HYDRODYNAMIC INVESTIGATION AND NUMERICAL SIMULATION OF INTERMITTENT AND EPHEMERAL FLOWS IN SEMI-ARID REGIONS: WADI MEKERRA, ALGERIA

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Semi-arid regions are characterized by important infrequent rainfall. They often occur in early autumn and give rise to devastating floods. Flooding problems at Wadi Mekerra, located in the Sidi Bel Abbes town (Northwest Algeria), was traditionally the main concern of researchers and government officials. In this work, the magnitude of raging flood wave in the studied catchment and the principal causes are discussed. After this, we present the main hydromorphometric features and the results of numerical simulations of flood wave. This simulation is done by using finite volume shock capturing schemes. It concerns applying the first order Godunov scheme and the second order Monotonic Central scheme. The Manning roughness coefficient was used as a calibration parameter. The comparison of numerical results with observed data confirms more stability and accuracy of applied numerical schemes in rising limb phase than in the falling limb phase. These results provide information on flood forecasting and monitoring of changes in the magnitude of the flow in Wadi Mekerra.

KEY WORDS: Semi-arid, Temporary Flows, Wadi Mekerra, Flood Wave, Saint-Venant Equations, Finite Volume, Shock Capturing.

Khaled Korichi, Abdelkrim Hazzab: HYDRODYNAMICKÝ VÝSKUM A NUMERICKÁ SIMULÁCIA OBČASNÝCH A KRÁTKODOBÝCH PRIETOKOV V SEMIARIDNÝCH OBLASTIACH: WADI MEKERRA, ALŽÍR. J. Hydrol. Hydromech., 60, 2012, 2; 114 lit., 13 obr., 7 tab.

Semiaridné oblasti sú charakterizované významnými občasnými dažďami. Vyskytujú sa spravidla v jeseň a spôsobujú devastačné povodne. Povodne vo Wadi Mekerra v meste Sidi Bel Abbes (severozápadné Alžírsko) sú často problémom pre výskumníkov a vládnych úradníkov. Práca analyzuje devastačné povodne v tomto povodí. Okrem toho sa uvádzajú najdôležitejšie hydromorfomertrické charakteristiky a výsledky simulácií povodňovej vlny. Simulácia je aplikáciou Godunovovej schémy prvého rádu a Monotonickkej centrálnej schémy druhého rádu. Ako kalibračný parameter bol použitý Manningov súčiniteľ drsnosti. Porovnanie simulovaných a pozorovaných údajov potvrdzuje väčšiu stabilitu a presnosť počas rastúcej vetvy prietokovej vlny, ako je tomu v klesajúcej fáze. Tieto výsledky sú informáciou o možnostiach predpovede a monitoringu povodní vo Wadi Mekerra.

KEÚČOVÉ SLOVÁ: semi-aridný, občasné povodne, Wadi Mekerra, povodňová vlna, rovnice Saint-Venanta, konečný objem, pohltenie nárazu.

Introduction

Almost all natural phenomena can produce disasters; however flooding is the most important in terms of lost materials and human lives (Mudd, 2006; Huang et al., 2010). Prediction and control of damage caused by these disasters require special identification of vulnerable areas and determination of factors amplifying the magnitude of generated

damage (Hansson et al., 2008; Jonkman et al., 2008).

The inundations risk is considered as the product of the occurrence probability of floods and consequences associated with these events (Raaijmakers et al., 2008). By possessing a diagram of the situation, it would be possible not only to manage risk, take the most appropriate steps to eliminate loss in life and minimize property loss, but also have di-

verse opportunities to provide the appropriate dimensioning of hydraulic structures (Breton and Marche, 2001).

The estimated impact of floods and the risk assessment that follows are issues for which the literature provides different analysis approaches (Gregory et al., 1993; Haase et al., 2003; Olfert et Schanze, 2008). The used tools for the floods treatment are flow modelling, analysis of flood frequency and evaluation of potential annual cost of damage (Boyle et al., 1998).

Semi-arid regions are characterized by infrequent rainfall but sometimes very important (Colombani et al., 1984; Pedro et al., 2006). These cloudbursts often give rise to flooding whose consequences are sometimes unforeseen (Hansson et al., 2008). However, streams in semi arid areas are characterized by irregular flow and a strong hydrologic fluctuation (Arab et al., 2004). During the year, intermittent flows dominate in most Wadis (Argyroudi et al., 2009). A mismatch between lotic and lentic conditions, is thus noticed (Morais et al., 2004).

It should be noted that this type of region where most rivers are ephemeral, do not have, to date a global treatment approach. This is justified by many reasons. (a) The lack of data that represents a handicap to study of changes in hydrological balance (Sivapalan et al., 2003). (b) The occasional change of climate and anthropic conditions, which leads to an imbalance in the hydrologic fluxes and physical characteristics of the basin (Puigdefabregas and Mendizabal, 1998; Xoplaki et al., 2004; López-Moreno et al., 2007). (c) The high variability of ephemeral events and the hydrology of dryland zones. This implies a strong nonlinearity of explored models (Nouh, 2006; Hreiche et al., 2007; Kingumbi et al., 2007; Lajili-Ghezal, 2007; McIntyre et al., 2007; Nasri, 2007). (d) Spatial heterogeneity generates a gradual change in the hydraulic capacity. So flood risks are confirmed (Lebdi et al., 2006).

Historical hydrology (Barriendos and Rodrigo, 2006; Brázdil and Kundzewicz, 2006) represents the best tool that can respond to treatment of these issues. The effort is focused on the methodology. Thus, robust methods are designed to ensure a compromise between the applicability and accuracy, especially in rainfall-runoff modelling field, which represents a major preoccupation in arid and semi-arid catchments (Camarasa and Tilford, 2002; Benkaci and Dechemi, 2004; Nasri et al., 2004; Cudennec et al., 2005; Hreiche et al., 2007; Moussa et al., 2007; McIntyre et al., 2007; Ouachani et al.,

2007; Chaponnière et al., 2007; (Pilgrim et al., 1988; Hughes, 1995; Sharma and Murthy, 1998; Wheeler et al., 2006). Other approaches propose the coupling of hydrological, hydraulic models and statistical analysis. This is especially the combination of surface and groundwater hydrology (Rozos et al., 2004; Bouri et al., 2007; Kingumbi et al., 2007), or between the stochastic and dynamic approaches (Hreiche et al., 2007; Lajili-Ghezal, 2007).

Recent works have been performed to calibrate and validate various models for Wadis whose flow regime is intermittent or even ephemeral (cf. Tab. 1). One cites innovative studies distributed in such semi-arid zones. It is about of hydrological model developed for Australian drylands (Costelloe et al., 2005), or hydro-biological model for the same type of zone in Brazil (Pedro et al., (2006). in the Middle East, Alexandrov et al., (2007) quantify the sediment transport in the Negev watershed. Modern techniques involve the use of Digital Elevation Models (DEM) for flood forecasting by parameterizing algorithms for two-dimensional hydrodynamic flood simulation (Sanders, 2007).

In Algeria, we can refer to the studies of Dechemi et al., (1994) in the Tafna watershed and the works of Achit and Ouillon (2007) for the Wadi Abd.

By tracking all these methods, numerical modelling of transient and complexes free surface flows can be explored as an interesting approach. Horritt and Bates (2002) present an example of a two-dimensional numerical model of flood wave propagation. Mathematically, floods waves can be governed by the Saint-Venant model which is a nonlinear hyperbolic system of partial differential equations. Nonlinear problems represent shock and rarefaction waves. To overcome the handicap in the treatment of non-linearity, several solving strategies have been developed (Liggett and Cunge, 1975; Cunge et al., 1980; Guinot, 2010).

Among all numerical approaches, the shock capturing finite volume methods have several advantages, since they combine between the simplicity of the finite difference method and the flexibility of the finite element method (Godunov, 1959; Lax and Wenderoff, 1960; Van Leer, 1977; Leveque, 2004 and Toro, 2006).

However, in the treatment of real problems (eg: channel with complex geometry), some schemes still need to overcome numerous difficulties (Wang et al., 2000). For instance, the treatment of the

Table 1. Targeted papers relevant to flood wave in Wadis and rivers.

Reference	River basin or region	Type	Main contribution
<i>Cherif et al. (2009)</i>	Wadi Mekerra, Algeria	Ephemeral	Statistical Study
<i>Dechemi et al. (1994)</i>	Tafna Wadi, Algeria	Intermittent	Statistical study
<i>Horritt and Bates (2002)</i>	Severn river, UK	–	2D Numerical model
<i>Nasri et al. (2004)</i>	El Gouazine and Dekekira, Tunisia	Intermittent	Flood impact
<i>Costelloe et al. (2005)</i>	Lake Eyre, Australia	Both	Hydrological models
<i>Pedro et al. (2006)</i>	Taperoá, Brasil	Intermittent	Hydro-biological model
<i>Alexandrov et al. (2007)</i>	Eshtemoa, Negev	Ephemeral	Sediment transport
<i>Achit and Ouillon (2007)</i>	Wadi Adb, Algeria	Intermittent	Sediment transport
<i>Bracken et al. (2008)</i>	Nogalte and, Torrealvilla, Spain	Both	Rainfall and flood
<i>Sanders (2007)</i>	Santa Clara River	–	DEM model

source term is one of the difficulties that may result in significant numerical error in such cases. This has been a problem that many researchers are seeking to solve (*Leveque, 1998; Garcia-Navarro and Vasquez-Cendon 2000; Zhou et al., 2001*) and more recently (*Roger et al., 2003; Audusse et al. 2004; Benkhaldoun et al. 2007; Kesserwani and Liang 2010; Liang and Wang 2010; Lee and Wright 2010*).

Sidi Bel Abbes town makes part of semi-arid regions. It is faced to floods phenomena and inundations that occur in a catastrophic manner (*Stucky, 2005*). These floods are frequent, recurrent and present a major constraint on economic and social development (*DGPC, 2008*). The management of this phenomenon presents a particular concern for public authorities. The objective is to conceive a hydraulic management project that responds to the city protection requirements against floods while preserving the benefits arising from the plain's immersion through an important contribution of natural fertilizers elements and the aquifers filling.

Having regard to the importance of these accentuated flooding, several preventive works have been proposed. Certain concern the study of the flooding phenomenon in Mekerra catchment (*Borsali et al., 2005*). Currently, the works of SAFEGE group constitute a promising start in the implementation of an alert system which provides flood and also for the delimitation of risk zones.

In this paper, we present numerical simulation of a flood wave in October 1995, at Wadi Mekerra. This contribution aims to identify potential risk areas, the wave magnitude and the time required to alert threatened agglomerations. The numerical simulation is achieved by applying finite volume methods. It is about of shock capturing first-order Godunov scheme and Monotonic Central second

order scheme. These schemes have proved their ability to obtain robust, stable and monotone solutions (*Ying et al., 2004; George, 2006; Begnudelli, 2007*). Such approach is based on solving the Saint-Venant equations by numerical treatment of Riemann problem. The simulation results are promising and confirm the stability and accuracy of these numerical schemes.

The remainder of the paper is organized as follows. “Study area” presents a global description and General information about the study site, flow regime and the chronology of floods generated at Wadi Mekerra. “Mathematical model and methods” presents governing equations of Saint-Venant model including numerical methods in which Godunov scheme and Monotonic Central scheme are briefly presented. “Numerical tests” presents a series of Dam failure and flood wave tests in order to verify and validate the model. We end by a Summary and Conclusion.

Study area: Wadi Mekerra

General information

The Wadi Mekerra is the main stream in the watershed which bears the same name. This latter is part of the great Macta basin. It occupies a great part of Sidi Bel Abbes town (Northwest of Algeria). It is located between the latitudes 34°31'N and 35°21'N and longitudes 1°16'O and 0°58'O (cf. Fig. 1). The Wadi has its source in the high valleys of the steppe at an altitude of around 1250 m. To the right of Sidi Bel Abbes city, the Wadi drains an area of 3000 km² and develops a talweg of 115 km in length. The catchment is elongated and north-south oriented. Its relief is mountainous in the hills. The absolute heights of the watershed crest are

1000 to 1100 m in the north, 1200 m in the oust, 1200 to 1260 m in the south and 870 to 1460 m at the east of Mekerra basin. 48% of the catchment area has an elevation above 1000 m, wich explains the torrential streams character. The average slope of Wadi is 5.5%. The Mekerra watershed is divided into three sub-basins (Hacaiba, Sidi Ali Benyoub and Sidi Bel Abbes city). Each one has a gauging

station. Tab. 2 summarizes the main hydromorphometric characteristics of each sub-basin.

The Mekerra catchment is divided into three hydrological areas that manage the hydrological response of the Wadi. It is about of the floods production zone at the upstream, discharges transfer zone in middle and expansion zone in downstream.

Fig. 1. Location of the Mekerra watershed and its major hydrometric stations.

T a b l e 2. Main hydromorphometric features of the Mekerra sub-basins.

Features	Symbol	Unit	Haçaiba Station (1946)	Sidi Ali Benyoub Station (1949)	Sidi Bel Abbes Station (1918)
Surface	S	[km ²]	957	1890	3000
Compactness index	K _C	[km]	1.15	1.29	1.43
Characteristic height	H _{max}	[m]	1440	1715	1714
	H _{min}	[m]	925	635	437
Roche Slope index	I _p	[%]	0.099	0.0936	0.0913
Wadi's length	L	[km]	54.00	92.00	115.00
Drainage density	D _d	[-]	0.06	0.050	0.02
Torentiallity	C _T	[-]	0.20	31.25	6.86
Time of concentration	T _c	[h]	6.369	10.560	13.448

Vegetation and geology

Across the Mekerra watershed, only 25% of sub-soil is impermeable. The northern part is much more permeable than the southern part. Tab. 3 resumes the land cover in the Mekerra watershed

(ANAT, 2000). It is thus noted that the vegetation cover in the basin is developed in an irregular manner. At the downstream of Ras El Ma city appear lands covered with Alfa, which towards the north give way to brushland. At the upstream, there are cereals, vines and citrus occupying Sidi Ali

Benyoub zone. Forests cover over 20% of the basin area (DGPC, 2000).

The influence study of the vegetation on the layout and the stream aspect has been described by Filliol (Filliol, 1954). Depending to rates of banks integral fixation or bed partial fixation by vegetation, it results in lateral erosion processes or depth erosion. There's also the factor of flow velocity influence. According Martinez-Mena et al., (1999) and Rachman et al., (2003), vegetation cover can reduce the rainfall kinetic energy at 50% and the runoff power to 75%.

Table 3. Mekerra basin landcover.

Landcover	Surfaces [km ²]	% surfaces
Forest and maquis	500.3	27.61
Annual cultures	649.1	35.82
Alfa	194.1	10.71
Agglomerations	21.16	1.17
Soil conservation works	7.56	0.42
Unproductive land	50.80	2.8
Route	103.9	5.73

Three main sets of soil dominate the Mekerra catchment. It is about of calcic brown soil, calcareous soil and alluvial soil. The first two types belong to the magnesium-calcic soil class. These soils are deeper, having a structure generally well developed granular or lumpy type in the surface horizon and polyhedral in the depth (Bouasria et al., 2010). On the agricultural front, their complex adsorbent is saturated with calcium and magnesium, the pH is above 6.8. The alluvial soils belong to the poorly evolved class. These soils are encountered on the Wadi terraces and characterized by a poorly differentiated and low-structured profile. This explains that these soils are good for agriculture (Askri et al., 1993).

Climate, rainfall and runoff

Situated in the semi-arid climatic region, the Mekerra catchment is characterized by hot dry summers spanning from April to August and a relatively mild winter and wet from October to March. The average annual temperature is around 15°C and average interannual day of frost is about 35 days. Prevailing winds are from northwest and west. The interannual average speed is about 20 m s⁻¹. Rainfall is irregular, characterized by intense autumnal rains causing major floods (cf. Fig. 2) whose effects are felt between Boukhanifis and Sidi Bel Abbas

agglomeration. During wet years the amount of rainfall may exceed 800 mm (cf. Fig. 3). In dry years, it decreases to 110 mm. The average interannual rainfall is around 400 mm.

Fig. 2. Variability of mean monthly precipitation and water discharge in Wadi Mekerra (Sidi Bel Abbas gauge Station, 1942–2001).

Fig. 3. Variability of annual precipitation (top) and mean annual water discharge (down) in Wadi Mekerra (Sidi Bel Abbas gauge Station, 1942–2001).

Fig. 3 shows in particular a strong flow fluctuations during the period (1942 to 2001) in which the Wadi Mekerra has passed through two long dry periods. The first lasts for nine (9) years (1952 to 1962) where the mean annual discharge was 0.221 m³ s⁻¹. The second period has lasted seventeen (17) years from 1968 to 1985 with an average discharge

of about $0.141 \text{ m}^3 \text{ s}^{-1}$. It is interesting to note that the lack of pluviometric inflows observed from the mid-seventies, confirms the drought in North Africa over the past four decades (*Tardy and Probst*, 1992). This lack is manifested in a dramatic decrease in water supplies drained by Maghrebi Wadis. Compared with normal flows, a decrease of 67% is observed in the North-West of Algeria (*Meddi et al.*, 1998).

Floods of Wadi Mekerra

Overview

Inundation's problem generated by cyclical floods of Wadi Mekerra has always constituted one of the main concerns of the managers of Sidi Bel Abbes city. Further to observations made during the various floods recorded, we can relate it to several causes, namely: (1) Intense and irregular cloudburst at high Mekerra can reach 200 mm h^{-1} . (2) Morphology of Mekerra watershed, particularly elongate, where time of concentration is very low ($T_c = 9$ hour). (3) Narrowing of the Wadi section at some stretches and hydraulic structures. (4) Almost-total obstruction of the Wadi's sections by sediments carried and deposited by anterior floods. (5) The downstream affluents have very low slopes if not negative. (6) Unplanned urbanization in the banks of the Wadi (DGPC, 2000). This factor affects seriously the magnitude of floods. Indeed, for the semi-arid basin (Wadi Medjerda in Tunisia) *Lebdi et al.*, (2006); *Zahar and Albergel* (2006) have shown that people's sensitization affects the vulnerability of flood risks.

Tab. 4 sums up the main damage, victims, injured and homeless, caused by the floods in Wadi Mekerra due by thunderstorms and heavy rains during twenty five years (1978–2000). One can add, agricultural land inundation, undermining of the Wadi banks and vegetation degradation.

Fig. 4 shows the major floods in Wadi Mekerra presented by peak discharges between (1942–2001). We observe in particular that the magnitude of floods has increased remarkably during the decade (1990–2001). Taking into account observations of 34577 instantaneous discharges for the period (1942–2001), spread over the three gauging stations (Hacaiba, Sidi Ali Benyoub and Sidi Bel Abbes) a statistical study showed that the class of dominants discharges is that less than $10 \text{ m}^3 \text{ s}^{-1}$. While the high discharges (more than $200 \text{ m}^3 \text{ s}^{-1}$) represents only 0.01% (*Stucky*, 2005) (cf. Tab. 5).

The daily flow duration curve (cf. Fig. 5), shows a very pronounced concavity reflecting the irregularity of the flow regime. A special flow discharges are shown in Tab. 6.

Flow regime

The flow regime of the Wadi Mekerra was investigated based on mean daily discharges recorded at the station of Sidi Bel Abbes for 59 years (1942–2001). The average number of days without flow reaches 328 days per year (90% of the year). This indicates the intermittent nature of the Wadi. In referring to the literature for the flow regimes characterization of Wadis in this type of region, we note the existence of several classifications. Some of them cover the world, others are continental

Table 4. Flood statistics in Sidi Bel Abbes City (1978–2000).

Inundated area [ha]	Location	Date	Cause	Number of sinister	
				Dead	Homeless families
160	Ain El Berd	02/10/78	Thunderstorm	–	08
24	Ben Badis	06/03/80	Heavy precipitation	–	08
10	Boukhanifis	06/03/80	Heavy precipitation	–	04
Not estimated	Sidi Bel Abbes	04/10/86	Thunderstorm	01	200
Not estimated	Sidi Bel Abbes, Telagh	30/04/90	Thunderstorm	02	130
Not estimated	Sidi Bel Abbes, Boukhanifis, Sidi Lahcen, Hassi Zahana, Hassi Dahou, Oued Sefioun	29/09/94	Thunderstorm	01	22
Not estimated	Sidi Hamadouche, Ain El Berd	16/10/94	Thunderstorm	–	70
Not estimated	Sidi Bel Abbes, Sfiseif, Ain El Berd	05/12/95	Thunderstorm	–	03
Not estimated	Hassi Zahan	13/06/96	Thunderstorm	–	02
Not estimated	Moulay Slissen	17/08/97	Thunderstorm	01	34
Not estimated	Sidi Bel Abbes	27/09/97	Thunderstorm	01	–
Not estimated	Sidi Bel Abbes	13/12/97	Thunderstorm	01	05
Not estimated	Ras El Ma	27/07/00	Thunderstorm	–	100

Fig. 4. Peak annual discharges recorded at Wadi Mekerra (1942–2001).

T a b l e 5. Distribution in [%] of floods in Wadi Mekerra (1942–2001).

Classes of discharges [$\text{m}^3 \text{s}^{-1}$]	> 200	200–100	100–50	50–10	<10
Frequency [%]	0.01	0.5	1.44	5.29	92.77

T a b l e 6. Special discharge recorded at Wadi Mekerra (1942–2001).

Duration [days/year]	Max flow	>10	>30	>90	>180	>270	>355	Low flow
Discharge [$\text{m}^3 \text{s}^{-1}$]	1.473	1.357	0.869	0.318	0.142	0.065	0.003	0.022

Fig. 5. Daily flow duration curve for Wadi Mekerra (1942–2001).

(Smakhtin, 2001). These classifications are based on several hydrological parameters. Poff and Ward (1989) have developed a conceptual model based on a hierarchical classification of the main components of the flow regime, namely (intermittency and flooding frequency). In tropical regions of Australia, Molière et al., (2009) has developed a classification analysis based on discharge variability, the flood regime model and the intermittency indexes. This analysis indicates that rivers could be classified as permanent, seasonal dry-seasonal and sea-

sonal-intermittent. Oueslati et al. (2010) note that the stations in eastern and southern parts of the Mediterranean basin are more intermittent and predictable in compared to those located in west and north. In fact, identified works for the case of Algeria's catchments, confirm this finding. As such, we cite the studies of Terfous et al., (2001) in Wadi Mouilah; Wadi Mina in eastern Algeria (Achite and Meddi, 2005); Wadi Abd (Achite and Ouillon, 2007) in western Algeria and Wadi Haddad in central Algeria (Touaibia et al., 2001).

In our case, the data exploration confirms the ephemeral character of Wadi Mekerra throughout the studied period except between 1997 to 1999 where he was intermittent. Further, we find that the flow regime of Wadi Mekerra hardly reflects the intensity of the heavy rains. Because, a high daily discharge does not necessarily result in a high mean annual discharge. This was well observed in 1997 (cf. Tab. 7) where the total volume flowed during the flood was $2,64.10^6 \text{ m}^3$ corresponding to a maximum instantaneous discharge reaching $170 \text{ m}^3 \text{ s}^{-1}$. While in 1999 the total volume flowed during the flood was $2,63.10^6 \text{ m}^3$ corresponding to a maximum instantaneous discharge which was about $90 \text{ m}^3 \text{ s}^{-1}$.

Floods are more frequent and severe in autumn than in spring. They are characterized by a rapid rising in flood phase and a slower and regularized

descent during the recession. This regularization is justified by the spreading fields in the downstream. In the recession phase, the water in spreading extents and the water stored in the alluvium (banks and bed) return by overflow into the streams. On the contrary, water stored in the alluvial part of the Wadi basin assists to the prolongation of the flood recession and contributes toward the extension of low flow during the initial phase of the dry period. The recharge is relatively slow, which favors the filling of the groundwater table and the settling of natural fertilizers elements. Peak discharges are larger in high Mekerra (Sidi Ali Benyoub) than in its lower part (Sidi Bel Abbes City), where there has been a flat spotting (lamination or natural regulation) of the flood wave (Sadeg, 2003).

Table 7. Main total volumes flowed at Wadi Mekerra.

Flood date	Maximum instantaneous discharge		Total volume flowed during the flood [10^6 m^3]	Mean total volume flowed in year [10^6 m^3]	[%] flood volume relative to mean annual volume
	Gross [$\text{m}^3 \text{ s}^{-1}$]	Specific [l/s/km^2]			
29/12/1950	110	60.71	6.52	22.30	29.24
14/04/1954	100	55.19	5.65	9.14	61.82
04/10/1986	104	57.40	4.82	12.89	37.39
15/03/1991	135	74.50	9.29	11.88	78.20
27/09/1994	215	118.65	4.78	30.94	15.45
22/10/1995	154	84.99	7.99	17.41	45.89
05/02/1996	151	83.33	8.64	20.94	41.26
16/09/1997	170	93.82	2.64	42.79	6.17
27/09/1999	89.88	49.60	2.63	13.67	19.24
24/10/2000	161.8	89.29	6.85	20.45	33.50
10/10/2001	111.17	61.35	5.19	23.12	22.45

Mathematical model and methods

Governing equations

One-dimensional unsteady flows in a natural river (*Wadi*) with irregular cross-section may be described by the Saint-Venant equations (*Saint-Venant*, 1871) which is derived from the mass and momentum conservation principles based on hydrostatic pressure distribution of incompressible fluid. The conservative matrix form of these equations is written as:

$$\frac{\partial U}{\partial t} + \frac{\partial F}{\partial x} = S(U) \quad (1)$$

where U , $F(U)$ and $S(U)$ are respectively, the vectors of conserved variables, fluxes and sources term, defined as follows:

$$U = \begin{bmatrix} A \\ Q \end{bmatrix}, F(U) = \begin{bmatrix} Q \\ \frac{Q^2}{A} \end{bmatrix} \text{ and} \quad (2)$$

$$S(U) = \begin{bmatrix} 0 \\ -gA \frac{\partial z}{\partial x} - g \frac{n^2 Q^2}{AR_h^{4/3}} \end{bmatrix}$$

where A – cross-sectional area [m^2], Q – discharge [$\text{m}^3 \text{ s}^{-1}$], g – gravitational acceleration [m s^{-2}]; Z – water level [m] (cf. Fig. 6), n – Manning coefficient [$\text{s m}^{-1/3}$], $R = A/P$ – hydraulic radius and P – wetted perimeter of the channel [m]. The above form of the Saint-Venant equations is commonly used in engineering practice.

Fig. 6. Definition sketch of cross-sectional geometry.

Shallow water equations represent a nonlinear hyperbolic system of PDEs, where the Jacobian matrix

$$J = \frac{\partial F(U)}{\partial U} = \begin{pmatrix} 0 & 1 \\ -u^2 + gh & 2u \end{pmatrix}$$

is diagonalisable with real and distinct eigenvalues $\lambda^1 = u - \sqrt{gh}$ and $\lambda^2 = u + \sqrt{gh}$, u is the average flow velocity [$\text{m}^2 \text{s}^{-1}$]. The nonlinearity in the Saint-Venant system presents shock and rarefaction waves.

Numerical methods

Among all the numerical techniques, the finite volume method is well adapted to treat conservation, hyperbolic and nonlinear problems such as Saint-Venant equations. This method is based on the discretization of the integral form, by subdividing the domain in a number of finite volumes (cells). The approximate conserved variable U_i^n represents the average value over each cell. It is defined at the cell centres as

$$U_i^n \approx \frac{1}{\Delta x} \int_{C_i} U(x, t_n) dx \quad (3)$$

And the fluxes $F_{i-1/2}^n$ and $F_{i+1/2}^n$ are respectively calculated at the cells interfaces $x_{i-1/2}$ and $x_{i+1/2}$ (cf. Fig. 7). Integrating Eq. (1) over the i -th cell with length of Δx_i yields:

$$U_i^{n+1} = U_i^n - \frac{\Delta t}{\Delta x_i} [F_{i+1/2}^n - F_{i-1/2}^n] + \Delta t S_i. \quad (4)$$

Fig. 7. Definition sketch of cell centered grid.

There are several approaches to evaluate the flux at the interfaces that construct various conservative numerical methods (Ying et al., 2003). We are interested in this work to shock capturing schemes based on solving Riemann problem.

The solution of the Riemann problem is the key ingredient of Godunov type scheme, the solution of the Riemann problem is computed by what is called a Riemann Solver, it is considered as a tool for the computation of the fluxes at the interfaces between the computational cells. Riemann problem represents (cf. Fig. 8) an initial value problem and defined as follows:

$$U(x, 0) = \begin{cases} U_l & \text{if } x < 0 \\ U_r & \text{if } x > 0 \end{cases} \quad (5)$$

Fig. 8. Definition sketch of Riemann problem.

In the literature, there are several types of Riemann solver; one can cite exact Riemann solver, but also approximate Riemann solvers that are easy to implement such as HHL solver (Harten et al., 1983) and Roe Riemann solver (Roe, 1981). Roe's solver, used in this study, is probably the most widely used Riemann solver to date. The basic idea is to convert hyperbolic system of conservation law to an equivalent linear one that is easier to solve by replacing the exact Jacobian in each interval by a constant Jacobian as follows:

$$\hat{A}_{i-1/2} = \begin{bmatrix} 0 & 1 \\ -\hat{u}^2 + g\bar{h} & 2\hat{u} \end{bmatrix}$$

where

$$\bar{h} = \frac{1}{2}(h_{i-1} + h_i), \quad \hat{u} = \frac{\sqrt{h_{i-1}}u_{i-1} + \sqrt{h_i}u_i}{\sqrt{h_{i-1}} + \sqrt{h_i}}$$

Godunov's approach indicated by the Eq. (4) is the basis of choc capturing methods (Godunov, 1959) where the conserved variable is approximated by a piecewise constant function (cf. Fig. 9a). In Godunov's scheme, the left- and right-states of the Riemann problem are taken from the average values of the variable over the cell on the left- and right-hand side of the interface respectively.

Higher-order Godunov-type schemes use a reconstruction procedure to estimate the variations of the conserved variable in the calculation cells. The variable in a given cell is reconstructed using the

average values in the neighboring cells. This allows the gradients to be located more accurately, thus leading to more accurate estimates of the fluxes.

Most second-order schemes use a linear reconstruction of the conserved variable (cf. Fig. 9b) as:

$$\tilde{U}_i^n = U_i^n + (x - x_i)a_i^n \quad (6)$$

The slope a_i^n of the profile within cell i is calculated as the average slope between cells $i-1$ and $i+1$.

By limiting the slope of the reconstructed profile, spurious oscillations are eliminated from the solution. In fact, the contributions of the gradient of the variable are limited using a so-called limiter, for which many formulations have been proposed in the literature.

In this study, we are interested in Monotonic Central (MC) scheme. This choice is justified by numerous previous numerical tests (Korichi, 2006). MC limiter function is given by (Guinot, 2010):

$$\Phi(\theta) = \max\left(0, \min\left(\frac{(1+\theta)}{2}, 2, 2\theta\right)\right), \quad (7)$$

where Φ is the limiting function and θ represent the monotony indicator given by:

$$\theta_{i+1/2}^n = \begin{cases} \frac{U_i^n - U_{i-1}^n}{U_{i+1}^n - U_i^n} & \text{if } CFL \geq 0 \\ \frac{U_{i+2}^n - U_{i+1}^n}{U_{i+1}^n - U_i^n} & \text{if } CFL \leq 0 \end{cases}$$

Fig. 9. Finite volume discretization (a) Piecewise constant (Godunov) (b) Piecewise linear (Monotonic Central).

Fractional step method is used for the treatment of source term. The boundary conditions are treated by checking the flow regime. Which is verified by a simple calculation of the so-called Froude number $Fr = \frac{u}{(gh)^{1/2}}$. It is defined as the ratio of flow velocity u to celerity c of the waves in still water. When the flow is subcritical at a given boundary, only one condition is required. When the flow is supercritical entering the domain, two boundary conditions are needed. When the flow is supercritical leaving the domain, no boundary condition is required. Such approach is sufficient for finite volume method (Sanders, 2001).

Numerical tests

The primary objective of these tests is to verify the reliability of Godunov scheme and compare it with a MC scheme. The comparative study includes the examination of unsteady flow. It is a dam break test on a wet bottom in a smooth rectangular channel. These two schemes are checked using an analytical solution. The other application is a one-dimensional flood wave simulation in Wadi Meker-

ra (October 1995) by applying the selected scheme. For calibration, we vary Manning coefficient to observe the influence of roughness on the numerical results. The flood hydrograph is used to validate the results.

Dam failure test

The first test is an examination of a dam break wave on wet bottom in a rectangular, smooth and horizontal channel. One admits an initial time step $\Delta t = 0.1s$ for a simulation period of 60s. The domain length is 2000m divided into 2000 pieces. The rupture point is located at $x = 1000m$. The upstream water depth is initially $h_l = 10m$, while the initial water depth at downstream $h_r = 5m$. The initial discharge is null.

At the end of the simulation ($t = 60s$) we observe that the Godunov scheme diffuse near discontinuities, while the MC scheme has more stability and precision (cf. Fig. 10).

Fig. 10. Comparison of different schemes for dam break test (a) Depth, (b) Discharge.

Simulation of flood wave at Wadi Mekerra in October 1995

Used data

Based on the results of the first test, the selected scheme (Monotonic Central) is applied to simulate

the flood wave dated on 11–12 October 1995. The channel length is about 44 km from Sidi Ali Benyoub station to Sidi Bel Abbes station. Fig. (11) shows a simplified longitudinal profile of Wadi Mekerra between these two stations where the stream is treated as a prismatic rectangular channel of width $b = 18.6m$. The boundary conditions are

represented by a flood hydrograph observed at Sidi Ali Benyoub station while fixing the water depth in downstream at 0.6m.

To understand the influence of frictional forces on the flood wave shape, three Manning coefficients considered in this study are ($n = 0.02, 0.03$ and $0.04 \text{ s m}^{-1/3}$); this choice is justified by the variability in the Wadi bed and the basin Landcover. Complete lists of the Manning coefficient, n were

established by Crausse (Crausse, 1951), Chow (Chow, 1959) and Graf (Graf, 1984) which ranges in general from 0.012 and 0.15. For earthen and natural channels the Manning coefficient is between $0.02 < n < 0.05$. The simulated results are compared with recorded measurements at the downstream station.

Fig. 11. Simulated longitudinal profile of Wadi Mekerra between Sidi Ali Benyoub and Sidi Bel Abbes stations.

The flow is initially uniform with a discharge $Q_0 = 1.02 \text{ m}^3 \text{ s}^{-1}$; while the maximum discharge recorded during this flood is $155 \text{ m}^3 \text{ s}^{-1}$ through total time $t = 48$ hours. The studied domain is divided into 100 cells. To keep the Courant number close to one, the time step is adapted as:

$$\Delta t = \frac{CFL}{\Delta x} (|u| + \sqrt{gh})$$

Analysis and comments

Observations extracted from the results (cf. Fig. 13) show that the flood strength decreases according the increase in the Manning coefficients. It is direct consequence of the frictional forces that damp the impact of a high velocity gradient. Tab. 8 summarizes the main results. One notes that for Manning coefficient $n = 0.02$, the maximum discharge is largely overestimated with an error of 0.07% and an advancement of 15 minutes. For Manning coefficient $n = 0.03$ and 0.04 , the maximum flow rates are respectively 121.3 and $116.28 \text{ m}^3 \text{ s}^{-1}$ but the rise time of flood wave is retarded. It reached 2.64 hours for the second case.

Table 8. Comparison between the simulated and recorded results for different coefficient of Manning.

	Simulated			Recorded at Sidi Bel Abbes station
	$n = 0.02$	$n = 0.03$	$n = 0.04$	
Flood rising time [h]	5.76	7.68	8.64	6
Difference in flood rising time [h]	-0.24	+1.68	+2.64	-
$Q_{\max} [\text{m}^3 \text{ s}^{-1}]$	124.4	121.3	116.28	116
Error (Q) [%]	7	4	3	-

The roughness influence is clearly established. However, the spatial distribution of roughness in the study area represents a major handicap for the numerical simulation of floods. The channel flow resistance is largely determined by the presence of dunes that usually develop in the beds. This resistance varies depending on the nature and topography of the Wadi, if it is minor bed (main channel) or major bed (floodplain). Udo et al., (2007) shows the relationship between roughness, expressed by the Chézy coefficient and discharge (cf. Fig. 12). They thus prove that the resistance in the main channel is much greater than that in the floodplain. Relationship (roughness, flow) is more linear in floodplains (Perumal et al., 2004).

In transient flows cases, flood modelling seems being complicated. In some hydraulic models, the roughness coefficient is directly related to the dunes sizes that are dimensioned by empirical relation-

ships (Warmink et al., 2007). Other models consider the temporal evolution of these dunes (Paarlberg et al., 2006). Einstein and Barbarossa (1952), consider that the total roughness height of the main channel (k_{total}) can be divided into a contribution of grains (k_{grain}) and dunes (k_{dunes}).

According to our results, the time required for the wave to reach Sidi Bel Abbas station is estimated at $t = 6$ hours. We can divide the recession phase in two stages, the first covers 12 hours (6h to 18h) in which the simulated discharge are larger than recorded discharge. This is due in part to the water lamination by the spreading field formation (swamps), on the other hand, exclusion of affluents influence to the main thalweg. The second phase lasts from $t = 18$ hours until the end of the simulation. It is thus observed that the simulation results and recorded hydrograph are clearly coincided.

Fig. 12. Calibration Chezy coefficient of main channel and the floodplain as a function of discharge (Udo et al., 2007).

Fig. 13. Comparison between measured and simulated hydrographs for different Manning coefficients.

Summary and conclusion

Wadi Mekerra, located in a semi-arid zone, is part of the ephemeral Wadis with temporal flow. In early autumn, it generates devastating floods that cause human and material damage. This is due to hydrological, geomorphological and human factors too. These floods threaten twenty five agglomerations. Since the 2006 disaster to date more than 120 billion dinars (\$ 14 million) have been mobilized to cope with inundations. A substantial budget has been devoted to the concreting works on Wadi Mekerra channel. It is about 13 kilometers connecting Sidi Lahcen city to the north of the Sidi Bel Abbes city adding Wadi banks development. Other efficient techniques; the hillside dams of various sizes have been built to store water, contributing to cope with the high variability and reduce flood risk.

Methodologically, the flood forecasting and monitoring of changes in the flow magnitude are realized by numerical simulation of the flood wave. This simulation is performed using the simplified one-dimensional Saint-Venant model. Finite volume shock capturing schemes (Godunov and Monotonic Central) are applied. Source term is treated by using fractional step method. Numerical results analysis for the dam break test confirms that slope limiter Monotonic Central scheme is more advantageous for the accuracy and stability. However, the first order Godunov scheme present more diffusion at discontinuities. The MC scheme is thus applied to simulate the flood wave caused by Wadi Mekerra dated on October 11th, 1995. The variation of Manning coefficient ($n = 0.02, 0.03$ and 0.04) confirms that maintenance activities in the bed can amortize the impact of the flood wave.

This promising work could be extended to the application of a two-dimensional model that takes into account both the storage coefficient influence (inflow-outflow) and also the spatial distribution of Manning coefficient. For more accuracy, application of well-balanced schemes rather than fractional step method is envisaged. Wetting and drying treatment is recommended too. An adaptive refined mesh applied to a proper digital elevation model would be a certain advantage (Begnudelli et al., 2008 ; Liang and Borthwick, 2008; Berger et al., 2010). New procedures in processing satellite images allow the calibration of the roughness module in the study area Schumann et al., (2007).

Other critical parameter has an interest in development of an appropriate model. It is about of the

sediment transport generated in flood period. The silty-clay fraction increases with the width of Wadis and influences the service life of hillside dams located in the upstream (Albergel et al., 2004; De Araujo et al., 2006; Remini and Wassila, 2006; Ben Mammou and Louati, 2007).

Acknowledgements. The authors warmly thank anonymous reviewers that their comments and suggestions were highly appreciated.

List of symbols

A	– cross section of the channel [m^2],
ϕ	– flux limiter function [–],
θ	– limiter variable [–],
λ	– eigenvalues of the Jacobian matrix [–],
CFL	– Courrant Number [–],
F	– vector of flux terms [–],
Fr	– Froud number [–],
g	– gravitational acceleration [m s^{-2}],
h	– water depth [m],
J	– Jacobian matrix [–],
k	– Dune height [mm],
n	– Manning coefficient [$\text{s m}^{-1/3}$],
P	– wetted perimeter [m],
Q	– discharge [$\text{m}^3 \text{s}^{-1}$],
R	– hydraulic radius [m],
S	– vector of source terms [–],
t	– time [s],
U	– vector of conservative variables [–],
u	– flow velocity [$\text{m}^2 \text{s}^{-1}$],
U_i^n	– approximation of the conservative variable [–],
x	– longitudinal coordinate [–],
z	– elevation [m].

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Received 21 December 2011

Accepted 13 March 2012