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Urban Flood modelling and Floodplain Mapping using ArcGIS, HEC-HMS and HEC-RAS in Abidjan city, Côte D'Ivoire – West Africa: Case study of the watershed of Bonoumin - Rivièra Palmeraie

Master of Science Thesis
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Abstract

Since 2009, Côte D'Ivoire has been experiencing floods events each rainy season in general and particularly the capital city Abidjan. According to the Office for the Coordination of Humanitarian Affairs (OCHA, 2014), from 2009 to 2014, an average of 13 flood-related deaths per year were recorded in Abidjan. The same report has shown that 26% of the Abidjan District is at risk of flooding (Danumah, 2016).

Flood modeling and simulation assist in the prediction of the hazard for better flood preparedness and thus reduce flood damages. The study had simulated flood occurrence at the watershed of Bonoumin – Rivièra Palmeraie which is an urban area in the capital city Abidjan, south Côte D'Ivoire. Digital elevation model (DEM) for this area was processed in the ArcGIS 10.3 environment (HEC-GeoHMS) using terrain preprocessing tools to delineate the basin, sub-basins, and stream network. And then results from the terrain preprocessing were used to extract the hydrologic parameters of the river basin. These hydrologic parameters were used in the estimation of streamflow runoff in HEC-HMS. The discharge generated by HEC-HMS was used in HEC-RAS for hydraulic simulations whose purpose was to show the conveyance of stormwater through the canal of Rue minister and also flood wage propagation for further floodplain delineation in HEC-GeoRAS and ArcGIS.

Chapter 1: Purpose and Significance of the Study

1.0 Background

Flooding occurs when water accumulates in places that are not normally submerged. In urban areas, they are usually the consequence of extreme rainfall, which creates an excess of runoff that is above the capacity of the drainage systems (Adetunji et al., 2013) or it occurs when an extreme event coincides with a vulnerable physical and socio-economic environment, surpassing society's ability to control or survive the consequences (Di Baldassarre et al., 2012). With more than half of the global population living in urban areas, these phenomena are becoming an increasing public concern (Nasiri et al., 2013). Their occurrences and amplitudes will increase due to climate change and the increasing urban population (Intergovernmental Panel on Climate Change, 2007).

Urban flooding is a serious and growing development challenge. Against the backdrop of demographic growth, urbanization trends and climate changes, the causes of floods are shifting and their impacts are accelerating. Urban floods are one of the most common and widely distributed natural risks for life and property worldwide. It has been reported that in the last decade, urban floods have impacted most parts of the world including the USA, Europe, Asia, and Africa. Flooding occurs in the least developed nations as well as in the most developed countries. In the developed world, urban floods are often related to hazards, such as climate change, storm surge, flash floods, and consecutive heavy precipitations. However, in addition to the highlighted causes that prevail in the developed world, flooding in developing countries is also due to the precariousness of the drainage system, the lack of maintenance of the infrastructures and the mismanagement of household wastes.

In Sub-Saharan Africa where about 72% of urban inhabitants live in slums, the consequences of urban flooding could be worsened by the lack of adequate protection infrastructures. Unplanned growth and development in such areas usually results in flooding when the flood protection structures fail under extreme hydrological weather conditions. Studies have reported that West Africa is particularly subject to urban floods. The region was struck in 2007 by heavy precipitation causing widespread regional flooding that affected a total of 792,676 people and caused 210 deaths.

Abidjan, the economic capital of Côte d'Ivoire, offers a prime example of this phenomenon of urban flooding. In that city, every rainy season is characterized by its number of deaths and the extent of its damages. According to the Office for the Coordination of Humanitarian Affairs (OCHA), from 2009 to 2014, an average of 13 flood-related deaths per year were recorded in Abidjan (Kablan et al., 2017). In these conditions, flood vulnerability assessment and forecasting are important not only because flooding inflicts harm to humans, but also for proper urban planning and adaptation to climate change. A good knowledge of flood-prone areas, the level of vulnerability of an urban area to flooding and the socio-economic, environmental and physical factors that play a major role in shaping the hazard, could be an important step toward improving the resilience of growing African cities like Abidjan, Côte d'Ivoire. Although flood hazards are natural phenomena, the vulnerability of an area to flooding is a combination of socio-economic and environmental factors that vary spatially from one place to another.

While the primary cause of flooding is normally heavy rainfall, it is also due to human activities such as land degradation, deforestation of catchment areas, sprawl and increased population density along river banks, poor land use planning, zoning and control of floodplains development, inadequate drainage system particularly in big cities and inadequate management of discharges from rivers' reservoirs (Danumah, 2016). Assessing extreme rainfall, predicting flood risks, mapping flood prone areas and forecast floods are more and more essential to mitigate floods and enable a good living environment to people.

1.1 Problem statement and Justification

Since 2009, Côte D'Ivoire has been experiencing floods events each rainy season in general and Abidjan in particular. According to the Office for the Coordination of Humanitarian Affairs (OCHA, 2014), from 2009 to 2014, an average of 13 flood-related deaths per year were recorded in Abidjan. The same report has shown that 26% of the Abidjan District is at risk of flooding. Indeed, 80,000 people are threatened by flooding in Abidjan with 40,000 at risk in Cocody, 12,500 in Abobo, 10,000 in Adjamé, 9,500 in Yopougon and 8,000 in Attécoubé municipalities. These statistics indicate the extent of damage resulting from flooding in the country and especially in Abidjan. To these lives' losses one should add destruction of infrastructures such as drainage systems, roads and buildings, water bodies' pollution by the conveyance of chemicals.

Abidjan is situated in the south of the country under the tropics and is characterized by important recurrent flooding events. Floods risk in the city is higher in slums due to violent storms (Dongo et al., 2008). Erosion phenomena resulted in landslides and especially flooding occurs in slums, generally settled in high risk areas and protected areas from the urbanization master plan of Abidjan (Dongo et al., 2008).

Information regarding the flooding characteristics and its effect are essential for management of water bodies for decision making in flood management strategies such as construction of flood protection structures, development of flood emergency plan and human settlement planning.

Thus, floods management in slums in particular and in the whole city has become a major concern for researchers interested in urban issues (Dongo et al., 2008).

However, recent scientific work undertaken in the District of Abidjan concentrated on certain factors controlling flood risk such as rainfall risks and uncontrolled urban growth in two municipalities of Abidjan: Attécoubé and Abobo (celestin, 2008). This is piecemeal approach and will not provide solution to the problem of flooding in the District. Other studies (Dongo et al., 2008 and Danumah, 2016) focused on flood analysis in some slums in Yopougon and flood risks assessment and mapping in Abidjan. These studies don't give any idea of the depth and propagation of floods. Ivorian decision makers are still lacking knowledge for appropriate decisions to save lives, infrastructures and economic activities.

With advances in hydrodynamic modelling these days it is possible to model flood extent, depth, duration and even flood propagation in the temporal and spatial dimension. The limitation of the required topographic data for representing the topography of river and floodplain in the modelling is a major problem.

Thus, this research is necessary to fill the gap by developing models for floods modeling and mapping useful in managing floods.

1.2 Research objectives:

From the background information and problem statement the following general and specific research objectives are formulated below.

General objective

To estimate floods along the canal of the "Rue ministre" and identify flood prone areas.

Specific objectives

- ✓ Foster a better understanding of floods in that part of the city
- ✓ To study the flooding characteristics in the catchment
- ✓ To produce maps of floods depth, level and extent for the most hazardous floods event (flood of June 19, 2018)
- ✓ To come up with floods preparedness strategies

1.3 Research questions

The relevant research questions that will be addressed in this research are:

- ✓ How accurate can the observed inundated flood area be simulated by the selected model approach?
- ✓ Why does flooding occur more and more frequently?
- ✓ How to present the extent of flood risk?
- ✓ How does the inundation pattern vary along the river?
- ✓ How to get prepared against flood damages?

1.4 Impacts of urbanization on floods in Abidjan city

Urbanization and growth go hand in hand, and no one can deny that urbanization is essential for socioeconomic transformation, wealth generation, prosperity and development.

Urbanization levels reflect the degree of economic development of a region, but it also changes water cycle in many aspects. Urbanizing is a major land use/cover change trend for most regions of China in recent decades. It modifies hydrological process by affecting runoff generation and concentration which has been research focus currently (Amini et al., 2011; Delgado et al., 2009; O'Donnell et al., 2011; Wu et al., 2007).

This phenomenon is partly caused by the massive departure of people from rural areas to the big cities seeking for a better life. This rapid population growth in large cities inevitably leads to the rapid urbanization of urban neighbourhoods and, as a consequence, an increase in impervious surfaces. Indeed, building roofs, parking lots, streets and sidewalks, all infrastructures in concrete, roads limit the infiltration of water into the soil. This results in an increase increased runoff as a result of decreased natural infiltration of stormwater which contributes to devastating urban floods.

Since 1995, Abidjan has been growing very fast and this can't take place without impacting the natural water paths of the watershed. It is not only the biggest city of the country but also the city where the government is and where the economic activities of the country are done. Table … gives some facts to better understand or better perceive what's happening in terms of urbanization in this city of West Africa.

Table 1

		Population of urban agglomeration ('000)				Annual rate of change (%)			Share in national urban population (%)			
Country	**City**	**1995**	**2005**	**2015**	**2025**	**1995-2005**	**2005-2015**	**1995-2015**	**1995**	**2005**	**2015**	**2025**
Algeria	El Djazaïr (Algiers)	1,973	2,282	2,594	3,149	1.45	1.28	1.37	12.0	10.5	9.0	9.0
Algeria	Wahran (Oran)	705	783	858	1,035	1.05	0.91	0.98	4.3	3.6	3.0	2.9
Angola	Luanda	1,899	3,533	5,506	8,567	6.21	4.44	5.32	54.3	59.0	54.8	54.9
Angola	Huambo	444	751	1,269	2,078	5.25	5.25	5.25	12.7	12.5	12.6	13.3
Argentina	Buenos Aires	11,390	13,330	15,180	16,479	1.57	1.30	1.44	37.1	38.3	39.2	39.0
Argentina	San Miguel de Tucumán	666	781	910	1,024	1.60	1.53	1.56	2.2	2.2	2.4	2.4
Argentina	La Plata	656	723	846	955	0.96	1.58	1.27	2.1	2.1	2.2	2.3
Burkina Faso	Ouagadougou	667	1,328	2,741	4,732	6.89	7.25	7.07	43.7	45.9	51.2	53.6
China	Beijing	8,305	12,813	20,384	26,494	4.34	4.64	4.49	2.2	2.3	2.6	2.8
Côte d'Ivoire	Abidjan	2,535	3,545	4,860	6,729	3.35	3.15	3.25	43.3	43.5	42.1	42.1
Côte d'Ivoire	Bouake	427	572	762	1,048	2.91	2.87	2.89	7.3	7.0	6.6	6.6
France	Paris	9,510	10,092	10,843	11,565	0.59	0.72	0.66	21.9	21.3	21.0	20.8
France	Toulouse	714	825	938	1,048	1.45	1.28	1.37	1.6	1.7	1.8	1.9
Germany	Berlin	3,471	3,391	3,563	3,654	-0.23	0.49	0.13	5.7	5.5	5.7	5.8
Germany	Köln (Cologne)	965	976	1,037	1,080	0.12	0.60	0.36	1.6	1.6	1.7	1.7
Ghana	Kumasi	909	1,544	2,599	3,707	5.30	5.20	5.25	13.5	15.3	17.8	19.0
Ghana	Accra	1,415	1,854	2,277	2,870	2.70	2.06	2.38	21.0	18.3	15.6	14.7
Niger	Niamey	552	816	1,090	1,744	3.91	2.89	3.40	38.2	37.0	30.2	27.5
Nigeria	Lagos	5,983	8,859	13,123	20,030	3.93	3.93	3.93	17.1	16.2	15.0	15.1
Switzerland	Zürich (Zurich)	1,048	1,130	1,246	1,406	0.75	0.98	0.86	20.3	20.8	20.5	20.6

It's true that cities like Luanda in Angola have an urbanization rate of 5% from 1995 to 2015 and Abidjan has 3%; nevertheless, urbanization should go together with a good sanitation or drainage master plan for a good management of stormwater.

Table 2

	Urban population ('000)				Level of urbanization (%)			
	1995	2005	2015	2025	1995	2005	2015	2025
WORLD	2,568,063	3,199,013	3,957,285	4,705,774	44.7	49.1	54.0	58.2
More developed regions	860,171	920,702	985,831	1,034,150	73.3	75.8	78.3	80.4
Less developed regions	1,707,892	2,278,311	2,971,454	3,671,623	37.4	43.0	49.0	54.0
Least developed countries	133,757	198,147	295,178	427,084	22.9	26.5	31.4	36.6
Less developed regions, excluding least developed countries	1,574,134	2,080,164	2,676,276	3,244,540	39.5	45.7	52.2	57.6
Less developed regions, excluding China	1,303,727	1,693,998	2,166,067	2,696,694	39.5	42.9	46.8	50.7
High-income countries	873,730	954,869	1,042,669	1,106,576	75.5	78.0	80.4	82.3
Middle-income countries	1,544,557	2,033,716	2,615,346	3,180,233	38.7	44.8	51.3	56.8
Upper-middle-income countries	928,664	1,229,547	1,574,772	1,857,018	44.6	53.9	63.5	70.7
Lower-middle-income countries	615,893	804,168	1,040,574	1,323,215	32.3	35.7	39.8	44.5
Low-income countries	133,543	191,782	278,657	397,055	23.2	26.4	30.8	35.7
Sub-Saharan Africa	163,172	240,036	359,534	522,530	29.1	33.0	37.9	42.9
AFRICA	236,904	330,742	471,602	658,814	33.1	36.3	40.4	44.9
Ethiopia	7,885	11,958	19,266	30,190	13.8	15.7	19.5	24.2
Kenya	5,007	7,757	11,978	17,973	18.3	21.7	25.6	30.3
Gabon	814	1,151	1,526	1,916	75.4	83.4	87.2	88.5
Algeria	16,416	21,677	28,739	35,145	56.0	63.8	70.7	75.6
Egypt	26,188	30,884	36,538	43,610	42.8	43.0	43.1	45.0
Botswana	776	1,033	1,181	1,357	49.0	55.1	57.4	60.5
South Africa	22,572	28,717	34,663	39,313	54.5	59.5	64.8	69.4
Côte d'Ivoire	5,859	8,147	11,538	15,968	41.2	46.8	54.2	60.5
Ghana	6,728	10,116	14,583	19,506	40.1	47.3	54.0	60.0
Niger	1,446	2,204	3,609	6,332	15.8	16.7	18.7	22.2
Nigeria	34,919	54,541	87,681	132,547	32.2	39.1	47.8	55.3
Senegal	3,452	4,634	6,544	9,283	39.6	41.1	43.7	47.8
China	383,156	560,518	779,479	947,540	31.0	42.5	55.6	65.4
Japan	97,117	109,174	118,572	118,715	78.0	86.0	93.5	96.3
Sweden	7,399	7,614	8,319	9,056	83.8	84.3	85.8	87.3
France	43,456	47,393	51,674	55,548	74.9	77.1	79.5	81.7
Germany	60,936	61,498	62,170	62,654	73.3	73.4	75.3	77.5

From this table, one can see that from 2005 to 2015, Côte D'Ivoire got respectively 46.8% and 54.2% level of urbanization which is an increase of 7.4% in ten years. While, the middle-income countries achieved an increase of 6.5% over the same period of time. And Côte D'Ivoire Is a middle-income country; so, its urbanization rate is above the average of this category of countries. Egypt’s urbanization is almost constant between 2005 and 2015.

What makes this comparative study interesting is that frequent flooding in Abidjan, the most urbanized city, has been occurring since 2007. This is also supported by the intense rainfall at Abidjan airport on June 20, 1983 with a water depth of 311 mm (SODEXAM) but did not flood any place in the city, while we got 302 mm on June 19, 2018 which resulted in a very severe flooding.

Therefore, urbanization is clearly intensifying flooding in the city of Abidjan.

Chapter 2: Literature review

2.0 Urban Flooding

As a result of uncontrolled human activities, urbanization and unpredictable tropical weather conditions, most of tropical developing countries are facing a risk of urban flood. This is becoming catastrophic during tropical cyclone and flash flood situations. Such events cause loss of lives, damage properties, road and drainage systems, and whole environment.

There are many studies which emphasize that, with the threat of climate change; such natural disasters are likely to amplify this trend in years to come. In many situations it is the population living in low lying urban areas that are most prone to these disasters. From this it follows that the scientific and professional communities have responsibility to constantly evolve with better stormwater management approaches to minimize hazard risks of urban flooding while addressing different climate conditions.

The water management practices can vary based on climate conditions, geographic location, availability of resources and culture. Especially for tropical islands the sudden and heavy rainfall can unexpectedly occur, since typically they have mountainous geographical character, low lands and basins are suddenly flooded with high flows. Therefore, in order to minimize the frequent urban flash flood damages in tropical Island, the structural measures, effective non-structural measures such as better hazard mitigation and prevention, improved preparedness and warning systems, well organized pre-emptive action and emergency response have to be effectively integrated. Therefore, the management of urban flooding is a multi- disciplinary process. The urban planners, economists; lawyers, emergency services and other professionals should be involved along with the engineers in this multi- disciplinary process to develop strategic plans for hazard reduction.

Parkinson and Mark (2005), have presented the short, medium and long-term objectives of storm water management strategies. In the short-term, the priorities are runoff control flood protection and pollution mitigation strategies, which in many developing countries have yet to be addressed effectively. The medium-term objectives focus on the development and implementation of water quality improvement, water conservation and strategies to preserve the hydrology and natural catchments. The long-term objectives place greater emphasis on preservation of natural resources and the amenity value of water in the urban environment for recreational activities and to promote an increased awareness of environmental issues. Although these objectives may initially appear to be somewhat idealistic goals especially considering the existing situation in many developing countries, it is important that planners and designers of urban drainage systems aim to satisfy the need of future generations while keeping with the objectives of sustainable development as defined by World Commission on Environment and Development in 1987.

2.1 Urban Flood Models

Flooding is a natural and variable phenomenon, it can occur on any land surface either in rural or urban. Flooding results in damage to lives, property, crops and negative impacts on human welfare. Flood Plain Management aims to minimize damages and reduce the threat to human life and welfare when major flood events occur. Numerical simulation provides good information of physical process. Expansion process and the distribution of the water depth by numerical simulation results are helpful for discussion and consideration comparing with field survey.

From the review of application concerning modelling of urban flooding it can be concluded that it is important to have a hydrodynamic model based on full dynamic equations in order to describe the flood sufficiently. In this section, a model of such specification has been reviewed. The one-dimensional numerical models are based on the cross-sectional averaged Saint-Venant equations, describing the development of the water depth h and the discharge Q or the mean flow speed U.

There are 4-point and 6- point numerical schemes. Examples of such model can be found in commercial software such as MIKE-11, MOUSE, SOBEK, Infoworks, SWMM, MIKE-21, etc. Briefly, the numerical solution is obtained from a finite difference formulation of the equations, using a scheme, which is based on alternating Q and h points (Abbott and Ionescu, 1967).

2.2 Runoff Curve Number Method

The SCS Runoff Curve Number method is used in this study to compute the runoff. It was developed by the United States Department of Agriculture (USDA) Soil Conservation Service (SCS) and is a method of estimating rainfall excess from rainfall (Hjelmfelt, 1991). The method is described in detail in National Engineering Handbook (2004). The chapter was prepared originally by Mockus (1964) and was revised by Hjelmfelt (1998) with assistance from the NRCS Curve Number work group and H.F. Moody. Despite the wide use of the curve number procedure, documentation of its origin and derivation are incomplete (Hjelmfelt, 1991). The conceptual basis of the curve number method has been the object of both support and criticism (Ponce and Hawkins, 1996). The major disadvantages of the method are sensitivity of the method to Curve Number (CN) values, fixing the initial abstraction ratio, and lack of clear guidance on how to vary Antecedent Moisture Conditions (AMC). However, the method is used widely and is accepted in numerous hydrologic studies. The SCS method originally was developed for agricultural watersheds in the mid-western United States; however, it has been used throughout the world far beyond its original developers would have imagined.

The SCS Curve Number method is one of the most popular methods for computing the volume of surface runoff for a given rainfall event from small watersheds. This method's usefulness is mainly dependent on its convenience, authoritative origins, simplicity and responsiveness to four distinct catchment properties: soil type, land use/treatment, surface condition and antecedent condition (Kamal Neupane, 2015). The CN method was used to performhydrologic modelling of the Cedar Creek Watershed for the construction of a rainfall-runoff model and gave satisfactory results (Kamal Neupane, 2015).

The basis of the curve number method is the empirical relationship between the retention (rainfall not converted into runoff) and runoff properties of the watershed and the rainfall. Mockus found equation 1 appropriate to describe the curves of the field measured runoff and rainfall values (National Engineering Handbook, 2004). Equation 1 describes the conditions in which no initial abstraction occurs.

$$\frac{F}{S} = \frac{Q}{P} \qquad \text{Equation 1}$$

where F = P – Q = actual retention after runoff begins;

Q = actual runoff

S = potential maximum retention after runoff begins ($S \geq F$)

P = potential maximum runoff (i.e., total rainfall if no initial abstraction).

For most applications, a certain amount of rainfall is abstracted. The three important abstractions for any single storm event are rainfall interception (Meteorological rainfall minus throughfall, stem flow and water drip), depression storage (topographic undulations), and infiltration into the soil. The curve number method lumps all three abstractions into one term, the Initial abstraction (Ia), and subtracts this calculated value from the rainfall total volume. The total rainfall must exceed this initial abstraction before any runoff is generated. This gives the potential maximum runoff (rainfall available for runoff) as P – Ia. Substituting this value in equation 1 yields following equation

$$\frac{P-Ia-Q}{S} = \frac{Q}{P-Ia} \qquad \text{Equation 2}$$

Rearranging terms in Equation 2 for Q gives

$$Q = \frac{(P-Ia)2}{(P-Ia)+S} \qquad \text{Equation 3}$$

The SCS provided the following empirical Equation 4 based on the assumption Ia was a function of the potential maximum retention S.

$$Ia = 0.2S \qquad \text{Equation 4}$$

The potential maximum retention S is related to the dimensionless parameter CN in the range of $0 \leq CN \leq 100$ by Equation 5.

$$S = \frac{1000}{CN} - 10 \qquad \text{Equation 5}$$

Substituting Equation 4 into Equation 3 yields,

$$Q = \frac{(P-0.2S)2}{P+0.8S} \qquad \text{Equation 6}$$

Equation 6 has only one parameter that needs to be evaluated (i.e., S) which can be determined by using Equation 5 and curve number tables published by the SCS.

2.3 GIS techniques in Hydrologic and Hydraulic Modelling

The increasing availability of spatial data (terrain and rainfall), GIS software to manage spatial data, faster processors, and the availability of interfaces to connect simulation models with GIS, have increased use of GIS in watershed modelling (Carpenter et al, 2001; Singh and Woolhiser, 2002; Vieux, 1991, Whiteaker et al, 2006: Garbrecht et al, 2001). HEC-GeoRAS is the geospatial tool used in this study, which serves as the interface between GIS and the simulation model HEC-RAS. Figure 15 shows the flow diagram for using HEC-GeoRAS (HEC GeoRAS User’s manual, ver 4.0, Sep 2005). HEC-GeoRAS allows engineers to concentrate on hydraulic model development and analysis rather than GIS mechanics. The user environment provides engineers an opportunity to view real-world systems of interest, which in turn assists them to rectify errors and make informed decisions in the model development (Ackerman et al, 1999). Tate et al, (2002) applied HEC-GeoRAS successfully to create a terrain model for floodplain mapping. A widely used approach is watershed modelling that divides the drainage basin into discrete units possessing similar rainfall-runoff and physical characteristics. This approach reduces model complexity and spatially distributed data requirements in basin-scale models (Beighley et al, 2005).

2.4 Model selection

For selecting potentially an appropriate modelling tool in any research, there are various criteria which can be applied to choose the most suitable model. According to Cunderlik and Simonovic (2003) the choice mainly depends on the requirements and needs of the research or project under interest. Cunderlik and Simonovic (2003) put the following as criteria:

- ✓ Required outputs of the model
- ✓ Availability of input data
- ✓ Prices and availability of the model and
- ✓ The model structures

2.5 HEC-RAS

The 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 (GUI), separate analysis components, data storage and management capabilities, graphics and reporting facilities.

The HEC-RAS system contains the following river analysis components for: (1) steady flow water surface profile computations; (2) one dimensional and/or two-dimensional unsteady flow simulation; (3) Quasi unsteady or fully unsteady flow 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 water surface profiles are computed.

User Interface

The user interacts with HEC-RAS through a graphical user interface (GUI). The main focus in the design of the interface was to make it easy to use the software, while still maintaining a high level of efficiency for the user. The interface provides for the following functions:

- ✓ File management

- ✓ Data entry/editing and GIS data interfaces
- ✓ River analyses
- ✓ Tabulation and graphical displays of input and output data
- ✓ Inundation mapping and animations of water propagation
- ✓ Reporting facilities
- ✓ On-line help

River Analysis Components

Steady Flow Water Surface Profiles. This component of the modelling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles.
The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions).

The effects of various obstructions such as bridges, culverts, dams, weirs, and other structures in the flood plain may be considered in the computations. The steady flow system is designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel modifications, and levees.

Special features of the steady flow component include: multiple plan analyses; multiple profile computations; multiple bridge and/or culvert opening analysis; bridge scour analysis; split flow optimization; and stable channel design and analysis.

Unsteady Flow Simulation. This component of the HEC-RAS modelling system is capable of simulating one-dimensional; two-dimensional; and combined one/two-dimensional unsteady flow through a full network of open channels, floodplains, and alluvial fans. The unsteady flow component can be used to performed subcritical, supercritical, and mixed flow regime (subcritical, supercritical, hydraulic jumps, and drawdowns) calculations in the unsteady flow computations module.

The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were developed for the steady flow component were incorporated into the unsteady flow module.

Special features of the unsteady flow component include: extensive hydraulic structure capabilities Dam break analysis; levee breaching and overtopping; Pumping stations; navigation dam operations; pressurized pipe systems; automated calibration features; User defined rules; and combined one and two-dimensional unsteady flow modelling.

Sediment Transport/Movable Boundary Computations. This component of the modelling system is intended for the simulation of one-dimensional sediment transport/movable boundary calculations resulting from scour and deposition over moderate to long time periods.

The sediment transport potential is computed by grain size fraction, thereby allowing the simulation of hydraulic sorting and armoring. Major features include the ability to model a full network of streams, channel dredging, various levee and encroachment alternatives, and the use of several different equations for the computation of sediment transport.

The model is designed to simulate long-term trends of scour and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage, or modifying the channel geometry. This system can be used to evaluate deposition in reservoirs, design channel contractions required to maintain navigation depths, predict the influence of dredging on the rate of deposition, estimate maximum possible scour during large flood events, and evaluate sedimentation in fixed channels.

Water Quality Analysis. This component of the modelling system is intended to allow the user to perform riverine water quality analyses.
The current version of HEC-RAS can perform detailed temperature analysis and transport of a limited number of water quality constituents (Algae, Dissolved Oxygen, Carbonaceous Biological Oxygen Demand, Dissolved Orthophosphate, Dissolved Organic Phosphorus,

Dissolved Ammonium Nitrate, Dissolved Nitrite Nitrogen, Dissolved Nitrate Nitrogen, and Dissolved Organic Nitrogen).

2.6 HEC-HMS

The Hydrologic Engineering Center-Hydrologic Modelling 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 the widest possible range of problems. This includes large river basin water supply and flood hydrology, and small urban or natural watershed runoff. Hydrographs produced by the program are 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, and systems operation.

The program is a generalized modelling system capable of representing many different watersheds. A model of the watershed is constructed by separating the hydrologic cycle into manageable pieces and constructing boundaries around the watershed of interest. Any mass or energy flux in the cycle can then be represented with a mathematical model. In most cases, several model choices are available for representing each flux. Each mathematical model included in the program is suitable in different environments and under different conditions. Making the correct choice requires knowledge of the watershed, the goals of the hydrologic study, and engineering judgment.

The program features a completely integrated work environment including a database, data entry utilities, computation engine, and results reporting tools. A graphical user interface allows the seamless movement between the different parts of the program. Program functionality and appearance are the same across all supported platforms.

Use of GIS in Urban Flood Modelling

As a result of rapid urbanization and climate changes urban flooding has become an increasing and continuous threat all over the world. Therefore, better analytical understanding and visualization of this disaster is essential to develop strategies that will minimize the risk of urban flooding. At present, 1D River models, digital elevation models and other GIS data sets for hydraulic modelling and floodplain mapping are often collectively used to predict areas at risk of flooding. Hydraulic and hydrological modelling are an obvious choice for predicting those areas of the floodplain most at risk to flooding and for providing information for use in the evaluation of the associated economic damage.

Geographic Information System (GIS) has evolved over the last couple of decades into a powerful tool for storing, managing, analyzing and displaying spatial data (Burrough and McDonnell, 1998). Generally, the integration of hydraulic models and GIS for floodplain mapping aims to provide

- ✓ Functions to extract information describing the channel system from a terrain model to provide a network description (e.g. topographic data of channel network and adjacent area)
- ✓ Tools that are capable of manipulation results from hydraulic models and displaying and automating mapping of floodplain in GIS (e.g. water surface profiles) (Jones et al, 1998).

Approaches for integrating hydraulic model to GIS have resulted in many different tools for flood prediction and floodplain mapping, e.g. HEC-GeoHMS, MIKE 11 GIS (Muller and Rungoe, 1995), HEC-GeoRAS (Ackerman et al., 2000).

Hydraulic models are complex tools, requiring large amounts of input data for their specification to a particular application and produce a vast amount of output data. The data requirements for distributed hydraulic models are grouped into topographic and hydrologic data (Cunge et al., 1980).

Topographic data: describing the channel geometry of the river system and adjacent areas (channel widths, cross-sectional areas) and elevations of the flood plain.

Hydrologic data: model boundary conditions (e.g. inflow hydrographs) and discharge and water level data for the calibration of model parameters (e.g. bed roughness and weir coefficients).

2.7 Data requirements

2.7.0 HEC-RAS

Geometric data consist of establishing the connectivity of the river system (River System Schematic), entering cross-section data, defining all the necessary junction information, adding hydraulic structure data (bridges, culverts, dams, levees, weirs, etc…), pump stations, storage areas, and two-dimensional flow areas. The geometric data is entered by selecting Geometric Data from the Edit menu on the HEC-RAS main window. The drawing area will be blank on your screen, until you have either drawn in your own river system schematic or imported data from a GIS.

Once the geometric data are entered, the modeller can then enter either steady flow or unsteady flow data. The type of flow data entered depends upon the type of analyses to be performed. For the discussion in this chapter, it is assumed that a steady flow hydraulic analysis will be performed. The data entry form for steady flow data is available under the Edit menu bar option on the HEC-RAS main window.

Steady flow data consist of: the number of profiles to be computed; the flow data; and the river system boundary conditions. At least one flow must be entered for every reach within the system. Additionally, flow can be changed at any location within the river system. Flow values must be entered for all profiles.

Boundary conditions are required in order to perform the calculations. If a subcritical flow analysis is going to be performed, then only the downstream boundary conditions are required. If a supercritical flow analysis is going to be performed, then only the upstream boundary conditions are required. If the modeller is going to perform a mixed flow regime calculation, then both upstream and downstream boundary conditions are required. The Boundary Conditions data entry form can be brought up by pressing the Reach Boundary Conditions button from the Steady Flow Data entry form.

HEC-RAS has the ability to import geometric data in several different formats. These formats include: a GIS format (developed at HEC); the USACE Standard Surveyor format; HEC-2 data format; HEC-RAS data format; UNET geometric data format; and the MIKE11 cross section data format. Data can be imported into an existing HEC-RAS geometry file or for a completely new geometry file. Multiple data files can be imported into the same geometric data file on a reach-by-reach basis.

2.7.1 HEC-HMS

The physical representation of a watershed is accomplished with a basin model.

Hydrologic elements are connected in a dendritic network to simulate runoff processes. Available elements are: subbasin, reach, junction, reservoir, diversion, source, and sink. Computation proceeds from upstream elements in a downstream direction.

Meteorological data analysis is performed by the meteorological model and includes shortwave radiation, precipitation, evapo-transpiration, and snowmelt. Not all of these components are required for all simulations. Simple event simulations require only precipitation, while continuous simulation additionally requires evapotranspiration. Generally, snowmelt is only required when working with watersheds in cold climates.

A geographic information system (GIS) can use elevation data and geometric algorithms to perform the same task much more quickly. A GIS companion product has been developed to aid in the creation of basin models for such projects. It is called the Geospatial Hydrologic Modelling Extension (HEC-GeoHMS) and can be used to create basin and meteorologic models for use with the program.

2.8 Stormwater management

2.8.0 Historical evolution of Abidjan drainage network

One finds in the references [2], [5], [7] and [9] elements which help in relating the evolution of the urban drainage network of Abidjan. This one was closely related to the sewerage in general, the technical choice in the first times consisting in treating wastewater and stormwater in the same unit network (combined system). The technical mission of the Consulting Engineers Colcanap and Dufour, appointed by the French Ministry of the Environment in 1981, produced a report of 300 pages on the sanitation of Abidjan (ref. [7]); that report focused on the treatment of wastewater but also dealt with the drainage of stormwater. That report makes understand that the construction of the drainage network of Abidjan goes back to the middle of the years 1970, the first investments being initiated within the framework of the Program FNA/BIRD: drainage networks were constructed in the neighbourhoods Adjamé, Attécoubé, Treichville, Koumassi, Vridi, Bouet Port, Williamsville and Zone 4 for a sum of 8.9 billion FCFA (an equivalent of 16.2 million USD) and in the “new” neighbourhood of Abobo and Banco for a sum of 5 billion FCFA (9.1 million USD). Early 1980, the choice of the separate system prevails and funds are allocated for development of Gouro catchment (5.5 million USD) and the implementation of the drainage network of Riviera (11 million USD). It was at the same time that the BCET of the Ministry of Transport, former BNETD, studied the development of the Creek of Danga (ref. [5]). Bouvier (ref. [2]) and Desbordes (ref. [9]) studied in 1989-1990 the urban drainage in West Africa and reported also the evolution of the drainage network of of Abidjan. Bouvier chose Yopougon as the case study of his PhD work of (ref. [2]) and described the open canal system the canal of UNIWAX included, which was constructed during the decade 1975-1985.

2.8.1 Characteristics of catchments in Abidjan

Several documents from technical study reports, doctorates thesis, scientific research papers give the geomorphological characteristics of the catchments of Abidjan (references [2], [6], [9], [10], [12] and [15]). The PhD theses of Bouvier (ref. [2]) and of Sighomnou (ref. [15]) give in particular experimental estimates of the permeability of soil and runoff coefficients. Jourda et al. [ref. [12]) studied the groundwater of the Continental Terminal, prevailing aquifer in the south of the country, and their works made us understand the low relative permeability of the grounds of the studied catchment. The research works within the framework of the experimental watershed of the ORSTOM at Adiopoudomé in the south of Yopougon - emphasized in the references [2], [6], [9] and [15] - also made us apprehend the aptitudes of the grounds infiltration capacity and then their ability to generate runoff. Hauhout gives in his paper (ref. [10]) an insight on the vulnerability of Abidjan's riverbanks and their exposure to the risk of landslide, with an estimation of the damage caused by rainfall events in the precarious neighbourhood of Attécoubé set up on riverbanks.

2.8.2 Hydrometeorological data and observations

If rainfall data are sufficiently available at SODEXAM, it is not the same case for the observations of flows in channels and/or flowrates in urban waterways. The theses of Bouvier (ref. [2]) and of Sighomnou (ref. [15]), mentioned above, particularly related the simultaneous measurements of hyetographs and hydrographs at the ORSTOM gage of Adiopoudomé. Desbordes and Bouvier report written on the behalf of CIEH (ref. [9]) made the synthesis of it whereas before them, Cazenave et al. (ref. [6]) and Sighmonou (ref. [16]) reported partial results of field measurements. CONCEPT and ICI for their part talked about thirty pluviographic years of recordings carried out by SODEXAM at Abidjan-Airport between 1970 and 2000, in the study of the Gourou catchment (ref. 8]) realized in 2012. In that last document, is described the frequency analysis of storms that made possible the establishment of design storm in an original way at this precipitation gage.

2.8.3 Recent and under way development

The urban area of Abidjan which has been undergoing for several decades a rapid growth, is subject to big urban development and infrastructures projects. On one hand, the projects that are already implemented or under way have direct impact on land use and thus on their imperviousness. On the other hand, the project of improvement of the drainage network must take into account future projects to come. The master plan of Abidjan planning (ref. [13]), completed in 2014, is obviously the collection of references concerning the synthesis of the urban evolution of the city and of the orientation given by the public authority to its future development. That document gave areas of future urban growth and infrastructures projects.

2.8.4 Hydrological methods and approaches

As one of the main topic is the conversion or transformation of rainfall into runoff, a great part of the literature review focused on the hydrological approaches. It was not about a pragmatic review tending, on one hand, to select the most suitable methods to treat the topic, on the the other hand to get estimated parameters and geomorphological factors whose direct measurements were not possible due to lack of time and equipment. The already mentioned work of Bouvier (ref. [2] and [3]), of Sighomnou (ref. [15]) and of Desbordes and Bouvier (ref. [9]) were obviously the first references as such. They offer the specific advantage indeed to estimate and quantify runoff in the context of the city subject of this study; equations and values of the hydrological parameters thes authors resulted in are for "Abidjan" or at least West African. On another side, recent reports of BRLi (ref. [4]) and previous report of CONCEPT-ICI (ref. [8]), report of study of Gourou catchment were made profitable considering they are another way of dealing with runoff generation issues compared to the traditional approaches of Caquot or other empirical methods. It was extracted from the last document original calculation of design storm whereas the first gave useful comparative tools so that waste of time was avoided for this work.

Study area

The District of Abidjan is located in the south of Côte D'Ivoire between latitude 5° 10' and 5° 38' North and longitude 3° 4' and 5° 21' West. It encompasses thirteen (13) municipalities since 2001 and has an estimated population of 4, 739,752 inhabitants (INS, 2013) which represents 20.3% of the national population.

Yearly floods are taking place in several places in the city (Gourou watershed, Angré, Koumassi, Yopougon, Palmeraie). The area of Koumassi known as **Koumassi catchment** will be the one for the study.

2.9 Previous studies on flood events in Abidjan

Though flooding is a common problem in the floodplain of the Gourou catchment, researches regarding this are limited. The current flood problems and the lack of studies in the area show the importance of an in-depth study. Researches by Danumah (2016) and Kablan et al., (2017) were on assessing flood risks under changing climate and land use in the District of Abidjan. They focused on developing maps of flood hazard and risk levels for the whole town including this catchment using remote sensing. He concluded that 34% of the city is at flood risk and that extreme rainfall are more frequent in the city nowadays. Other studies, Célestin (2008) and Savane et al., (2003) treated certain factors controlling flood risk such as rainfall risks and uncontrolled urban growth in two municipalities which don’t include this study area. Kouamé et al., (2013), Jourda et al., 2003 and Ahoussi et al., (2013) raised the issues of inefficiency of the drainage network and impervious areas as main divers of flooding. Dongo et al., (2008) developed a hydraulic model to prevent flooding events in some slums of Yopougon the biggest municipality of Abidjan in the north of the town. Kangah and Alla Della (2015) used digital elevation models (DEM) and geographic information systems (GIS) to determine flood areas and identify the type of flood, and the risk factors in the Bonoumin-Palmeraie watershed, one of the most impacted neighbourhoods in Abidjan. In that work, they use multicriteria to identify floodprone areas in the watershed of Bonoumin – Rivièra Palmeraie and then did not include any hydrologic nor hydraulic modelling to estimate flood that is likely to occur.

The lack of researches on the catchments of the country in general and of Abidjan in particular because of the scarcity and quality of long-term hydrological data (Nka et al., 2015) is a major problem in Ivorian water issues management.

Chapter 3: Methodology

3.0 Study area

The District of Abidjan is located in the south of Côte D'Ivoire between latitude 5° 10' and 5° 38' North and longitude 3° 4' and 5° 21' West. It encompasses thirteen (13) municipalities since 2001 and has an estimated population of 4, 739,752 inhabitants (INS, 2014) which represents 20.3% of the national population.

Yearly floods have been taking place in several places in the city: Gourou watershed (rounabout of Indénié), Angré, Koumassi, Abobo, Yopougon, Palmeraie.

The watershed of Bonoumin-Rivièra Palmeraie (figure 1) is located entirely in the municipality of Cocody and between longitudes 3° 50' and 4° 10' West and latitudes 5° 10' and 5° 30' North. This watershed is itself a sub-basin of the Bonoumin - Rivièra Golf watershed and is the largest watershed of the municipality of Cocody and covers more than ten neigbourhoods of which Bonoumin, Palmeraie, Allabra, Riviera 2. It remains a beautiful residential area and is recognized for the quality and the architecture of its buildings. It is a place where people who have high income live including diplomats.

In terms of geomorphology, the municipality of Cocody is on a large plateau. This relief is separated from the municipalities of Adjamé and Plateau by a large neckline which starts in the south of Abobo and ends at the bay of Cocody. With an average altitude around 40-50 m, it has a north – south flow direction. Indeed, altitudes vary between 80 and 100 m in the north and between 20 and 30 m in the south. This plateau ends on the Ebrié lagoon but the transition is a clift on the side of the neighbourhood of Cocody. This cliff is detached from the lagoonal costal line at Riviera where it behaves like a real bank from, separating the plateau from a small plain of 2 to 9 m. With a west-east direction, this bank, just like the cliff, is made of a series of valleys. These valleys and their tributaries are used as natural channels for the drainage of stormwater of the municipality of Cocody (Alla Della, 2015).

Map of west Africa, Côte D'Ivoire and Bonoumin Palmeraie

Because of the insufficiency and lack of maintenance of the drainage network on the catchment as well as lawless settlement in the sites planned for infrastructures, residential areas are flooded recurrently each rainy season, causing extensive damages on the existing infrastructures as well as disturbances of the traffic, economic activities and loss of lives.

3.1 Models and softwares description

3.1.0 Overall Methodology

This chapter provides the theoretical background for the understanding of the data processing and modeling procedures used in this study. The three software solutions and the mathematical models used in this study are presented in detail. ArcMap is used for all GIS related tasks, HEC-HMS for hydrologic- and HEC-RAS for hydraulic modelling. HEC-GeoHMS and HEC-GeoRAS serve as the interface between GIS and the hydraulic and hydrologic modelling. At the end of this section a brief literature review gives an overview about the applicability and limitations of the applied models.

The forcing condition for any hydrologic model is the rainfall. Several rainfall sources are available in variety of formats and can be used as is or in combination. HEC-HMS simulates rainfall-runoff process. From the given precipitation, it deducts losses and convolutes the excess rainfall with specified unit hydrograph and routes it through the channel to generate a runoff hydrograph. This flow information is used then in HEC-RAS to estimate water surface elevations. HEC-RAS simulates one and two-dimensional flow and generates water surface profiles for given flow conditions. It is capable of modelling both steady and unsteady flow conditions. The geometry file necessary for HEC-RAS simulation can be created in a GIS environment using HEC-GeoRAS toolbar and can be imported to a RAS environment. However, with the new version 5.0.4 of HEC-RAS this is possible directly in HEC-RAS using RAS mapper. In this case the output from HEC-RAS simulation can be exported into a GIS environment for floodplain delineation using either HEC-GeoRAS or a RAS mapper.

The models chosen for hydrologic and hydraulic simulations are the United Army Corps of Engineers Hydrologic Engineering Center's HEC HMS and HECRAS models. The models were used together with the GIS tool HEC-GeoRAS and HEC-GeoHMS coupled with AutoCAD and the ArcGIS data model; HEC-GeoHMS to provide an interface with GIS. Drainage features and Hydrologic Response Units (HRUs) were delineated using the Digital Elevation Model (DEM) and HEC-GeoHMS. Stage-Discharge rating curves were generated at each HRU outlet using the simulation model HEC RAS. The necessary geometry files for HEC-RAS simulation were developed using the Triangulated Irregular Network (TIN) model and the GIS tool HEC-GeoRAS in ArcGIS 10.3 and also from topographic data (AutoCAD) collected from ONAD. Fields measurements were carried out to get observed discharged at a particular outlet. HEC-HMS was used to simulate the watershed response to rainfall and was calibrated to match the observed stream hydrograph. Notably, the Peak Rate Factor of 230 was used in HEC-HMS simulation for specifying transformation hydrograph. After calibrating and validating the HMS model for WPBW, the NRCS statistical storm events for 2, 10, 25, 50 and 100-year recurrence intervals were run to determine the flows through various hydrologic elements of the model. The flows for the 2, 5, 10, 25, 50 and 100-year storm event were input in HEC-RAS to generate water surface profiles. The water surface profiles from HEC-RAS were exported to the GIS environment and the floodplain was delineated using the HEC-GeoRAS interface. The resulting floodplain was compared against the observed flood level.

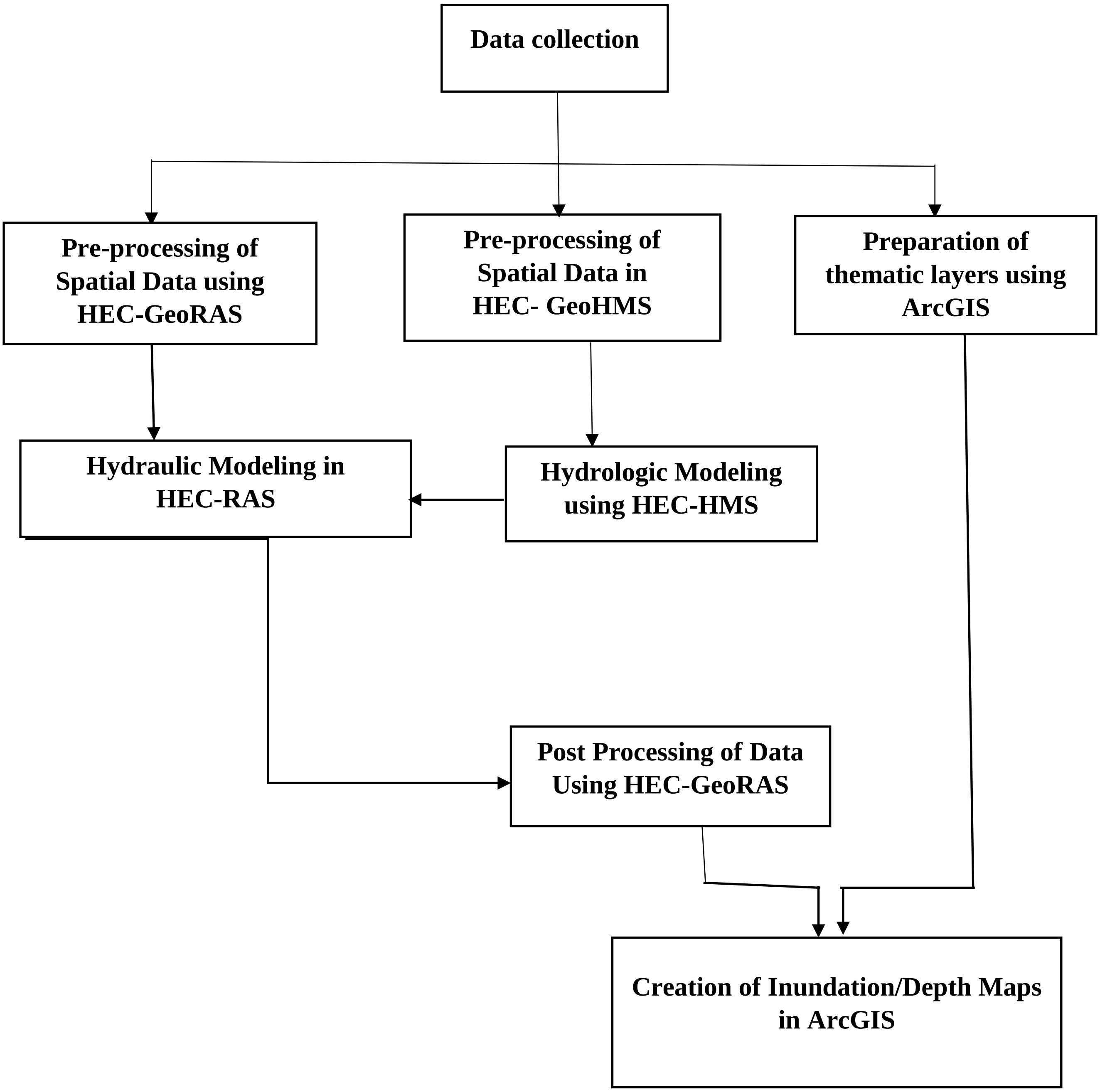


Figure 1: Chart showing the process involved in the methodology

3.1.1 Rainfall-Runoff Model: HEC-HMS

3.1.1.0 Fundamentals

HEC-HMS is an open source software for the modeling of the rainfall-runoff process developed by the U.S. Army Corps of Engineering´s Hydrologic Engineering Center. The software includes a graphical user interface for the management and analysis of the model data. It is important to mention that HEC-HMS itself is not an actual hydrological model rather than a software that enables the user to perform hydrological modeling based on a wide selection of common mathematical models used in hydrology. In HEC-HMS, the rainfall-runoff process in a watershed is represented in a simplified manner as shown in Figure…

This simplified representation of the runoff process does not account for the storage and movement of water vertically within the soil layer. It is however sufficient to model a flood hydrograph as the result of a storm (HEC, 2000). For modeling purposes, this simplified hydrologic cycle is further divided into four components, which are modeled separately. The models included in the software can thus be categorized as follows:

Loss Method: A model to compute the runoff volume is often referred to as the loss method since it accounts for the losses that occur during a rainfall event as a result of infiltration and evapotranspiration. For each time interval in the modelling process, the loss method calculates the amount of water that contributes to the runoff in the river (effective rainfall).

Transform Method: Models of direct runoff are also called transform method, since they convert the effective rainfall over a watershed into a hydrograph at the outlet of the watershed. These models account for the surface roughness and geometry of the watershed.

Baseflow Method: Baseflow models are used to simulate the fraction of the runoff contributed by groundwater.

Routing Method: If the analysed watershed is divided into sub-watersheds, the flow at the outlet of a certain upstream watershed has to be routed through the river channel in the downstream watershed. The models used to simulate this routing process are therefore called routing methods. They account for the geometry and roughness of the relevant river channel.

Software Components

Figure 2.2 on the next page shows a schematic overview of the HEC-HMS software environment. In the control specifications, the computational time step and the date of the run are defined. The meteorological model is the representation of the rainfall event that is intended to be modelled. The physical basin model is essentially a simplified physical representation of the watershed which is prepared with HEC-GeoHMS in this study. The main features of the basin model are sub-basins, reaches and junctions. The modelling results comprise runoff hydrographs for each sub-basin as well as graphical and numerical representations of rainfall, losses and direct runoff for each sub-basin.

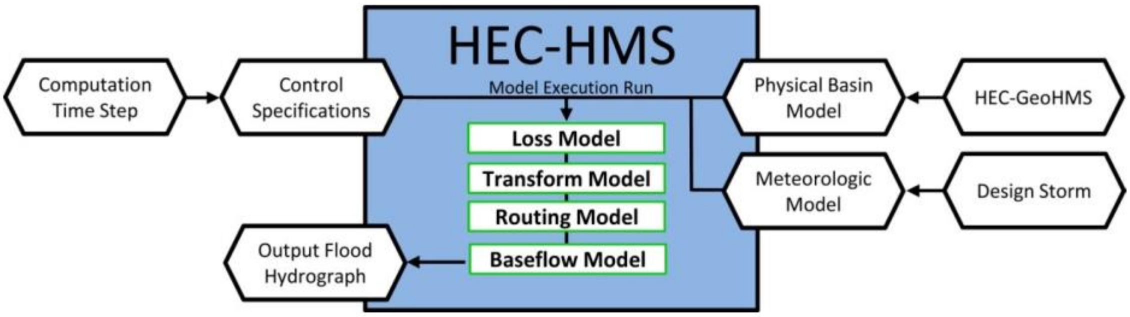


Figure 2.2: HEC-HMS software components

3.1.1.1 Hydrologic Model Selection and Description

Depending on the situation that is being modelled and the available data, an adequate mathematical model for each of the previously defined four components of the rainfall-runoff process needs to be chosen. In this study, the hydrologic modelling is performed primarily to generate flood hydrographs with certain statistical return periods resulting from single design storm events with the same statistical return periods. Since baseflow does not occur in the concrete canal, it can be neglected in the modelling process. Furthermore, the rivers or canal and the watersheds are ungauged and due to lack of equipment and time, complex field surveys were not possible. As the object of this study is a drainage canal which must convey stormwater from upstream at Saint Viateur (a sub neighbourhood in the northern part of Rivièra) of Abidjan, we should first look at the hydrological issue of converting rain into runoff. Several traditional methods (statistics) of rainfall-runoff conversion exist, but nowadays, with the development of technologies, this task is carried out by the means of computer tools which model the hydrological phenomenon of generation of runoff starting from the rains.

Computer-based or mathematical models are many and have been in use for more than 40 years (CONCEPT, 2015), this period coincides with the significant development of data processing, and then computers. They are each one based on a well-defined approach of the hydrological phenomenon of conversion of precipitations into runoff. There are as many approaches as of schools or hydrologists, each one privileging physical processes rather than others among the set of processes or methods which generate runoff; some approaches account for, for example, the retention of precipitations by vegetation for losses computation, contrary to other approaches which neglect this part of the hydrological process. In addition, each part of the process hydrological is explained differently according to the authors, following the example of the infiltration which is modelled by more than one equation: Horton, Green-Ampt, Holtan. Hydrological models choose one approach or at least a limited number among them.

IDF diagrams to be added

In addition, each one of the mathematical equation which describes part of the hydrological process comprises parameters in more or less great number and may be calculated easily. More the approach adopted to explain the hydrological process is exhaustive, the more the number of parameters is large. But at the same time, the determination of these parameters becomes challenging, because it requires preliminary works, and measurements are not always possible, neither easy to realize. The larger the number of parameters is, the more the total error of the hydrological process simulation of the is high, because it is the result of the partial errors made on the determination of each parameter.

Hydrologists specialist of models for rainfall-runoff are always challenged by the following questions:

- Which approaches explain the best the studied hydrological phenomenon?
- Among those approaches, which are those which require less parameters and whose determination is carried out easily in the allocated time?

The selected model should in all logic be based on an approach describing suitably the hydrological process of the study regarding the objectives.

3.1.1.2 Suitability of HEC-HMS for a data scarce region like Abidjan

As it can be seen from the presentation of HEC-HMS, it computes runoff and determine the hydrograph at the outlet of any given watershed, by accounting for the most significant parts of the hydrological process of generation of runoff from a given rain. For the estimation of those various parts, HEC-HMS offers the hydrologist/engineer a large variety of calculation methods that touch the discussed cases. Each sub-model corresponding to those methods of calculation requires more or less parameters and whose determination is more or less easy.

There exists a certain number of hydrologic and hydraulic models - sometimes coupled - designed to better adapt to urban areas. We can quote SWMM among most classical or MIKE URBAN among most recent. But these models require particularly large inputs to describe the watershed and render the transform and transfer method equations. For SWMM for example, not less than 13 parameters are necessary and if the hydrologist/engineer can to determine 8 from literature, it will remain 5 parameters which only be got from field measurements or by calibration with observed and recorded events. As mentioned above, more the number of parameters is large, the more the error of the hydrological process simulation is high, because it is the result of partial errors made on the determination of each parameter (CONCEPT, 2015).

However, in Abidjan and in spite of the frequent rain causing floods with loss of lives, neither flows nor water depth through drainage canals are recorded exempt for those of the Gourou catchment project located in the Gourou catchment. And this makes the hydrological model calibration difficult even impossible. Besides, soil parameters' values are not either available, as works interested in this field about Abidjan region are limited.

HEC-HMS, at the same time complete from the explanation point of view regarding the process of transforming rainfall into runoff, offers alternatives of sub-models that don't require an excessive number of parameters. It is thus better appropriate for a situation of data scarce region; and these data serve as basis to calculate some other parameters. Indeed, when there are not available the use of other more sophisticated models will make them not very reliable and trivial.

Based on this background, the models shown in Table 2.1 were chosen for each of the four components of the runoff process.

Table 3 : Hydrologic model selection and categorization

Component	Chosen Model	Categorization
Loss Method	SCS Curve Number	event, semi-distributed, fitted parameter
Transform Method	SCS Unit Hydrograph	event, lumped, empirical, fitted parameter
Routing Method	Muskingum	event, lumped, measured parameter

All three chosen models are designed to model single storm events rather than continuous precipitation data (HEC, 2000). Furthermore, they are lumped models, meaning that spatial variations of processes and characteristics are not considered explicitly rather than averaged for each sub-watershed. The SCS Curve Number (CN) and Unit Hydrograph (UH) models are both of empirical nature meaning that they are based on observations of the in- and output of a certain system without trying to represent the actual conversion processes as done in conceptual models. The Muskingum model is quasi conceptual since it is based on simplified equations of shallow water flow. Each of the three chosen models and the underlying mathematical equations are described in detail in the following sections.

Muskingum Method

Calculation of time of concentration (TC)

Kirpich method can be used for Calculation of TC as below (Alizadeh, 2001):

$Tc = 0.0195L0.77S^{-0.385}$

Where: TC is time of concentration (hour), S is mean slop of main river (m/m) and L is length of main river (m).

Flow calculation in reaches

In Muskingum method, for flow modelling X and K parameters must be evaluated.

Theoretically, K is time of passing of a wave in reach length. They can be calculated respectively by below equation (Alizadeh, 2001):

$$K = \frac{0.6L}{V}$$

Where: L is length of reach and V is velocity (m/s)

X parameter presented by Manning equation as below:

$$X = \frac{I^{0.5}}{np^{2/3}}$$

Where: I is river slop, n is roughness coefficient of Manning and P is wet perimeter (m) (Mahdavi, 2005).

Loss Method: SCS Curve Number Method

The U.S. Natural Resource Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS)) Curve Number method used in this study estimates the effective rainfall as a function of the cumulative rainfall, the land use, the soil type and the antecedent moisture condition of the soil. The model is described in detail in the National Engineering Handbook (NEH) (NRCS 2004). It was created based on the analysis of a large number of small and gauged agricultural watersheds throughout the US. Apart from the input precipitation, the method uses a single parameter, the CN to characterize the watershed. The CN quantifies the infiltration capacity and theoretically ranges between 0 (100% of the total rainfall infiltrate) to 100 (0% of the total rainfall infiltrate). The basic runoff equation of the CN method is shown in Eq. 2.1. as described previously in chapter 2.

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S}$$

where Q = runoff (mm)

P = rainfall (mm)

S = potential maximum retention after runoff begins (mm)

Ia = initial abstraction

Transform Method: NRCS Unit Hydrograph

The transformation of excess precipitation into runoff is commonly done using a unit hydrograph (UH). A UH is defined as the hydrograph of surface runoff resulting from effective rainfall in a unit of time (1 min or 1 hour) produced uniformly in space and time over the watershed (Sherman, 1942). Every watershed has a unique UH which highly depends on the topography and shape of the area and is usually generated through rainfall and runoff measurements. Based on the UH, the flood hydrograph of any given rainfall event, can be generated based on the principle of superpositioning as shown in Figure 2.3.

Model Representation of Rainfall

In order to address different hydrological modelling requirements, HEC-HMS includes a variety of different ways to model precipitation. For the development of the flood hazard zoning, the impact of floods with different statistical return periods is estimated. This approach is commonly used to design flood protection infrastructure in a way that it can handle a threshold flood with a specified return period. The return period is used to define the likelihood of flood or storm events. A flood with a RP of 100 years can be expected to occur once in a time span of 100 years. Since such a 100-year flood has a 1 % chance to occur in any given year, its annual exceedance probability (AEP) is 1 %. For instance, in the USA and Germany, major drainage-system elements are designed to resist a 100-year flood (Chin, 2006). If the river of interest is gaged, flood discharges for different AEPs can be determined using statistical analysis methods. For ungaged watercourses like the ones analysed in this study, it is a common approach to estimate the flood discharge for a specified AEP by modelling a design rainfall with the same AEP (HEC, 2000). In this approach, the definition of the storm duration and the distribution of the rainfall intensity within the specified duration are crucial since both parameters significantly influence the shape and magnitude of the resulting flood wave. Since intensity distribution patterns for different rainfall durations were not found for the area of investigation, the frequency-based hypothetical storm method included in HEC-HMS was used to create design storms with specified return period. This method is based on the alternating-block method which is described in detail by Chow et al. (1988). Hereby, design storms are generated based on IDF curves in a way that the amounts of rainfall for any time interval within the storm, centered around the peak intensity, have a consistent return period.

The input rain data for a frequency-based design storm with a return period of 100 years (100 years storm) are the precipitation depths for various rainfall durations with a 100-year return period which can be derived from IDF curves. IDF curves are created based on continuous precipitation data from a rain gauging station and represent the long-term precipitation characteristics of the area around the gauging station. The IDF curves used in this study are presented in Chapter 3. The development of design storms used for hydrologic modelling is shown in Chapter 4.

3.1.2 Hydraulic Model: HEC-RAS

HEC-RAS is a hydraulic modeling software developed by the U.S. Army Corps of Engineer´s Hydrologic Engineering Center. In this study, version 5.0.4 of HEC-RAS was used. The software is capable of performing one, two and one/two-dimensional (1D/2D) steady and unsteady-flow simulations. It comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities as well as graphics and reporting facilities (HEC, 2016). The HEC-RAS Technical Reference Manual (HEC, 2016) is a detailed and complete documentation of the model and the underlying equations. The following section however explains the basics of 1-D unsteady-flow routing as well as the most important features of the model that are necessary for the understanding of the applied methodology.

3.1.2.0 Basics of One-Dimensional Flow Routing

In inundation analysis, flow modelling is used to simulate the flow of a flood wave through a river reach and its floodplains. In hydraulics, the flow of water in a river is referred to as open channel or free-surface flow since the water surface is exposed to the atmosphere. For modelling and design purposes, continuity, momentum and energy equations have been developed in the past to represent open channel flow in a mathematical way. These equations are based on three basic laws of physics, which are the continuity of mass, the continuity of energy and the continuity of momentum. Flow models simulate the flow through an open channel in a way that satisfies these basic equations for open channel flow or simplified versions of them.

In one-dimensional flow routing, flow through the river channel and the floodplains is treated only in the longitudinal direction parallel to the conduit. Even though in reality, the flow in a natural channel is never truly 1-D, these flow models were found to deliver acceptable results for predicted hydraulic parameters in many applications (Arizona Department of Water Resources, 2002). In the 1-D HEC-RAS flow model, the geometry of the channel and the floodplains is represented by a series of cross sections along the reach.

In general, 1-D models are subdivided into steady and unsteady-flow models. In steady-flow simulations, a constant inflow is modelled so that the depth of flow at any specified location does not change over time. In comparison to that, a discharge hydrograph is applied as the inflow into the modelled reach in unsteady-flow simulations resulting in changes in depth at specified locations over time. The appropriate choice between these two basic approaches highly depends on the situation that is intended to be modelled. Since the purpose of this study is to identify flood prone areas and then delineate inundated areas and not to design detention pools upstream, steady-flow modelling was chosen in order to account for the area reached by water.

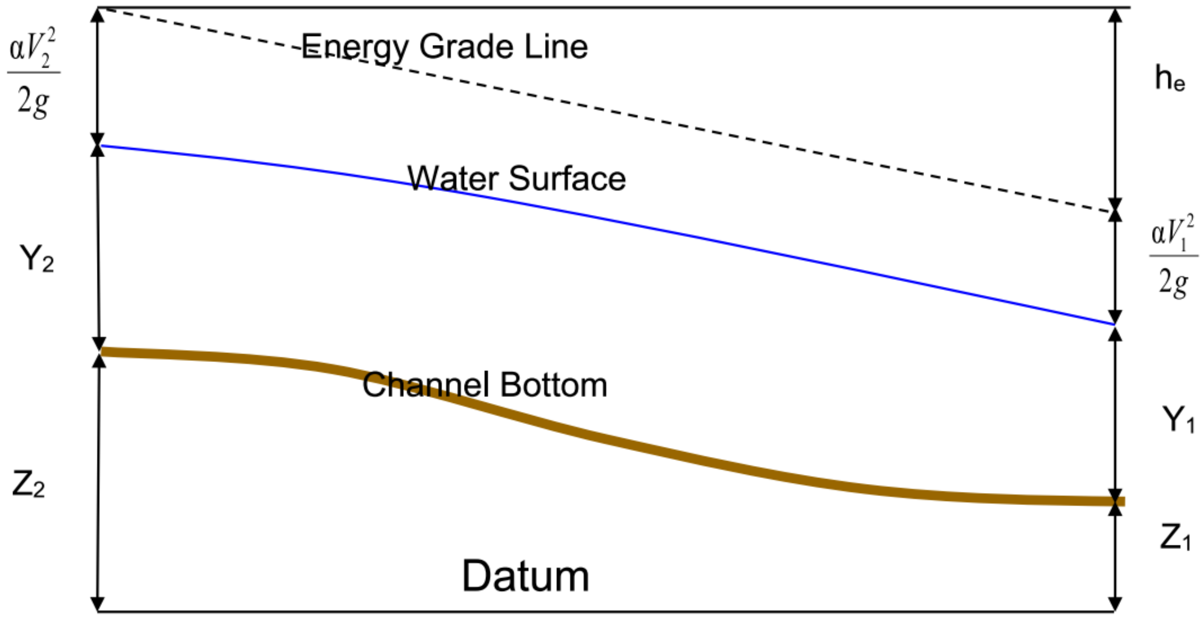


Figure 2: Representation of Terms in the Energy Equation (HEC, 2016)

Another relevant aspect that is important for the modelling of open channel flow is the distinction between subcritical and supercritical flow. This distinction is due to the fact, that for a fixed discharge, there is a critical flow depth, for which the specific energy of the flow is at its minimum. The specific energy of a flow is defined as the energy of the flow with reference to the channel bed as the datum and thus only depends on the depth and the velocity of the flow. If the specific energy is not at its minimum, there are two possible flow depths for a fixed discharge. If the flow depth is higher than the critical depth, the flow is subcritical, if it is lower the flow is supercritical. A practical distinction can be made based on the fact that in supercritical flow, a wave resulting from any type of disturbance cannot propagate upstream compared to subcritical flow. In consequence, water surface computations from one cross section to another are done in downstream direction for supercritical flow and upstream direction for subcritical flow. Situations in which both flow types occur in the modelled reach are referred to as a mixed-flow regime.

3.1.2.1 1D Steady Flow Water Surface Profiles

HEC-RAS is currently able of performing 1D water surface profile calculations for steady gradually varied flow in natural and constructed channels. Subcritical, supercritical and mixed-flow regime water surface profile can be calculated (HEC, 2016). Topics discussed in this section include equations for basic profile calculations; cross section subdivision for conveyance calculations; contraction and expansion losses; application of the momentum equation and limitations of the steady flow model.

a. Equations for Basic Profiles Calculations

Water surface profiles are computed from one cross section to the next by solving the energy equation with an iterative procedure called the standard step method. The energy equation is written as follows (from the above figure):

$$Z_2 + Y_2 + \frac{a2V2}{2g} = Z_1 + Y_1 + \frac{a1V1}{2g} + h_e$$

$$Z2 + Y2 + \frac{a2V2}{2g} = Z1 + Y1 + \frac{a1V1}{2g} + he$$

Where Z1, Z2 = elevation of the main channel inverts,

Y1, Y2 = depth of water at cross sections,

V1, V2 = average velocities (total discharge/ total flow area)

a1, a2 = velocity weighting coefficients,

g = gravitational acceleration,

h_e = energy head loss

The energy head loss (h_e) between two cross sections is comprised of friction losses and contraction or expansion losses. The equation for the energy head loss is as follows:

$$h_e = L\bar{S}_f + C\left|\frac{a2V2^2}{2g} - \frac{a1V1^2}{2g}\right|$$

where: L = discharge weighted reach length

$\bar{S}_f$ = representative friction slope between two sections

C = expansion or contraction loss coefficient

b. Cross Section Subdivision for conveyance Calculations

The determination of total conveyance and velocity coefficient for a cross section requires that flow be subdivided into units for the velocity is uniformly distributed. The approach used in

HEC-RAS is to subdivide flow in the **overbank** areas using the input cross section n-value break points (locations where n-values change) as the basis for subdivision (Figure…) Conveyance is calculated within each subdivision from the following form of manning's equation:

$$Q = K{S_f}^{1/2}$$

$$K = \frac{1}{n} A R^{2/3}$$

Where: K = conveyance for subdivision

n = Manning's roughness coefficient for subdivision

A = flow area for subdivision

R = hydraulic radius for subdivision (wetted perimeter)

S_f = slope of the energy gradeline

The program sums up all the incremental conveyances in the overbanks to obtain a conveyance for the left overbank and the right overbank. The main channel conveyance is normally computed as a single conveyance element. The total conveyance for the cross section is obtained by summing the three subdivision conveyances (left, channel, and right) (HEC, 2016).

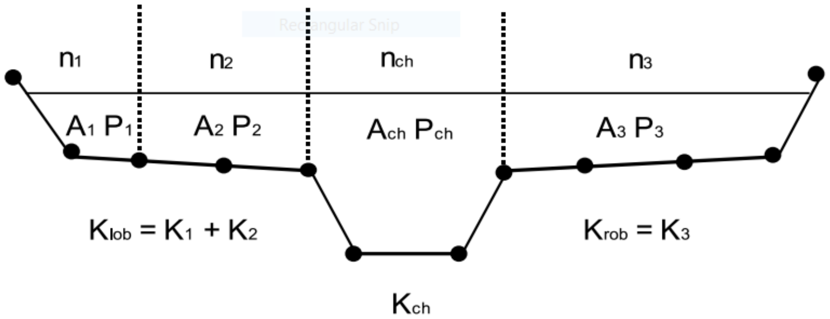


Figure 3: HEC-RAS Default Conveyance Subdivision Method (HEC,2016)

An alternative method available in HEC-RAS is to calculate conveyance between every coordinate point in the overbanks (Figure…). The conveyance is then summed to get the total left overbank and right overbank values. This method is used in Corps HEC-2 program. The method has been retained as an option within HEC-RAS in order to reproduce studies that were originally developed with HEC-2.

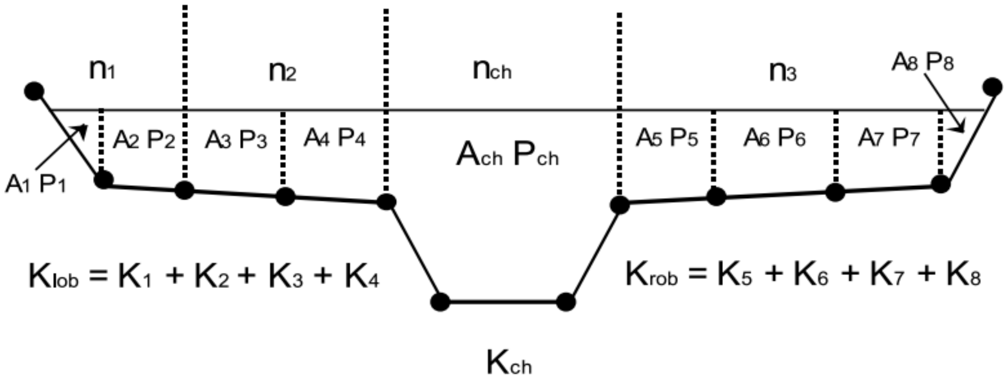


Figure 4: Alternative Conveyance Subdivision Method (HEC, 2016)

The two methods for computing conveyance will produce different answers whenever portions on the overbank have ground sections with significant vertical slopes. In general, the HEC-RAS default approach will provide o lower total conveyance for the same water surface elevation.

Contraction and Expansion Losses Evaluation

Contraction and expansion losses in HEC-RAS are evaluated by the following equation:

$$h_{ce} = C\left|\frac{a2V2^2}{2g} - \frac{a1V1^2}{2g}\right|$$

where: C = contraction or expansion coefficient

The program assumes that a contraction is occurring whenever the velocity head is downstream is greater than the velocity upstream. Likewise, when the velocity head upstream is greater than the velocity head downstream, the program assumes that a flow expansion is occurring (HEC, 2016).

Typical C value can be found in chapter, “Basic Data Requirements.”

c. Application of the Momentum Equation

Whenever the water passes through critical depth, the energy equation is not considered to be applicable. The energy equation is only applicable to gradually varied flow situations, and the transition from subcritical to supercritical or supercritical to subcritical is a rapidly varying flow situation. There are several instances when the transition from subcritical to supercritical and supercritical to subcritical flow can occur. These include significant changes in channel slope, bridge constrictions, drop structures and weirs, and stream junctions (HEC, 2016). In this work, the momentum equation has been chosen to compute the energy losses at the junction at the confluence of flow between the canals of" Rue minister" and "C2". In some of these instances empirical equation can be used (such as drop structures and weirs), while at others it is necessary to apply the momentum equation in order to obtain an answer.

Within HEC-RAS, the momentum equation can e applied for the following specific problems: the occurrence of a hydraulic jump, low flow hydraulics at bridges, and stream junctions. In order to understand how the momentum equation is being used to solve each of the three problems, a derivation of the momentum equation is shown here.

The momentum equation is derived from Newton's second law of motion:

Force = Mass x acceleration (change in momentum)

$$\sum F_x = ma$$

Applying Newton's second law of motion to a body of water enclosed by two cross sections at locations 1 and 2 (Figure…) the following expression for the change in momentum over a unit time can be written:

$$P_2 - P_2 + W_x - F_f = Q\rho\Delta V_x$$

Where: P = Hydrologic pressure force at locations 1 and 2.

W_x = Force due to the weight of water in the X direction.

F_f = Force due to external friction losses from 2 and 1.

Q = Discharge.

ρ = Density of water.

V_x = Change on velocity from 2 to 1, in the X direction.

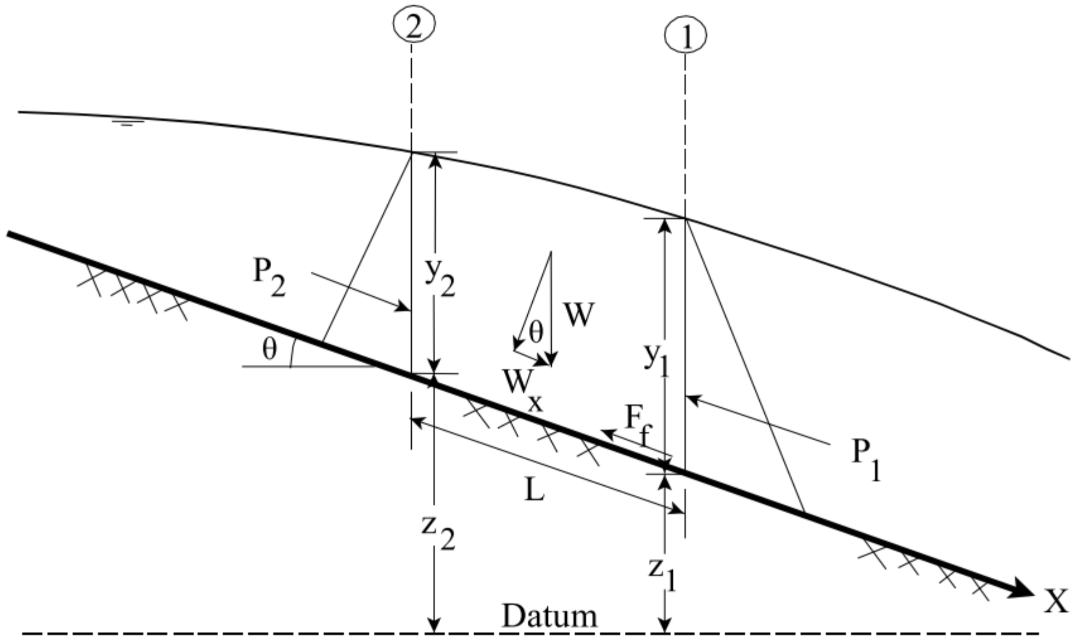


Figure 5: Application of the Momentum Principle

Hydrostatic pressure Force

The force in the X direction due to hydrostatic pressure is:

$$P = \gamma A \bar{Y} \cos\theta$$

The assumption of a hydrostatic pressure distribution is only valid for slopes less than 1:10. The $\cos\theta$ for a slope of 1:10 (approximately 6 degrees) is equal to 0.995. Because the ordinary channels slope is far less than 1:10, the $\cos\theta$ correction for depth can be set to equal to 1.0 (Chow, 1959). Therefore, the equations for the hydrostatic pressure force at section 1 and 2 are as follows:

$$P_1 = \gamma A_1 \bar{Y}_1$$

$$P_2 = \gamma A_2 \bar{Y}_2$$

Where: γ = Unit weight of water

A_i = Wetted area of the cross section at location 1 and 2

Y_{i} = Depth measured from water surface to the centroid of the cross sectional area at locations 1 and 2.

Weight of Water Force

Weight of water = (unit weight of water) x (volume of water)

$W = \gamma(\frac{A_1+A_2}{2})L$

$W_x = W \times \sin\theta$

$\sin\theta = \frac{Z_2 + Z_1}{L} = S_0$

$W_x = \gamma(\frac{A_1 + A_2}{2})L \times S_0$

Where: $L =$ Distance between section 1 and 2 along the X axis.

$S_0 =$ Slope of the channel, based on mean bed elevation

$Z_i =$ Mean bed elevation at locations 1 and 2

Force of External Friction

$$F_f = \tau\bar{P}L$$

Where: $\tau =$ Shear stress

$\bar{P} =$ Average wetted perimeter between sections 1 and 2

$$\tau = \gamma\bar{R}\bar{S_f}$$

Where: $\bar{R}$ = Average hydraulic radius (R=A/P)

$\bar{S_f}$ = Slope of the energy grade line (friction slope)

$$F_f = \gamma\frac{\bar{A}}{\bar{P}}\bar{S_f}\bar{P}L$$

$$F_f = \gamma(\frac{A_1 + A_2}{2})\bar{S_f}L$$

Mass time acceleration

$$ma = Q\rho\Delta Vx$$

$$\rho = \frac{\gamma}{g} \text{ and } \Delta Vx = (\beta_1 V_1 - \beta_2 V_2)$$

$$ma = Q\frac{\gamma}{g}(\beta_1 V_1 - \beta_2 V_2)$$

Where: β = Momentum coefficient that accounts for a varying velocity Distribution in irregular channels.

Substituting Back into Equation…and assuming Q can vary from 2 to 1

$$\gamma A_2\bar{Y}_2 - \gamma A_1\bar{Y}_1 + \gamma\left(\frac{A_1 + A_2}{2}\right)LS_0 - \gamma\left(\frac{A_1 + A_2}{2}\right)\bar{S}_f L = \frac{Q_1\gamma}{g}\beta_1 V_1 - \frac{Q_2\gamma}{g}\beta_2 V_2$$

$$\frac{Q_2\beta_2 V_2}{g} + A_2\bar{Y}_2 + \left(\frac{A_1 + A_2}{2}\right)LS_0 - \left(\frac{A_1 + A_2}{2}\right)L\bar{S}_f = \frac{Q_1\beta_1 V_1}{g} + A_1\bar{Y}_1$$

$$\frac{{Q_2}^2\beta_2}{gA_2} + A_2\bar{Y}_2 + \left(\frac{A_1 + A_2}{2}\right)LS_0 - \left(\frac{A_1 + A_2}{2}\right)L\bar{S}_f = \frac{{Q_1}^2\beta_1}{gA_1} + A_1\bar{Y}_1$$

Equation …is the functional form of the momentum equation that is used in HEC-RAS. All applications of the momentum equation within HEC-RAS are derived from equation…

1D Steady Flow Program Limitations

The following assumptions are implicit in the analytical expressions used in the current version of the program:

1. Flow is steady
2. Flow is gradually varied. (Except at hydraulic structures such as bridges, culverts, and weirs. At these locations, where the flow can be rapidly varied, the momentum equation or other empirical equations are used.)
3. Flow is one dimensional (i.e, velocity components in directions other than the direction of flow are not accounted for.)

4. River channels have "small" slopes, say less than 1:10.

Flow is assumed to be steady because time dependent terms are not included in the energy equation (equation …Bernoulli.). Flow is assumed to be gradually varied because Equation 2.1 is based on the premise that a hydrostatic pressure distribution exists at each cross section. At locations where the flow is rapidly varied, the program switches to the momentum equation or other empirical equations. Flow is assumed to be one-dimensional because Equation 2.19 (total energy head of a cross section: $H = WS + \frac{aV^2}{2g}$) is based on the premise that the total energy head is the same for all points in a cross section.

The limit on slopes as being less than 1:10 is based on fact that the true derivation of the energy equation computes the vertical pressure head as:

$$H_p = d\cos\theta$$

Where: H_p = vertical pressure head

d = depth of the water measured perpendicular to the channel bottom.

θ = channel bottom slope expressed in degrees.

For a channel bottom slope of 1:10 (5.71 degrees) or less, the $\cos(\theta)$ is 0.995. So instead of using $d\cos(\theta)$, the vertical pressure head is approximated as d and is used as the vertical depth of water. As you can see for a slope of 1:10 or less, this is a very small error in estimating the vertical depth (0.5%).

If HEC-RAS is used on steeper slopes, you must be aware of the error in the depth computation introduced by the magnitude of the slope. Below is the table of slopes and the $\cos(\theta)$:

Slope	Degrees	Cos (θ)
1:10	5.71	0.995
2:10	11.31	0.981
3:10	16.70	0.958
4:10	21.80	0.929
5:10	26.57	0.894

Table 2: Slopes and $cos(\theta)$ (HEC, 2016)

If you use HEC-RAS to perform the computations on slopes steeper than 1:10, you would need to divide the computed depth of water by the $\cos(\theta)$ in order to get the correct depth of water. Also, be aware that very steep slopes can introduce air entrainment into the flow, as well as other possible factors that may not be taken into account within HEC-RAS.

3.1.2.2 HEC-RAS justification in the context of Abidjan

Abidjan is a city where the system of waterways or creeks and channels are particularly dense and long of 137 km (the canal we are working on is about 2.4 km). These waterways are large in some places and present steep slopes which facilitate the drainage of stormwater towards various points of the lagoon and its bays. The buried network of drainage (conduits) is not very developed and all its tributaries or ramifications end in open channels.

In Abidjan, the most known historical events of flooding that caused more losses of lives and damages from stormwater were the result of drainage canal and drains overflow. In comparison, the damage which can be caused by the buried network remains relatively "minor" and are especially due to an inversion of the hydraulic gradient because of the overflow of the major open canal network. Dealing with the issues of stormwater drainage in Abidjan results mainly recalibrating and redesigning the major network of waterways which is supposed to cope with it. This can only be done by a modelling of this network, agreeing with the reality and allowing to simulate flowing conditions that are taking place in canals or creeks with a high reliability. HEC-RAS is indicated perfectly to achieve these goals. The stormwater management model (SWMM), for it's part simulates also open channels but requires a lot of data.

3.2 Models development

3.2.0 HEC-HMS

In this study, as mentioned initially, the ArcGIS extension of HEC-HMS, HEC-GeoHMS was used to preprocess the hydrologic model data. The DEM of Cocody of 30 m resolution resampled to 15 m has been applied in order to extract the physical characteristics of the watershed.

Before the extraction of the watershed characteristics, a land use map prepared on Envie 5.1 was combined with a soil map to generate the curve numbers of the subbasins of the watershed.

For the calibration of the model, we got discharge data for the sole June 19, 2018 flood event from the Gourou catchment project. It was just a test for them to measure those data. Therefore, in lack of observed discharges, the validation was done indirectly by comparing the flood extent from HEC-RAS and HEC-GeoRAS to the may 11, 2018 flood event observed by the modeller.

3.2.0.0 HEC-HMS inputs preparation

For a watershed to respond to a rainfall event on HEC-HMS model, this should include a basin model, a meteorological model and a control specification.

This software thus allows the modeller to use different types of methods for the modelling basins and rain. The control specification allows to specify for each case the start date and end date of the simulation and the time step of the.

3.2.0.1 Pre-processing and watershed physical extraction

Kouamé et al. provided soil characteristics and detailed explanation on their granulometry. and this put together with the NRCS hydrologic soil group allowed the computation of the CN in ArcGIS.

Table 4: Soil texture (Kouamé et al., 2011)

TYPES DE SOL	% Sable	% Limon	% Argile	Texture

Table 5: Hydrologic Soil Group (NRCS, 1986)

HSG	*Soil textures*
A	Sand, loamy sand, or sandy loam
B	Silt loam or loam
C	Sandy clay loam
D	Clay loam, silty clay loam, sandy clay, silty clay, or clay

The NRCS gave four hydrologic soil groups for the computation of the CN. The groups are shown below in the table:

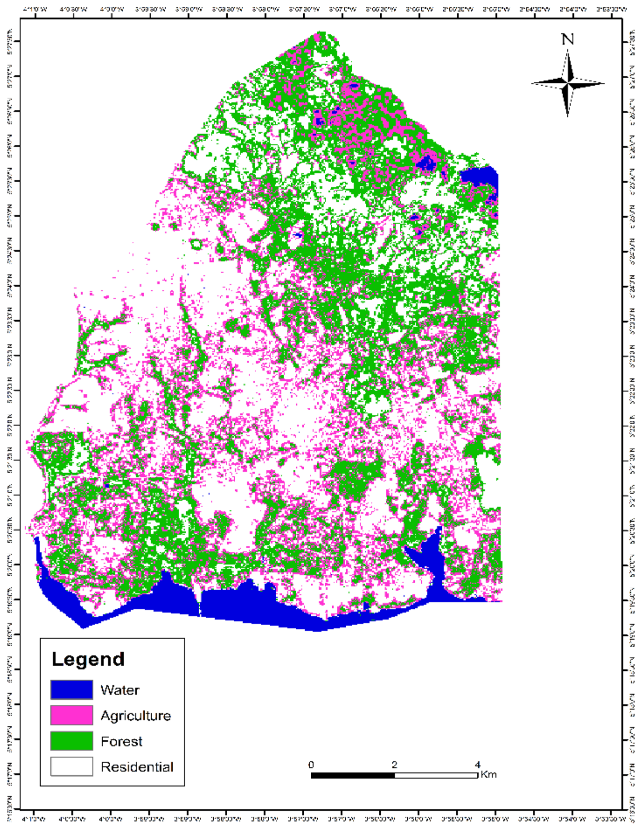
In the TR-55, these four hydrologic soil groups are described regarding their infiltration rate and main characteristics as follows (NRCS, 1986):

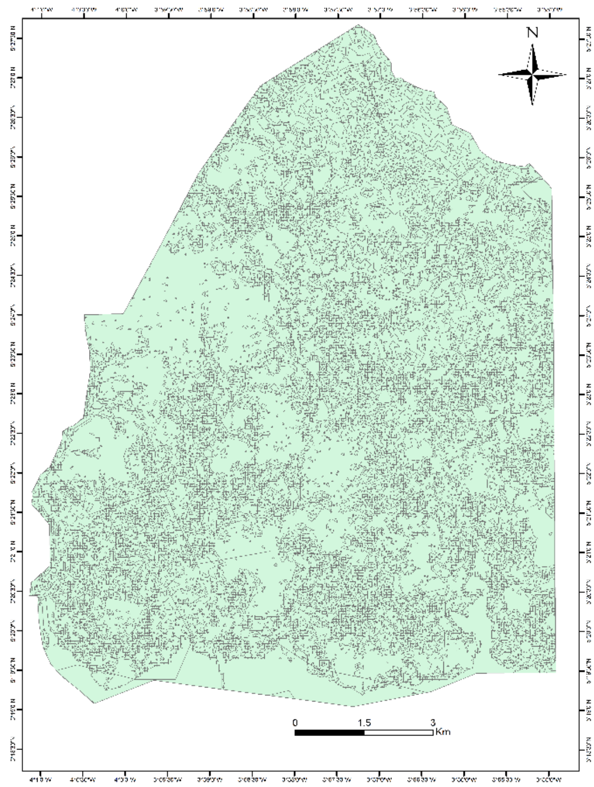
Group A soils have the highest infiltration rates and consist chiefly of deep, well to excessively drained sand or gravel.

Group B soils have moderate infiltration rates and consist of moderately well to well drained soils with moderately fine to moderately coarse textures.
Group C soils have low infiltration rates and consist of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture.
Group D soils have very low infiltration rates and consist chiefly of clay soils with a high swelling potential.

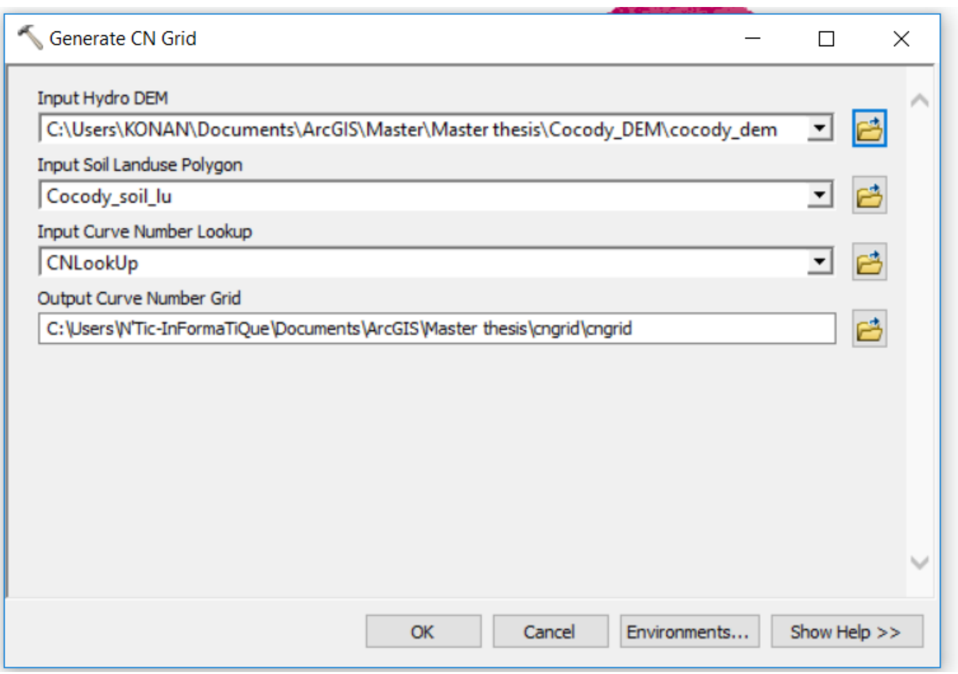
Considering the above information and added to the soil file we got from the BNETD, the soil of the watershed was characterized and further provided all the necessary information to generate the CN.
We have al the four groups present in the watershed. Abidjan is located in the south of the country as said earlier, and this part is mainly sandy but mixed to some clay and loam. This has been mentioned in the description of the geology of the study area.

The step was the processing of the land use, soil data and the DEM to generate the CN. The land use was reclassified and converted into polygon so that it can be merged with the soil data.

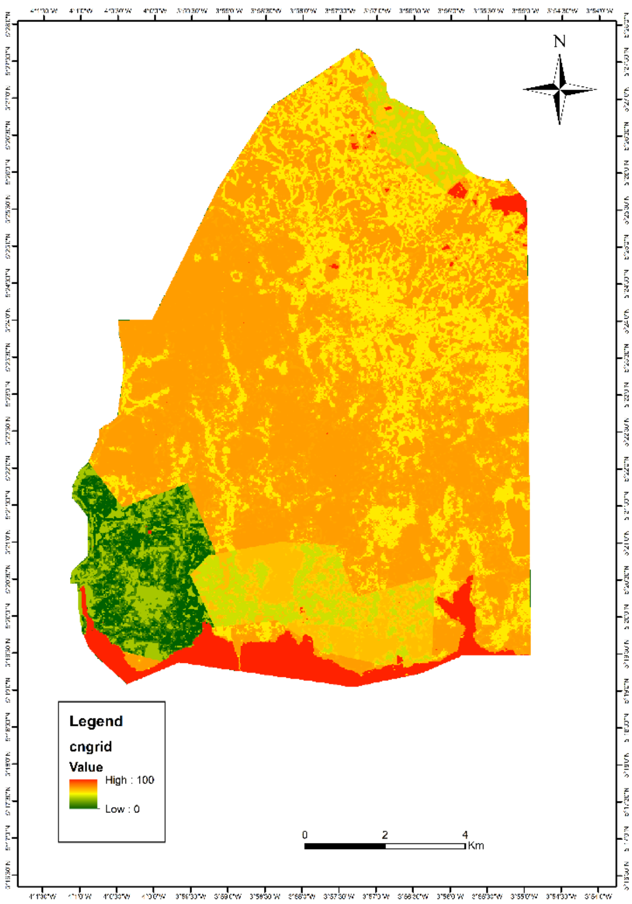


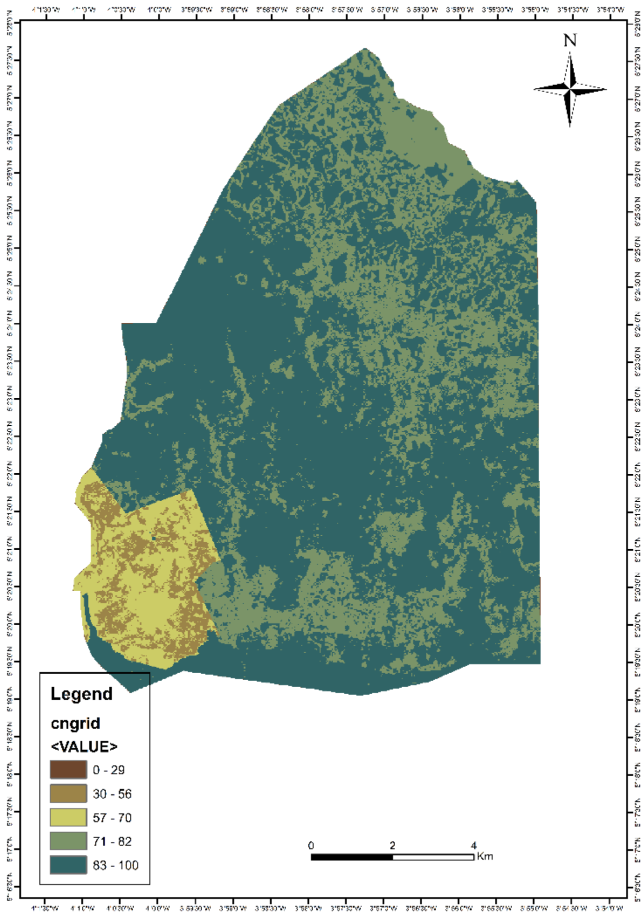


The land use was reclassified in four groups before merging it with the soil data. Ones they are merged, they are combined with the DEM to generate the CN numbers.

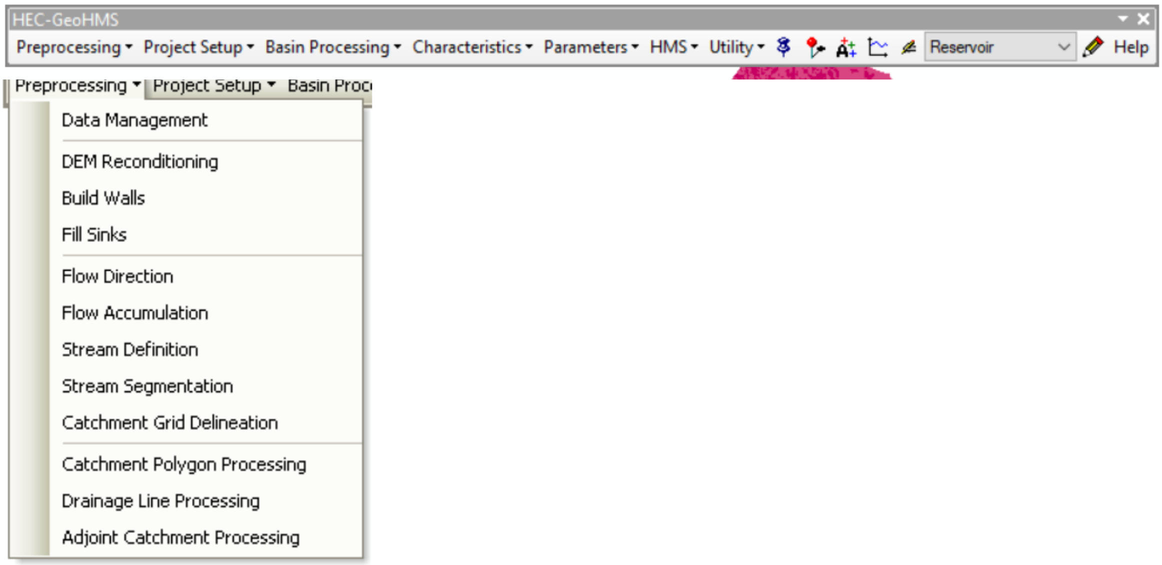


The CN lie between 82 and 86. This is understandable as we are in an urban area. It means that this watershed is likely to generate much runoff.





Watershed physical characteristics extraction is known as terrain pre-processing. To do it, the following steps are performed successively using HEC-GeoHMS in ArcGIS. This also can be done using ArcHydro tools in ArcGIS.



The basin model was created by applying the HEC-geoHMS software functionality within the ArcMap GIS environment. The first major step in creating the basin model was to delineate the stream network and the watershed boundaries of the area of interest. This process is commonly referred to as terrain preprocessing as mentioned earlier and is entirely based on the input DEM. The following grid files were derived from the DEM by following the step by step functionality of HEC-HMS.

- **Fill Sinks grid:** This function creates a depressionless or hydrologically corrected DEM based on the input DEM. Therefore, the software automatically increases the elevation value of any pit cell to the level of the surrounding terrain.

- **Flow Direction grid:** This grid is delineated from the Fill Sinks grid. In the grid processing, the direction of the steepest descent to a neighbour cell is defined for each grid cell.

- **Flow Accumulation grid:** This grid is delineated from the flow direction grid and defines the number of upstream cells draining into any given cell in the grid.

- **Stream Definition grid:** In this step, the cells that form the stream network are defined based on a threshold number of cells that drain into a given cell. In this analysis

the threshold for the definition of streams was set to 1 km2
respectively. The result is a grid, in which the stream network is represented by lines of connected grid cells that all fulfils the threshold criteria.

- **Stream Segmentation grid:** This grid is created by splitting the streams as defined in the stream definition grid at any junction.

- **Catchment grid:** For every stream segment defined by the stream segmentation grid, the corresponding watershed is delineated and stored in a grid file.

To complete the terrain processing, the vector layers were created based on the outcomes of the previous computational steps:

- **Catchment Polygons**: This function uses the catchment grid to delineate the boundaries of each subbasin in the form of a vector layer.

- **Drainage Line:** The stream segments defined by the stream segmentation grid are transformed into a vector stream layer by this function.

- **Adjoint Catchment:** In this step, the upstream subbasins are aggregated at any stream confluence. This step is not hydrologically relevant but enhances the computational performance in subsequent steps.

A project point has to be defined after the completion of the terrain preprocessing. The project point defines the outlet of the watershed that is intended to be modelled and thus has to be placed on a drainage line. Based on the outcomes of the terrain preprocessing and the definition of the project point, HEC-HMS delineates the project area and creates all necessary layer files for this area. All the created data is stored in a new geodatabase.

The resulting watershed has a total area of 37.47 km2 and includes 8 subbasins with a minimum area of 0.011 km2.

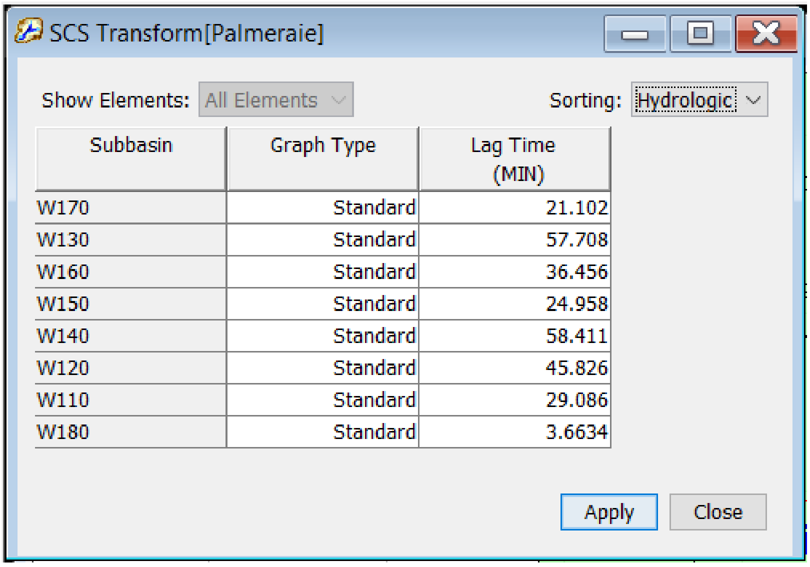
For each of the resulting stream segments and the related subbasins, a serious of physically based characteristics were computed based on the depressionless DEM by using the

HECgeoHMS functions. These characteristics include the lengths and slopes of each river segment as well as the average basin slope and the longest flowpath of each subbasin. The resulting data is automatically stored in the attribute table of the river and subbasin layer.

Figure on the following page shows the extend of the outlet and the 8 subbasins as a result of the previously described methodology in detail. For each subbasin, the flow channel as well as the longest flow path is illustrated.

As mentioned in Chapter 2, the hydrologic modelling was based on the NRCS Curve Number loss method, the NRCS Unit Hydrograph transform method and the Muskingum routing method.

The computation of the CN has been described in this chapter, and the review of literature gave some details on the method of unit hydrograph. For the Muskingum routing method part, it is based on some work done in Abidjan and also based on the knowledge of the study area on one hand and on the other hand on the results of the terrain processing. The lag times of the sbubasins were computed during the process and are low. The figure below is the illustration:



Subbasin	Graph Type	Lag Time (MIN)
W170	Standard	21.102
W130	Standard	57.708
W160	Standard	36.456
W150	Standard	24.958
W140	Standard	58.411
W120	Standard	45.826
W110	Standard	29.086
W180	Standard	3.6634

Figure 6: Lag times computed in ArcGIS

The NRCS shew that $T_{Lag} = 0.6T_c$, where T_{Lag} is the lag time and T_c is the time of concentration. It means that the T_c will also be low, this show that water flows rapidly on the watershed and then from the knowledge of the watershed, Muskingum K and X were estimated.

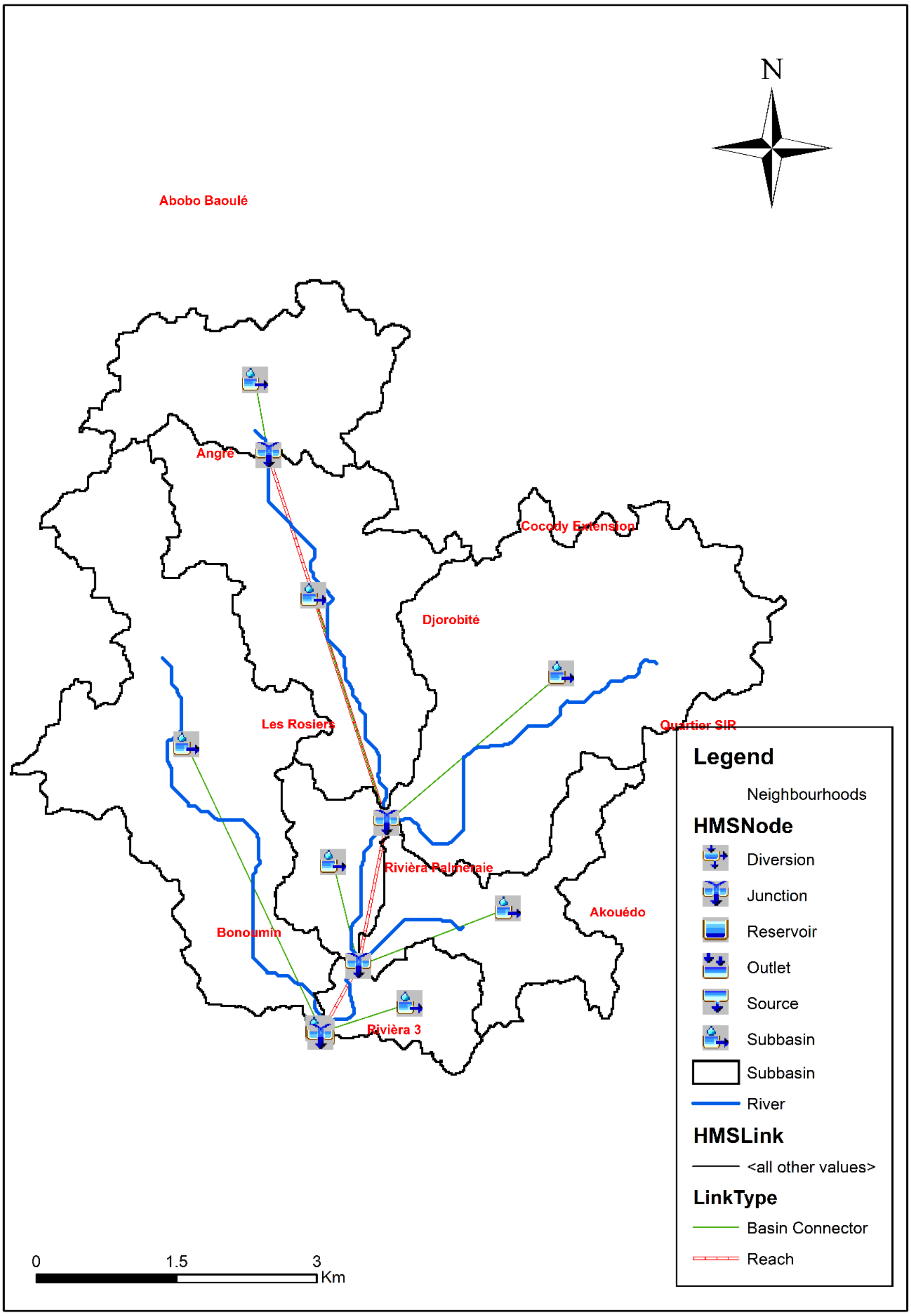


Figure 7: Basins model of Bonoumin and Rivièra watershed

3.2.0.3 Import of the GeoHMS results to HEC-HMS

After extracting all the characteristics of the watershed and computing some HEC-HMS input parameters, the next step is to import this project to HEC-HMS for it to be finalised and prepared for simulations. The main input that is to be entered now is the precipitation. Whatever it is just a water depth or rainfall intensity. So, the GeoHMS results were imported to HEC-HMS, the figure below shows details:

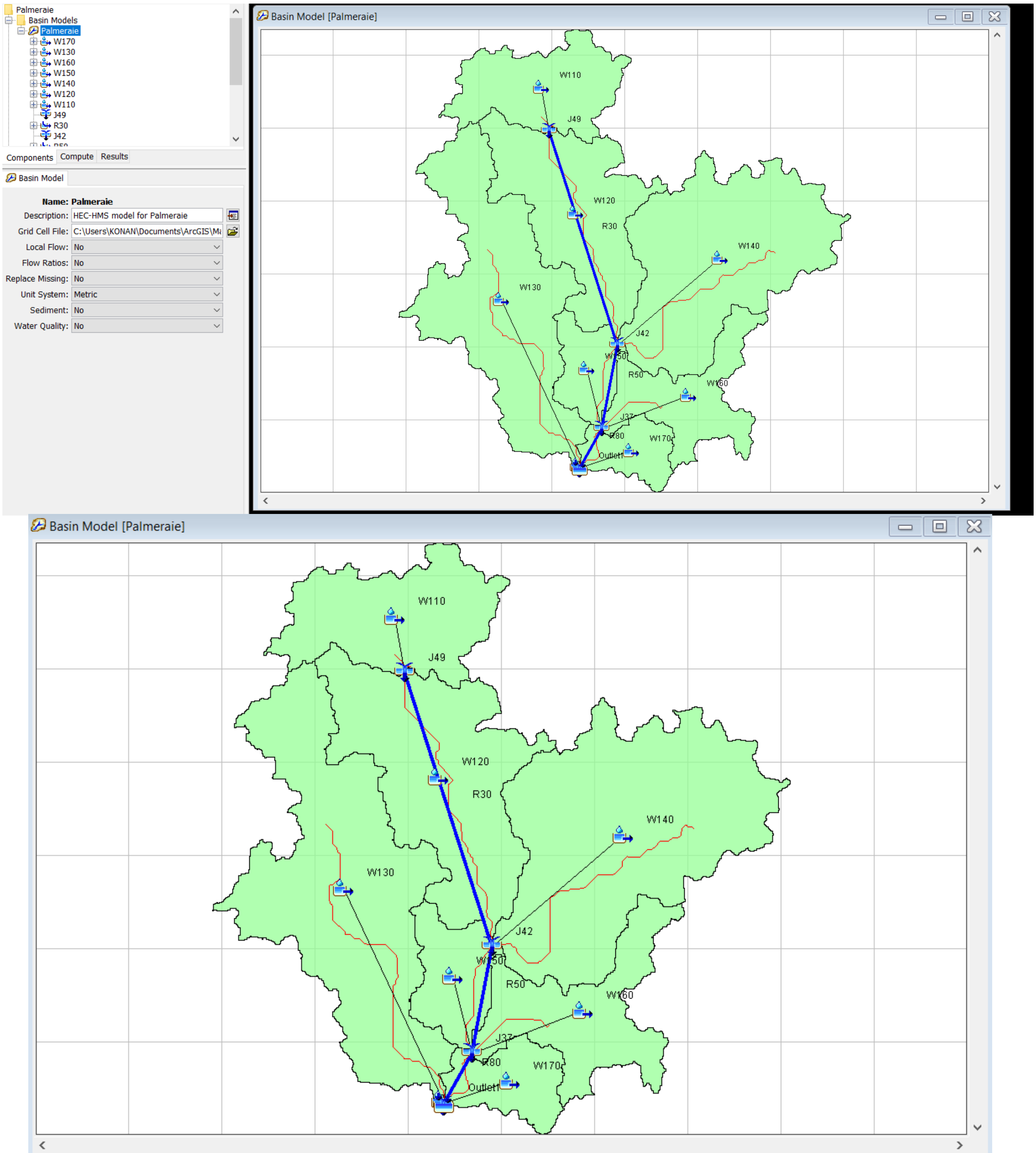


Figure 8:Basin Model in HEC-HMS

3.2.1 HEC-RAS

As it was not possible for the canal, subject of this study, to appear on the TIN of the watershed because the precision was not so good, AutoCAD data and field measurements were combined to establish the canal geometric data. Indeed, HEC-GeoRAS needs a topographic data of good precision. Therefore, the DTM or TIN should not have a resolution less than 10 m. but one can work with DTM of 30 m resolution for natural rivers. Topographic data constitute the basis on which all hydraulic modelling works are based. It is thus very essential and crucial to pay particularly attention to the method of procuring those data. Several works highlight the importance of precision of topographic data: Casas al. (2006) quoted by Geoffroy (2007) put ahead topographic precision like the "most critical factor" when hydraulic modelling is being carried out. In this article, they tested various topographies on one section of a river of 2 km using HEC-RAS software by varying the different parameters (coefficient of Manning-Strickler and flow). They compare the results (extent or inundated areas and average water surface elevation) starting from a topography of reference defined thanks to a ground survey using a GPS and a bathymetric survey. Sensitivity analysis carried out on the coefficients of Manning confirms the idea according to which more the flow is important less the model is sensitive to the variations of these coefficients.

But in this study, due to urbanization, the natural waterway has been modified and then has been built in a canal with variable dimensions (cross sections). A field measurement campaign was initiated in order to get the profiles of the canal

3.2.1.0 Model Geometry creation

3.2.1.1 Model Completion in HEC-RAS (grand titre)

a. Manning's n-Value

Selection of an appropriate value for Manning's n is very significant to the accuracy of the computed water surface elevations. The Manning's n is highly variable and depends on a number of factors including: surface roughness; vegetation, channel irregularities, channel alignment; scour and deposition; obstruction; size and shape of the channel; stage and discharge; seasonal changes; temperature; and suspended material and bedload (HEC, 2016).

There are several references a user can access that show Manning's n values for typical channels. An extensive compilation of n values for streams and floodplains can be found in Chow's "Open-Channel Hydraulics" (Chow, 1959).

In this work, the canal is in concrete and then the Manning's n values are known ($K_s = 70$; then $n = \frac{1}{70} = 0.014$). Except the floodplains which are of variable constitutes: asphalt road and earth.

Type of Channel and Description	Minimum	Normal	Maximum
C. *Excavated or Dredged Channels*			
1. **Earth, straight and uniform**			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. **Earth, winding and sluggish**			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. **Dragline-excavated or dredged**			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. **Rock cuts**			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. **Channels not maintained, weeds and brush**			
a. Clean bottom, brush on sides	0.040	0.050	0.080
b. Same as above, highest stage of flow	0.045	0.070	0.110
c. Dense weeds, high as flow depth	0.050	0.080	0.120

Figure 9: Manning's n values (Chow, 1959)

b. Expansion and Contraction Coefficient

These loss coefficients are applied in the hydraulic computations in order to account for energy losses resulting from contraction and expansion of flow due to changes in cross section geometry along the reach. The energy loss caused by a transition in channel geometry is calculated by multiplying these coefficients by the absolute difference in velocity head between one cross section and the next downstream cross section (HEC, 2016). For gradual transitions and supercritical flow, HEC (2016) suggests to use values of 0.01 for contraction and 0.03 for expansion. When the change in river cross section is small, and the flow is subcritical, coefficients of contraction and expansion are typically on the order of 0.1 and 0.3 respectively. When the change in effective cross section area is abrupt such as at bridges, contraction and expansion coefficients of 0.3 and 0.5 are often used. After a first simulation the flow in the canal has been found to be a mixed-flow regime with supercritical flow prevailing. At the junction (Carrefour commissariat), the flow is supercritical and there is a culvert just after the junction. Then, values of 0.01 for contraction and 0.03 for expansion were used for the computation of the losses.

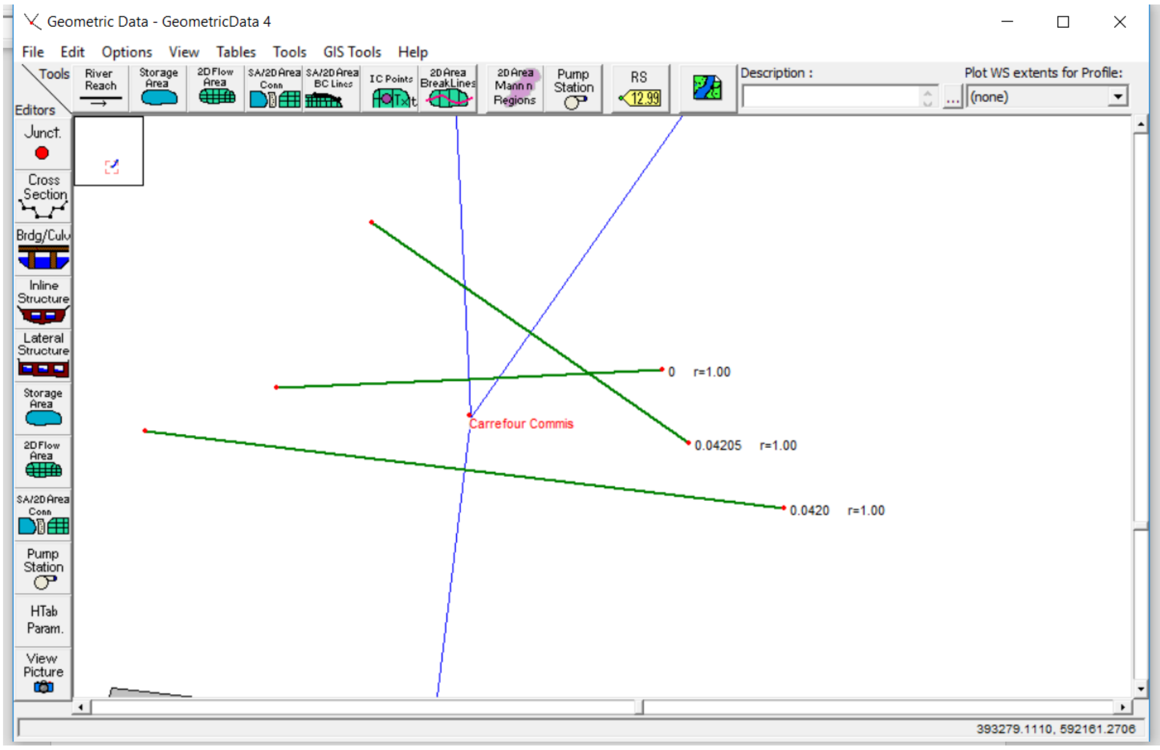


Figure 10: Junction at Carrefour Commissariat

In HEC-RAS, for steady flow hydraulic computations, a junction can be modelled by either the energy equation or the momentum equation. The energy equation does not take into account the angle of any tributary coming in or leaving the main stream, while the momentum equation does (HEC, 2016). In most cases, the amount of energy loss due to the angle of the tributary flow is not significant and using the energy equation to model the junction is more than adequate. However, there are situations where the angle of the tributary can cause significant energy losses. In these situations, it would be more appropriate to use the momentum approach. Therefore, it is safer to use the momentum approach here as not only the different angles are known but also serous hydraulic jumps take place at the junction during rain events.

c. Flow Data and Boundary Condition

Steady flow data are required for performing a steady water surface profile calculation. Steady flow data consist of: flow regime; boundary conditions; and discharge information (peak flow or flow data from a specific instance location).

The flow regime (subcritical, supercritical, or mixed flow regime) is specified on the steady flow analysis window of the user interface. In cases where the flow will pass from subcritical to supercritical, or supercritical to subcritical, the program should be run in a mixed.

At the beginning of the research, the relevant literature on urban floods, flood modelling and flood hazard mapping will be reviewed again to acquire more knowledge about conventional and current methodologies.

To achieve the objectives of the research, the following methodology will be used

- ✓ Data collection:

Data to be used in this study are elevation data (topographic, DEM could be used), land use and crops maps and climate data from public agencies. The climate data include daily rainfall, daily and minimum-maximum and monthly stream flow data. Solar radiation, wind speed and relative humidity. The climatic data can be got from the Ivorian meteorological service (SODEXAM). Population data and constructed infrastructures for water (waste water and water supply) are needed.

- ✓ Data analysis

Data analysis consists of checking the consistency of climate data, estimate missing data, and build input files for hydrologic/hydraulic model. Hydrodynamic model will be simulated to understand the physical phenomena of flooding. Remote sensed precipitation data will be averaged over a certain area and have different spatial resolution. So it will be necessary to do a transformation in order to be able to compare them and to match them with study area. A weather generator will also be used to transform monthly statistic climate data into daily time series. The simulation period depends on stream available data.

- ✓ Model set up

The model to be used in this research will be HEC-HMS coupled with HEC-GeoHMS for hydrologic modelling and HEC-RAS with HEC-GeoRAS for hydraulic modelling. They will be combined with ArcGIS for watershed delineation, streams definition and subbasins classification from DEM, land use and soil maps, and climate data and also with remote sensing techniques.

Chapter 4: Expected results

In this section results of model simulating will be presented. Before model simulation, global data sets will be compared to ground station precipitation in the same periods of simulation and at the same location. No attempt will be done to interpolate data in order to improve spatial resolution except the estimation of areal precipitation of a sub basin of the study area.

After the model has finished the steady or unsteady flow computations one can begin to view the output. Output is available in a graphical and tabular format. The current version of the program allows us to view cross sections, water surface profiles, general profiles, rating curves, X-Y-Z perspective plots, hydrographs, detailed tabular output at a single location, and summary tabular output at many cross sections. Additionally, as my model is georeferenced, I will also create inundation maps, and perform animations of inundated areas within HEC-RAS Mapper (HEC-GeoRAS combined with ArcGIS). Floodplains delineation will also be performed. Then the extent and the magnitude of flood will be known.

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