIS. In this method, the shape of the unit hydrograph is determined by travel time through the basin as well as basin shape and storage characteristics, and a time–area diagram that is generated for the basin from the measurements of the areas between alternate isochrones (i.e. contours of fixed travel times) is combined with a linear reservoir at the basin outlet (Clark 1945). The required translation hydrograph can conveniently be derived from the time–area relationship by determining the runoff volume generated by the instantaneous application of unit excess rainfall (or unit direct runoff) for each time–area increment over the basin, then dividing the volume by the time interval between the isochrones and plotting the results for all the increments against the travel time represented by the isochrones. Naturally, a difference exists between the time–area translation hydrograph and the basin outflow hydrograph simply because of the fact that the resulting hydrograph is attenuated under the effect of basin storage. In order to represent this attenuation, the Muskingum storage coefficient, R , is used in Clark's method. R coefficient is either computed from an observed storm hydrograph if available or estimated from basin characteristics by using such formulas as suggested by Clark or Linsley (Linsley et al. 1998; NOAA 2005). After certain unit adjustment, IUH is obtained by means of linear reservoir routing, and the final unit hydrograph is then derived by averaging the same two IUHs that are apart from each other on the time axis by the time step chosen for the time–area relationship (Wanielista et al. 1997).

HEC-HMS, which was selected in the study as the main hydrological modeling platform, is a modeling tool that uses, together with other inputs, the design storm hydrograph and the final unit hydrographs generated for the subbasins to simulate the rainfall-runoff response of a watershed by representing the entire watershed as an interconnected system of hydrologic and hydraulic components, including subbasins, streams, reservoirs, diversions, etc. It is a model that was designed to work in a compatible way with various GIS data and to facilitate the use of GIS technology in hydraulic engineering. In the HEC-HMS model, each project includes a basin model, meteorologic model and control specifications as main project components. Precipitation is described by using either historical data (mainly for calibration and verification of model parameters if available) or the data generated for design (or hypothetical) storms to evaluate the performance of proposed designs or regulations (HEC 2001a). A similar optional use together with historical or hypothetical data, depending on the purpose, applies to the evapotranspiration. A user-specified unit hydrograph, generated as explained above, can then be used in the model to generate runoff. In ungauged basins without runoff observations, there is no way of calibrating the model parameters and validating the simulated results from the model. In such cases, the physically based routing models will generally be easier than the empirical models to set up and apply with some confidence if observed data are not available. As the lack of historical flow records does not allow users to perform a proper calibration, only best estimates can be generated (for the parameters that are required for model operation) through a set of additional computations. In this case, the sensitivity of the results to the parameter uncertainty should be investigated through a set of sensitivity analyses to ensure that the model findings would not greatly differ if the actual parameter values were somehow different than their best estimates used in operating the model.

HEC-GeoHMS, on the other hand, is an extension that was developed to be an assistant for engineers and modelers in deriving geospatial data by using GIS (HEC 2000). The visualization of spatial information and documentation of watershed characteristics are

possible by using HEC-GeoHMS. The extension allows the user to perform spatial analysis, delineate subbasins and streams, construct inputs to hydrologic models, prepare reports, and generate such basic elements as background map files, raster parameter files, and lumped or distributed basin models that can be directly transferred into the HEC-HMS model (HEC 2000; Bedient and Huber 2002).

Outputs from the HEC-HMS model can be utilized as inputs to the HEC-RAS hydraulic model, which has a graphical user interface that makes it possible to enter, edit, and display data and graphics easily. This capability allows users to better visualize the stream and its condition as well as the three-dimensional plotting of the stream geometry (Tate 1999). In addition to hydraulic design features, HEC-RAS has several hydraulic calculation capabilities, including flow water surface profile computations, steady or unsteady flow simulations, and movable boundary sediment transport computations (HEC 2001b). The main components of the data required for any hydraulic modeling in HEC-RAS are geometric data that define physical and topographical features, flow data that can be either designated from observations or extracted from the outputs of a hydrologic model, sediment data when sediment transport modeling is desired, and hydraulic design data supplied for hydraulic structures. All physical and topographical characteristics that define a system comprising river reaches and hydraulic structures (bridges, culverts, levees, etc.) are entered into HEC-RAS through a geometric data editor. Cross sections are to be provided at the representative nodes where changes occur in discharge, slope, shape, and roughness; at locations where levees begin and end; and at hydraulic structures such as bridges, culverts, and weirs together with their required information that includes reach and station identifiers, coordinate information, downstream reach lengths, Manning's roughness coefficients, main channel bank stations, and contraction and expansion coefficients. Flow data are used pursuant to the preferred computational option, steady or unsteady, and are defined either manually or imported directly from the HEC-HMS run. HEC-GeoRAS supports the use of geospatial data in HEC-RAS as a specific extension of GIS. It was mainly developed to assist users with less GIS knowledge in constituting an HEC-RAS import file, where geometric attribute data from an existing digital terrain model (DTM) and complementary data are available. The HEC-RAS model results can be transferred into HEC-GeoRAS in order to obtain more meaningful outputs through available visualization techniques (HEC 2002).

As part of the model-specific requirements, the fundamental data utilized in this study mainly include soil and land use information, relevant characteristics of the installed and/or planned hydraulic structures in the basin area, and a DTM. Two sections of corresponding 1/25,000 scaled soil and land use maps were supplied by the General Directorate of Rural Services (KHGM) in digital format, while the remaining two underwent an on-screen digitizing process to extract necessary digital information. While the positional and attribute information about the weirs was compiled from relevant publications of IZSU (2004) and the required characteristics of the projected Bostanlı Dam were obtained from the Regional Directorate of State Hydraulic Works (DSI) located in Izmir, a large number of spatial data were also collected from the GPS and in situ geometric measurements compiled during the field surveys for hydraulic structures. Digital terrain data required for the hydrologic modeling operations in the upper basin sections were extracted in the form of elevation contours at 5 m intervals from the spatial database, which was provided by IZSU. A detailed DEM was then generated in 5 m × 5 m grid resolution by using the specialized tool of the GIS software for creating hydrologically correct terrain surfaces from the topographic data, which also included elevation contour lines and a known stream network. In the study, such a tool, which is based on a discretized thin plate spline technique, was

preferred to other local interpolation methods (e.g. inverse distance weighting) or global methods (e.g. kriging), as it basically employs an interpolation method specifically designed for the creation of hydrologically correct DEMs (Hutchinson 1988, 1989). On the other hand, the terrain information required for the hydraulic computations in HEC-RAS was compiled from the same IZSU database but in a TIN model structure that represents both the channel bottom geometry and the adjacent floodplain area. This terrain TIN is used in combination with the water surface TIN generated by the hydraulic model to compute water depths and includes basic terrain types of the lower basin sections considered in floodplain determination as well as the buildings and other urban features of different heights. The data of the channel cross-section geometry are generally extracted from the TIN and then revised by a series of minor corrections based on the in situ measurements. As the channel geometry normally dictates the flow in river systems, the TIN model utilized prior to the hydraulic computations should be precise enough to allow the most accurate determination of the channel geometry and the surrounding landforms.

Although most of the GIS operations conducted by HEC-GeoHMS and HEC-GeoRAS extensions were mainly performed in Arcview GIS software developed by ESRI (Environmental Systems Research Institute, Inc., US), a number of the preprocessing operations, including the raw DEM generation, were implemented in the ArcGIS system.

4 Application

4.1 Deriving design storms and unit hydrographs

The computation of instantaneous intensity hyetographs with such predetermined parameters as recurrence interval (T), storm duration (T_d), time interval of hyetograph (Δt), and computation interval (Δt_i) was achieved through the use of a computer program that was developed in the Visual Basic programming language (Lopcu 2004). In addition to instantaneous intensity functions [$f(t_a)$ and $f(t_b)$] and minimum/maximum intensities, the program was enabled to calculate incremental precipitation values and ordinates of the hyetograph by using the minimum computation time interval. Necessary input files used for drawing intensity-duration functions and hyetographs with the time interval, Δt , were then obtained in this wise. Figuer 2 shows instantaneous intensity hyetographs that correspond to the precipitations with 100- and 500-year recurrence intervals, each with 6-h storm durations. The results shown in the figures were obtained for a later use in the meteorologic component of the HEC-HMS model.

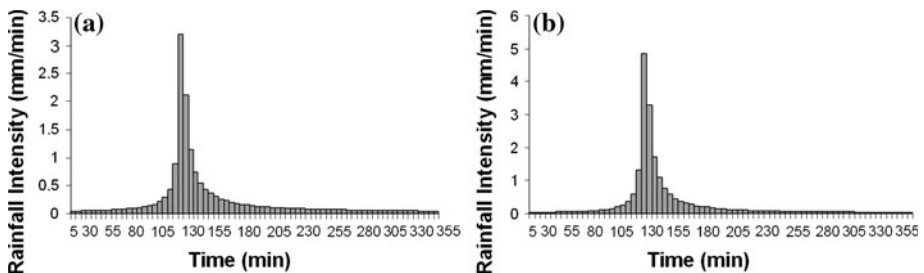


Fig. 2 Instantaneous intensity hyetographs **a** for $T = 100$ years, and **b** for $T = 500$ years

For generating Clark UHs in the study, the basin DEM with a spatial resolution of 5 m × 5 m was used as a basis for determining the time–area diagram for the Bostanli Basin. Flow directions and travel distances of flow (flow lengths) from each cell to the basin outlet were first calculated from this model (Fig. 3), and travel distances were then converted to travel time values in order to obtain the travel time histogram for the minimum possible time interval (Fig. 4a). After calculating storage attenuation coefficients (R values), the instantaneous hydrographs for the subbasins were developed by routing the time-area diagrams through the linear reservoir (Onuşluel 2005). The final unit hydrographs with 1 min time-steps were obtained by shifting the instantaneous hydrographs (IUHs) by 1 min for all the subbasins and then averaging the two hydrographs, which are apart from each other by 1 min, for each subbasin (Fig. 4b).

4.2 Hydrological modeling

In the study, hydrological simulations mainly focused on the upstream part of the river basin in order to provide inputs to the follow-up hydraulic model for the basin downstream, which is highly urbanized and thus needs more attention in regard to the detrimental impact of potential flooding (Fig. 1c, d). Prior to any hydrologic modeling, the pre-processing of some of the spatial data in HEC-GeoHMS was needed eventually to generate the lumped basin model and the schematic background map files. These two files summarize

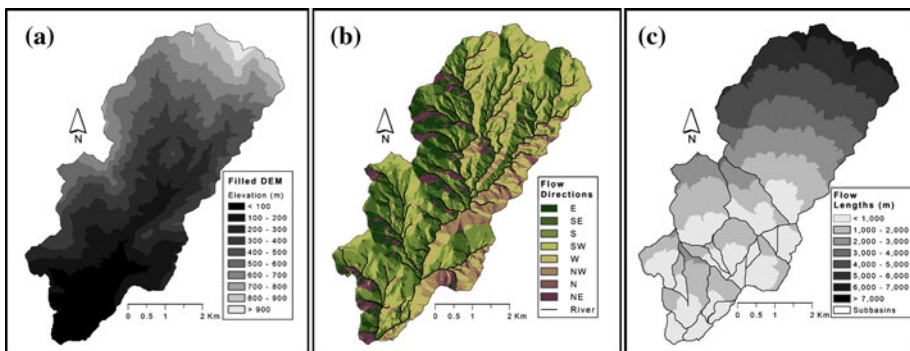


Fig. 3 a Depressionless DEM of the basin, b flow directions, and c travel distances of the flow (flow lengths) in the basin

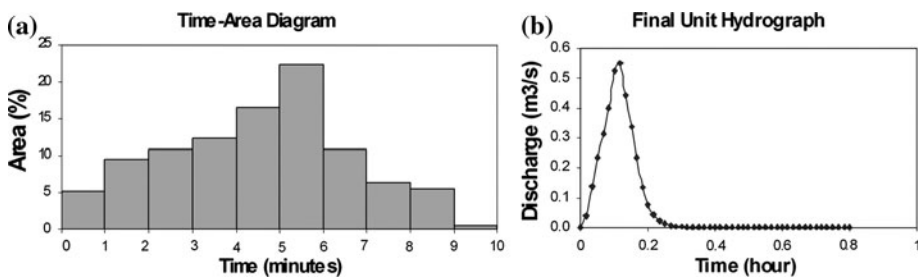


Fig. 4 a Time–area diagram converted from the travel time histogram, and b final unit hydrograph for one of the subbasins

some physical characteristics of basin features (e.g. river slopes, subbasin centroids, longest flow paths, centroidal flow paths as well as the locations and characteristics of hydraulic structures) in each subbasin and are also necessary for providing direct data entry into the model (Fig. 5). Afterward, hydrologic modeling was performed on the HEC-HMS v.2.2.2 model by taking the design storms and the resulting hyetographs into consideration. Here, the SCS CN method was selected for the computation of loss rates in all subbasins, and unique soil group-land cover polygons were first identified over the whole basin area in order to obtain averaged representative CNs for single subbasins. For this operation, the primary soil groups were defined based on a soil map showing primary soil characteristics (i.e. texture, depth and stoniness), and a vegetation map comprised of predetermined NDVI (normalized difference vegetation index) values was utilized to represent the vegetation cover inside the area. Representative CNs for the subbasins were finally obtained by using areal CN averages as the local statistics and were manually assigned to each of the subbasins during the model set up.

Later, Clark’s synthetic hydrographs individually derived for all subbasins outside the model were first transferred into the model and were then successively assigned to the subbasins. Base flow was assumed to be zero for all subbasins in the study. For routing processes in stream reaches, the Muskingum-Cunge Method, with its parameters based on

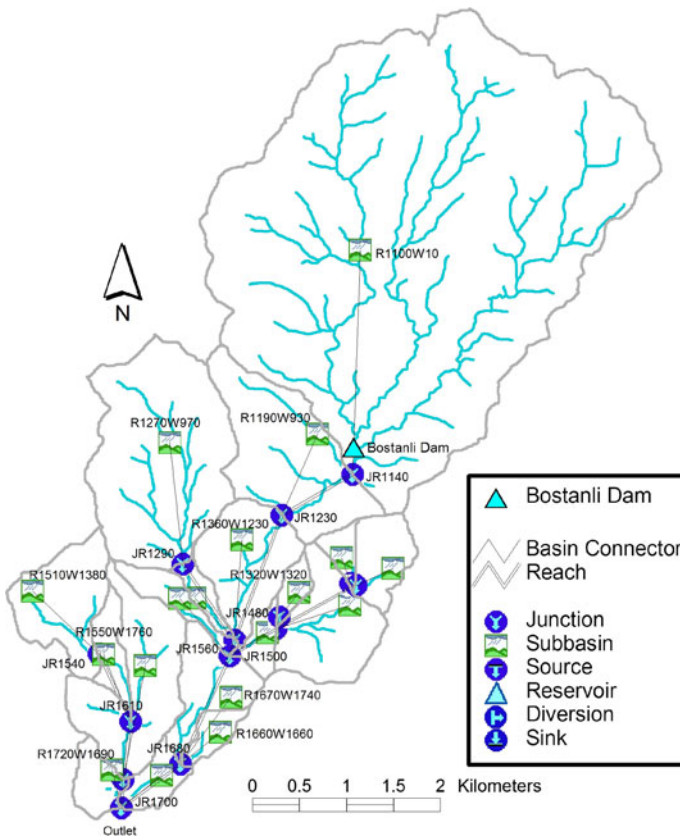


Fig. 5 Basin schematic generated with HEC-GeoHMS

the stream geometry, was utilized through the calculation of necessary parameter values with some relevant GIS operations. For flood routing in the Bostanli Dam, “storage vs. elevation” option was selected from the set of reservoir routing options available in the model.

In constructing the meteorologic model for the basin, a number of imaginary rain gages were first defined for the subbasins with the assumption that each would simply represent the meteorologic conditions in a single subbasin. Rainfall values obtained from the instantaneous intensity method were then literally assigned to (or defined for) all the gages as if they were the records from an observed storm event. With the selection of a suitable simulation time and a computational time interval of 5 min., all necessary preparations for the modeling phase were finalized and the model was run for both (a) the 100-year design storm, and (b) the 500-year design storm, initially considering the existing basin conditions excluding the dam and then the design case regarding the dam operation.

As the case-study examined an ungauged basin without an applicable record of flow stations, it was not possible to perform any calibration of the model parameters that also included CN or verify any output from a simulation with the best parameter estimates. It is suggested in a number of relevant literatures to explore the sensitivity of the outputs in such cases to the CN parameter, as it is the most uncertain parameter in hydrologic computations (Schneider and McCuen 2005, Lastra et al. 2008). The sensitivity analysis for CN was performed by generating 250 new sets of random CNs (by means of Monte-Carlo simulations), that varied between 66 and 82, separately for each design storm case (i.e. with recurrence intervals of 100 and 500 years), and assigning each set sequentially to the subbasins for each test run in HEC-HMS. The CN range for Monte-Carlo simulations was basically determined by considering the hydrologic soil groups (mostly types C and D) and the dominating land uses (largely poor, woody vegetation cover and partly meadows in upper basin sections). Random sampling through the Monte-Carlo method and the test runs with the sampled CN parameters provided 250 hydrographs for each design case, and 250 peak discharges were acquired from these. Lastra et al. (2008) proposed a method, which is based on a comparison between the mean peak discharge extracted from the test runs and the simulated peak value from the model run with the appointed parameter estimates, to check the relevance and adequacy of the best CN estimates in a model run. In this approach, it is also investigated whether the simulated peak value is well-located between the upper and lower confidence limits, which are computed from the peaks of the test runs for 90 and 95% confidence levels, in order to help assess the eligibility of the estimated parameters used in hydrologic simulations.

4.3 Hydraulic modeling for designating floodplains

For the hydraulic modeling part of the study, the lower part of the Bostanli Basin, which mainly consists of highly urbanized areas, was selected to keep the analyses parallel to those realized in hydrological modeling. The HEC-HMS outputs in the form of computed hydrographs were utilized as inputs to the HEC-RAS model. Other necessary input data including stream centerlines (2D), cross-sectional cut lines (2D), main channel banks and flow path centerlines, and some intermediate data consisting of stream centerlines (3D) and cross-sectional surface lines (3D) were generated by using the processing functions available in HEC-GeoRAS, and all were then combined into a single *RAS GIS Import File*. Owing to the great number of iterations between the cross sections, the model requires more cross sections than the ones already measured and defined on a reach in order to arrive at reliable results. The model serves as a specific tool through the geometric data

editor for interpolating additional cross sections with two options: either within a reach or between the two selected cross sections. By selecting the former option and fixing the length between the cross-sections at 1 m, a number of new cross sections were developed in the study (Onuşluel 2005).

Alongside the inputs adapted from the import file, spatial characteristics and attribute information of the hydraulic structures in the model area, i.e. the culverts, bridges and weirs along the Bostanli River and its branches, and appropriate Manning’s n roughness values for the channels and neighboring overland areas were additionally described in HEC-RAS to obtain a complete geometry model for the Bostanli River.

Prior to any hydraulic computation, two scenarios (with and without the dam operation) were basically defined because of the need to assess any recovery with the dam operation in regard to the extent and depth of potential flooding. These were structured in such a way that would eventually allow any positive dam impact on floodplains under the storm conditions for the two considered return periods, 100 and 500 years, to be evident. The hydraulic model was then run repeatedly for each of the scenario test cases.

5 Results and discussion

For checking the sensitivity of the simulated flows to potential discrepancies in the uncertain model parameters in the study, hence for testing the reliability of the results based on the best parameter estimates, sensitivity analysis of the CN parameter was applied separately for the two return periods. Table 1 presents some statistical information about the peak flows generated in the sensitivity runs as well as the peak flows simulated with the best parameter estimates to demonstrate whether the simulated peaks based on the pre-determined parameter values are different from or sufficiently close to the mean peak discharges calculated from the sensitivity runs. As can be seen from the table, the peak flows simulated in the study with the predetermined parameter estimates are very close to the mean peak values computed from the hydrographs of the sensitivity simulations (only with minor positive differences of around 1%) and are also inside the 95% confidence limits identified both for the return periods of 100 and 500 years.

Table 1 Comparisons of simulated peak flows to the peaks from the sensitivity runs

Model output	Return period	
	100-year	500-year
Simulated peak flow (m ³ /s)	68.9	158.7
Statistical characteristics of peak flows from sensitivity runs (m ³ /s)		
Mean	68.2	157.7
SD	9.2	11.6
Maximum	90.4	187.8
Minimum	42.8	137.5
Confidence limits (90% confidence level)		
Upper	69.1	158.9
Lower	67.2	156.5
Confidence limits (95% confidence level)		
Upper	69.3	159.1
Lower	67.0	156.2

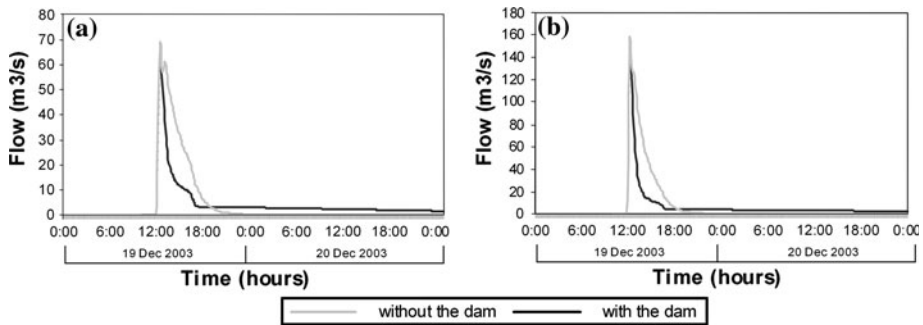


Fig. 6 Output hydrographs excluding the dam and considering the dam operation **a** for the 100-year and **b** for the 500-year design storms

The runoff hydrographs, resulting from the hydrologic simulations that differ with regard to each scenario, provide a clear indication of the dam's influence from a flood control point of view. Figure 6a shows the hydrographs modeled for the upper basin outlet in the two initial runs with the 100-year design storm consideration, first by excluding the dam's operation, and then assuming that the dam was already commissioned. As can be seen in both graphs, Bostanli Dam would potentially have an attenuation effect of decreasing the flood peak of $68.9 \text{ m}^3/\text{s}$ (under the existing conditions with no dam) to $65.5 \text{ m}^3/\text{s}$ (when the dam is in operation). Figure 6b gives similar results that were obtained again at the upper basin outlet but correspond to the scenario tests with the 500-year design storm. The positive impact of the planned dam can still be observed as a way to decrease the flood peak from a level of $158.7 \text{ m}^3/\text{s}$ to $150.7 \text{ m}^3/\text{s}$. It becomes evident from the results generated for both design storms that the planned Bostanli Dam fulfils the expected service of decreasing flood peaks up to a certain extent, but the overall attenuation effect still seems to be rather limited, especially when compared to the desired mitigation impact of a major flood control reservoir as such. This may best be explained by different upstream and downstream characteristics of the modeled upper basin area. The highly urbanized nature of the downstream parts seems to somehow hide the effectiveness of the dam and dominate the flow conditions at the outlet.

In regard to the extent and depth of flooding in the lower basin, which covers a major part of the Bostanli District with considerable housing and a dense population (Fig. 1d), flooding remains a significant threat both along the left and right overbank areas, especially where the surface elevations, mainly for streets, open spaces and small number of agricultural parcels, are remarkably low. For both the storm conditions with 100- and 500-year intervals, the dam appears to have little influence on decreasing the spatial extent of potential floodplains by almost 3.5% (Figs. 7a, b, 8a, b). The dam operation would, however, have a secondary impact by lowering the depths of flooding, especially in the built-up area (Figs. 7c, 8c). This positive influence is much more appreciable in the case of the 500-year storm, with a wider areal coverage of considerable improvement.

6 Conclusions

Computer-aided numerical modeling provides a robust tool for evaluating the efficiency of a hydraulic structure or an overall system in basin-wide studies. GIS support in such

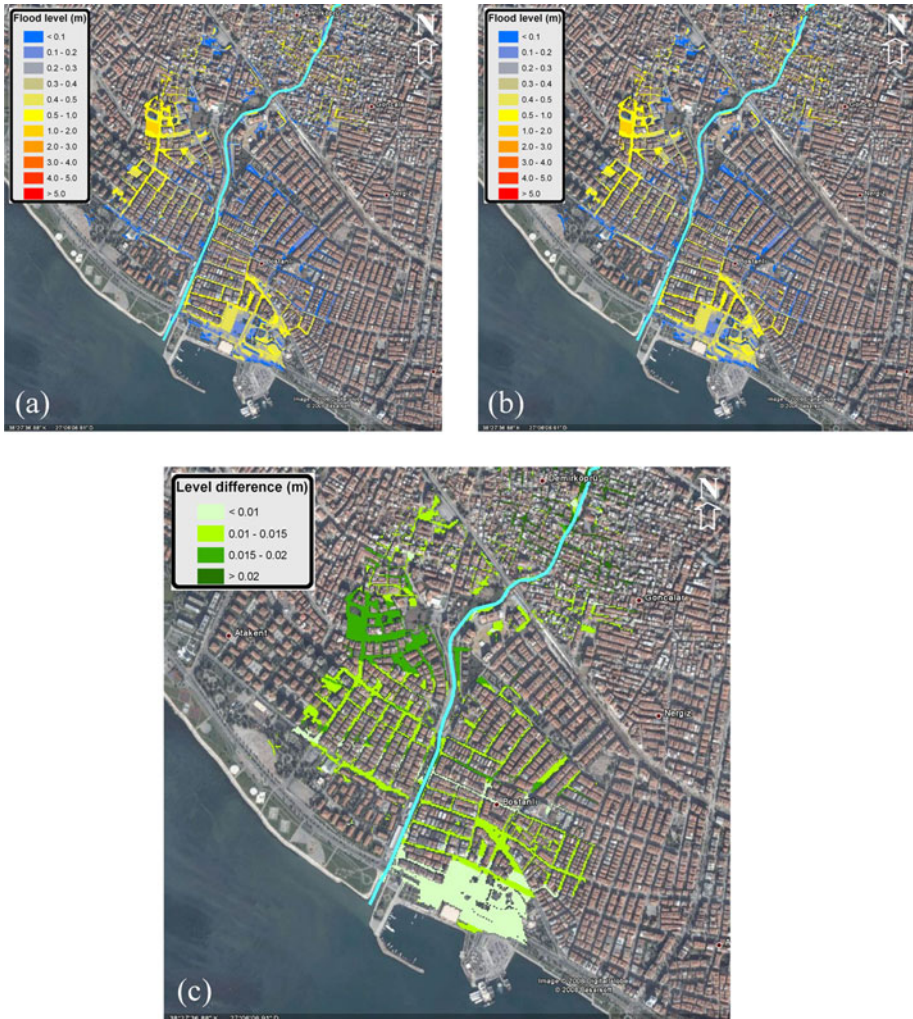


Fig. 7 Modeled floodplains for the 100-year design storm **a** under existing conditions without the dam, **b** considering the dam, and **c** the expected rates of change in flood depths due to the dam operation

models provides convenience and remarkable value to the performed analyses by mostly eliminating errors that may result from manual data entry and facilitating the generation of input files, which are used to define the connections between different models and model elements. The credibility and accuracy of any study targeting a performance evaluation strongly rely on the physical suitability and scientific soundness of the methodological approaches employed in corresponding exploratory activities. The simulation results from detailed and comprehensive modeling studies that combine hydrologic and hydraulic analyses, as in the case of the Bostanli study, can in this regard be confidently considered in hydraulic design or control applications for candidate or existing structures within the basins under examination. Additionally, a set of integrated controls can also be generated to observe whether basin-wide planned projects or their individual components, e.g. for

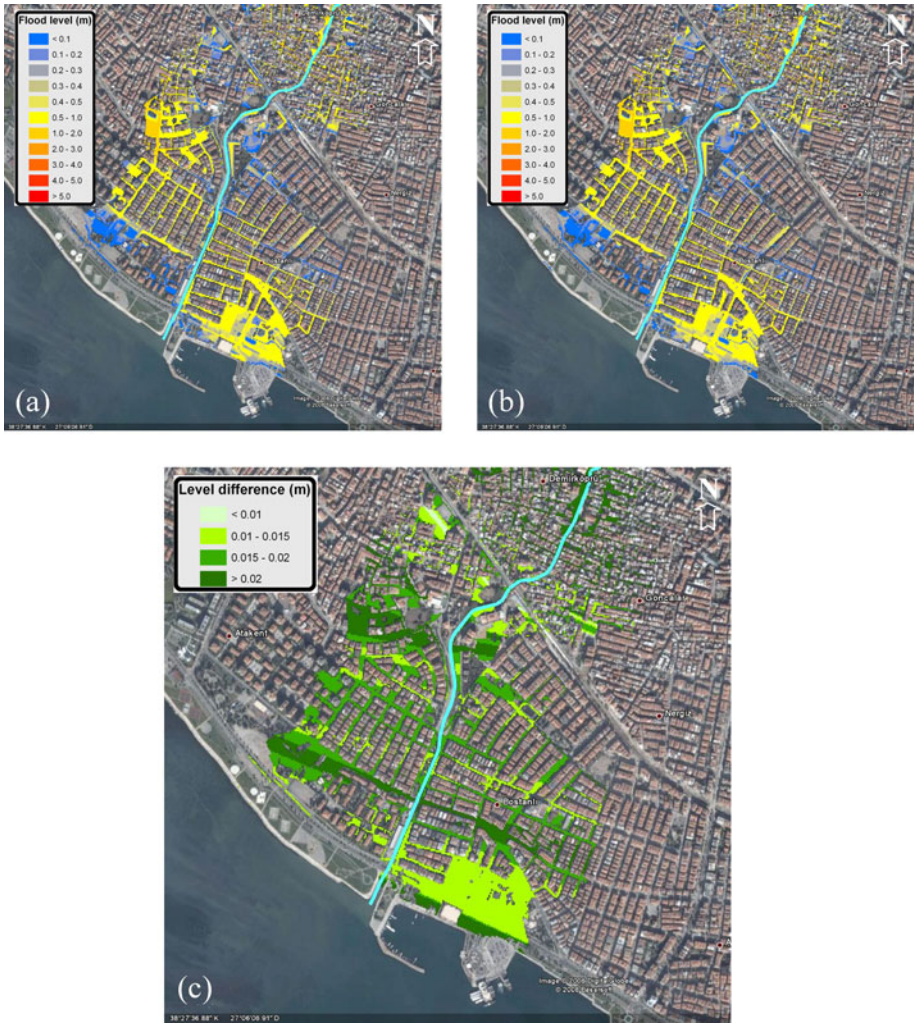


Fig. 8 Modeled floodplains for the 500-year design storm **a** under existing conditions without the dam, **b** considering the dam, and **c** the expected rates of change in flood depths due to the dam operation

flood mitigation purposes, would be as effective as was initially envisaged or would need further improvements/modifications to function properly together with other available hydraulic structures in a complete system.

In order to cope with the serious flooding problem in the densely populated downstream part of the Bostanli River, local authorities have performed a number of critical actions, mainly by improving channel capacities and building new hydraulic structures for flood prevention. As these actions were already completed and there were no utilizable records about the hydraulic characteristics of the poorly functioning systems of the past, this current study focused on the identification of any improvement that is expected to arise with the construction of the Bostanli Dam in the upper basin area. From an overview of the modeling results in the study, it becomes evident that Bostanli Dam would somewhat serve

to decrease flood peaks at the basin outlet; however, it may not suffice alone for complete protection against harmful flood impacts on human life and properties in case of a severe event; the dam, after it is commissioned, will contribute little to the overall flood mitigation performance in the basin together with the other hydraulic structures downstream.

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