THREE DIMENSIONAL

HYDRO-MORPHOLOGICAL MODELING OF

TIGRIS RIVER

A Doctoral Thesis

Submitted By

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Ammar Adel Ali
Luleå, March 2016
Dedicate

To the memories of
My dear mom, I wish you’re happy now
My dear grandparents, I wish you’re proud now

بسم الله الرحمن الرحيم ((وقل رب ارحمهما كما ربياني صغيرًا)) سورة الإسراء
Abstract

The River Tigris is a major river in Iraq. It divides Baghdad, the capital of Iraq, in two parts. The reach of the river within Baghdad is about 60 km long. The climate change within the region and the construction of hydraulic structures upstream of Baghdad has reduced the water discharge of the river by 44%. Despite the fact that huge volumes of sediment have been trapped in the constructed headwater reservoirs, substantial changes have occurred in the topography of the Tigris River within Baghdad City and the number of depositions is increasing. The debris of the destroyed bridges from the wars of 1991 and 2003 and their subsequent reconstruction have contributed to the development of these depositions.

As a consequence, the ability of the river to carry the peak flood waters has been reduced. This has led to a potential increase of flooding in parts of the city. To predict the maximum flood capacity for the river, the bathymetric survey that was conducted for 50 km of the Tigris River by the Ministry of Water Resources in 2008 has been used with the one-dimensional flow model “HEC-RAS”. Calibration of the model was carried out using field measurements for water levels along the last 15 km of the reach, and the water level observations at the Sarai Baghdad gauging station for the last 10 years were used to validate the model. The model showed a significant reduction in the river’s capacity compared with what the river had carried during the floods of 1971 and 1988. This result agrees with previous surveys conducted on the same reach indicating that the ability of the river to convey high water has decreased.

To overcome this problem, dredging operations started along most of the Tigris River inside Baghdad City to remove many islands and side bars, as well as cleaning water intakes. An examination for the dredging plan currently in progress and two additional proposed plans was conducted using the ‘HEC-RAS’ model for the 50 km long river reach to investigate whether the designed flooding capacity of the river can be recovered and how much it can be improved. Comparing the historical records of water level and discharge for the last three decades, some improvement of flood capacity was achieved. Cautions about the water intakes should be considered to maintain their functionalities with the expected drop in water levels due to dredging operations.

Bathymetric and land surveys were conducted for the northern Tigris River reach (18 km length) in Baghdad, producing 180 cross sections. A riverbed topography map was established from these cross sections. Sediment transport rates and bed composition were investigated by collecting three different types of sediment samples at the quartiles of 16 cross sections along this reach. The Helley-Smith sampler was used to collect 288 bedload samples, a suction pump was used to collect 212 suspended load samples from different depths. The Van Veen grab was used to collect 46 bed material samples. The velocity profiles and the water discharges were measured using ADCP at the sampling sections.

Bed sediment compositions were investigated by analysing the collected bed material samples. It was noticed that fine sand dominated the riverbed (90.74%). The average median size within the reach was 2.49 phi (0.177mm) whilst the mean size was 2.58 phi (0.16mm). In addition, the sediments were moderately sorted, fine skewed and leptokurtic. The size of the bed sediment relatively decreased compared to older investigations due to the decrease of the competence of the river. The bed elevation had increased compared to previous surveys. It was noticed that dredging
operations and obstacles (e.g. fallen bridges and islands) disturbed the flow of the river and the sediment characteristics in several sites.

Bedload rates were computed using the weights of the collected bedload samples. The spatial distribution of sampling cross sections took into consideration the variance of river topography where 7 meanders, 2 islands and several bank depositions characterize the geometry of the river reach. Twenty bedload predictors were applied to the same reach. The annual transported quantities of the bedload were estimated to be 36 and 50 thousand tons in 2009 and 2013 respectively.

The total load discharge rate in the northern reach of the Tigris River was computed using the sediment concentrations of the collected suspended load samples after adding the bedload rate at each of the sampling cross sections. The results indicated that the suspended load is the dominant mode in the total load with a minimum percentage of 93.5%. The total load ranged from 29.1 to 190.3 kg/s. A total load rating curve of the power function was established. The associated errors from using the proposed rating curve are within reassuring levels and less than the errors produced from most of the other twenty-two total load formulas, which were applied to the same reach. The scattering of the results from the other formulas can be attributed to the spatial variance in the topography of the riverbed. According to the final results obtained, it is recommended to use the proposed procedure for establishing a spatial total load rating curve to estimate sediment rates for morphologically complicated rivers. The annual transported quantities of the total load were estimated at 2.47 and 4.23 million tons for 2009 and 2013 respectively.

The three-dimensional morphodynamic model (Simulation of Sediment movements In water Intakes with Multiblock option - SSIIM) was used to simulate the velocity field and the water surface profile along the northern reach of the Tigris River using the findings of the current bathymetric survey of the river. The model was calibrated for the water levels, the velocity profiles and the sediment concentration profiles using different combinations of parameters and algorithms, those available in the model. The set of parameters that gave a minimum root mean square error (RMSE) was used for the validation process using another set of field measurements. The calibration and the validation results showed good agreement with field measurements, and the model was used to predict the future changes in river hydro-morphology for a period of 14 months. The results of the future predictions showed increases in depositions on the shallow part of the cross section having lower velocity and, on the other hand, the river deepens the incised route to fit its current hydrologic condition leaving the former wide section as a floodplain for the newer river. The net deposition/erosion rate was 67.44 kg/s in average and the total deposition quantity was 2.12 million tons annually. The locations of depositions are compatible with those of the river in the real world. An expansion in the size of current islands was predicted. An indication of the potential threats of the river banks’ collapse and the bridge piers’ instability was given by high erosion along the thalweg line.

**Keywords**: Flood capacity, Dredging, HEC-RAS, Bathymetric survey, Bed sediment, Bedload, Total load, Helley-Smith sampler, Sediment transport, ADCP, Prediction formulas, 3-D morphodynamic model, Bed changes, SSIIM, underfit river, regulated river, Tigris River, Baghdad.
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Paper II
   Evaluation of Dredging Operations for Tigris River within Baghdad, Iraq

Paper III
   Flow of River Tigris and its Effect on the Bed Sediment within Baghdad, Iraq

Paper IV
   Spatial Measurement of Bedload Transport in Tigris River

Paper V
   Total Sediment Load in Tigris River at Baghdad City

Paper VI
   Three-Dimensional Morphodynamic Modelling of Tigris River in Baghdad
Introduction

This chapter presents general information about sediment transport in rivers. It includes a brief background on erosion and deposition and their interaction with the changes in the hydrologic system of the river, simulation models for erosion and deposition processes in rivers, the changes in the hydrology of the Tigris River and the consequences, the objectives of the study and the novelty.

1.1. General Background

Natural rivers carry water, sediment and solute from watersheds to the estuaries. During this long trip, rivers have the ability to erode the land or deposit some of the load which then composes the bed or banks. Change in riverbed morphology is a continuous process with time and of different scales due to the variation of different variables such as water discharges, velocities, sediment supply rates, and composition of bed and banks. The changes can be in the size and shape of stream channel, bed material composition, slope and pattern. Long term changes in river morphology that date back millions of years ago attract geomorphologists and geologists whilst hydraulic engineers are interested in short term changes that date back 20 years (Garde, 2006).

Aggradation and degradation in a river’s course resulting from erosion and sedimentation mean that the river is not in equilibrium and the source of disturbance might be either natural or man-made activity. Construction of large dams trap the sediment from the watershed causing aggradation upstream of the dam whilst the released water from the dam is almost sediment free causing degradation of the river downstream of the dam. Barrages on the rivers convert part of the river flow away causing aggradation downstream. Therefore, permanent regulation schemes on river systems and/or climatic changes within the catchment will disturb the water flow and sediment supply, and this will lead to changes in the river course dimensions and characteristics (Garde, 2006). Erosion and sedimentation have major effects on different issues related to the use of the river and human life such as stability of hydraulic structures, scour at bridge piers, bank erosion, operation of water intakes, navigability, flooding capacity, etc.

Estimating erosion and sedimentation rates and their locations along river’s reach can be as important as estimating flow and velocity for preparing future plans. These kinds of estimations require a good knowledge of the effective variables those control the processes, as well as also understanding the relationships which control these variables. Discharge, flow velocity, bed material, sediment supply and channel slope are the major effective variables that control the erosion and sedimentation processes.

In the past, estimation of velocity was achieved by using physical models jointly with field measurements. However, after the great development in computational methods and computer capabilities, simulation models became popularly used for this purpose. Simulation models usually using some of the well-known equations such as Navier-Stokes equations established based on conservation concepts such as conservation of mass and momentum to govern the relationships between the control variables in the treated phenomena. Moreover, simulation models started coupling many models together (such as hydrodynamic model, sediment transport model and morphology model) to reach for the integrity in the subject of erosion and sedimentation.
Simulation models are powerful and relatively inexpensive tools for predicting future morphological changes, evaluating alternatives of river control works, and environmental management of the river. The reliability and the accuracy of a simulation model’s performance in reproducing field conditions of real-life problems in the model, are questionable unless they are subjected to calibration and validation processes to adjust to the parameters and the algorithms of the model and re-examining the results for different site conditions.

Field investigations and measurements are necessary to provide simulation models with different sets of values for the important variables in the model, such as flow velocity, water level and sediment concentrations. These variables are to be used for comparisons with model results during calibration and validation steps.

SSIIM (Simulation of Sediment movements In water Intakes with Multiblock option) model is a three dimensional morphodynamic model that is used to simulate velocity field, water surface profile, sediment transport, morphological changes as well as pollution dispersion in rivers and reservoirs with high flexibility for dealing with complex geometries. It couples several models (convection-diffusion model, transport-dispersion model and sediment continuity model) with geometry editor and input file for pre-processing step and graphical display package for post-processing step (Olsen, 2014).

1.2. The Tigris River

The Tigris River is one of the most important rivers in the Middle East. It rises from the Taurus Mountain range in the south-eastern part of Turkey and flows towards the southeast for 1580 km, passing through Turkish-Syrian borders and entering Iraq. In Iraq, it flows towards the south until it combines with the Euphrates River at Qurnah, forming the Shatt Al-Arab River. This river discharges its water into the Arabian Gulf (Figure 1.1.A).

Within Baghdad City, the capital of Iraq, the Tigris River bisects Baghdad into two parts for a distance of about 50 km within the urban zone starting from Al-Muthana Bridge to the north and ending at the confluence with the Diyala River to the south. This river reach has single thread, compound meanders, and alluvial plain characteristics. The river banks are protected against erosion for 66% of the length in the urban zone by aligned stones and cement mortar between levels 29 and 37 m.a.s.l. at the start of the reach and drop gradually to the south. Recently, the dominant water levels in the reach are below the protection level.

The flow of the river is fully controlled in Baghdad by a system of dams and regulators constructed on the main river and the tributaries upstream of Baghdad (Figure 1.2). These regulating schemes have decreased the average monthly discharge of the river according to the records of the Sarai Baghdad gauging station from 1207 m$^3$/s for the period 1931-1959 to 927 m$^3$/s for the period 1960-1999 due to the construction of dams. Furthermore, for the period 2000-2013, the discharge dropped to 522 m$^3$/s. The Tigris River hydrograph at Sarai Baghdad (Figure 1.3) shows that the maximum flow takes place during April and May. Most of the sediment is transported in that period (Al-Ansari et al., 1979). Furthermore, Figure (1.3) shows that the hydrograph has become flatter and flatter since 1990. This flattening is mainly due to the effect of the regulating schemes in Iraq and Turkey as well as climate change where a drought period is affecting the region (Al-Ansari, 2013 and 2016; Al-Ansari et al., 2014 and 2015a).
Introduction

Figure 1.1: (A) Map of Iraq (Encyclopaedia Britannica, 2010) (B) The Tigris River inside Baghdad (the islands and sandbars were bordered by red).

Figure 1.2: Schematic Diagram of the Tigris River Hydrological Scheme (MWR, 2005).
Chapter One

Figure 1.3: Decadal hydrographs of the Tigris River at Sarai Baghdad for the period 1930-2013 (data source: Al-Shahrabaly, 2008).

Sediment transport rates are highly affected in the course of the Tigris River upstream of Baghdad due to the trapping of sediment within the reservoirs of dams upstream of Baghdad. However, during the last two decades many new islands, side depositions and point bars appeared in the Tigris River’s reach within Baghdad (Figure 1.1.B). Eighteen obstacles between islands and sandbars were recognized inside Baghdad in 2008 (Ali, 2013; Ali et al., 2012 and 2014). In 1990 and after 2003 three bridges on the river in the centre of Baghdad were destroyed. All of them were reconstructed again at different periods, but a significant part of the fallen debris could not be removed from the river bed. Consequently, the debris were disturbing the river flow.

Sedimentation in the Tigris River course had its impact on the hydraulic performance of the river, such as reducing its flood capacity, impeding navigation and reducing the efficiency of water intakes of water treatment plants, as well as the environmental and aesthetic impacts (Ali et al., 2012 and 2014). On the other hand, banks protection stability has been threatened at some locations due to eroding deep incisions in the river bed on the outer banks of meanders.

An effective rapid action was required to consider preventing this deterioration in the performance of the river. The Ministry of Water Resources (MoWR) in Iraq started dredging operations at specific sites along the river inside Baghdad to overcome the sedimentation problems. Despite the fact that dredging is undertaken, several questions remained unanswered.

These are: Is the dredging efficient from a hydraulic performance viewpoint? For how long will dredging operations be undertaken? Are more depositional areas expected to appear? These questions are still waiting to be answered.

1.3. The objectives of the study

The highlighted questions mentioned in the previous paragraph are to be answered in this research. To answer these questions, the following points were considered:

1. Determining the pre-dredging flood capacity of the Tigris River.
2. Evaluating the post-dredging improvement in the hydraulic performance of the Tigris River.
3. Investigating the nature of sediment transport in the Tigris River.
4. Predicting future changes in bed topography and morphology of the Tigris River.

1.4. The Novelty of the Study

The Novelty of this research can be summarised in the following points:
1. Measuring bedload and total load transport rate and their spatial distribution in the Tigris River in Baghdad which has not been measured before.
2. The characteristics of the flow and sediment and their spatial distribution in the study reach.
3. Applying 3D morphodynamic model on the Tigris River to predict the future changes in river bed topography and morphology.
Chapter Two

Literature Review

Studying the changes in river morphology requires dealing with a wide range of topics those are related to the river hydrology, sediment transport and simulation models. Some of these subjects were extensively studied and applied where many developments were introduced such as the prediction formulas for sediment transport. However, they are still theoretically limited to the conditions where they were applied. Some other topics are still developing, however, they show promise in performance and applications such as morphodynamic modelling. The main topics considered in the field of river morphology are:

1. Training measures applied to rivers to improve their hydraulic performance.
3. Morphodynamic modelling of changes in flow field and river morphology.

Some of the conducted studies which are related to these topics were reviewed as well as other related studies conducted on the Tigris River in Baghdad.

2.1. Flood Capacity and River Training

The need to apply training measures to rivers is to improve the hydraulic performance of the rivers, where the most critical need is to prevent the catastrophic damage of floods to human life and the infrastructures. River training measures include canalization, artificial cut-offs, bank protection, dredging, etc. The side effects of river training can be observed from the changes in river bed elevations and bed sediment. Several river training studies have been conducted on many rivers worldwide.

Marchi et al (1995) evaluated river training work in the lower Po River in Italy. Their training activities had successfully reduced the overflow frequency as a consequence of protection and regulation work on the tributaries and also on the main river. The storage capacity of the river bed in flood has been reduced due to a reduction in flood expansion areas in the upper and middle parts of the drainage basin.

Lammersen et al (2002) investigated the impact of river training and retention measures on the flood peaks on the River Rhine in Germany. They found that the constructed weirs along the upper reaches and other retention measures had successfully influenced the flood conditions along the river. The SYNHP hydrological model was used to describe the flood routing processes in the river by using single linear stores and this was used to evaluate the effects of retention measures in the upper reaches. The 1-D river flow model SOBEK, was used to perform flow calculations for the middle and lower reaches, based on the Saint-Venant equations. Both models indicated that the river training activities led to an increase in peak flow.

Korpak (2007) demonstrated the influence of river training on channel erosion in Polish mountain rivers. Using observation data for 53 years, she showed that debris dams and groynes built before 1980 had caused great changes in channel patterns and increased the channel gradient and the rate of river incision. She considered that although the measures to decrease river down cutting in alluvial deposits worked well, it had not been eliminated. Korpak noted that river training schemes distort the equilibrium of the channel systems and that most of such projects were of limited success in the long term because they rarely considered the entire reaches of the rivers.

2.2. Sediment Load: Measurement and Prediction

Sediment transport in rivers is an important and significant item to be measured at a hydrometric station. Measurements have to be conducted for suspended and bedload discharge in rivers with
natural regimes as well as with human management activities. The effect of the sediment problem on the development of water resources is the main motivation to measure sediment discharge at certain locations along the river’s reach. The data obtained from measurements are important for understanding the fluvial process and the erosion or sedimentation conditions in the river reach (WMO, 2003). A review of the literature dealing with sediment load measurements has been made.

Helley and Smith (1971) designed a pressure difference sampler to measure coarse bedload in natural streams. Later on, different versions of the sampler were widely used by many researchers under different conditions than those it was designed for.

Andrews (1981) used the H-S sampler to measure the bedload of the Muddy Creek stream sand-bed in Wyoming, as part of measuring the total load of bed material discharge. His measurements were in agreement with Engelund-Hansen, Yang, Shen-Hung and Ackers-White prediction formulas.

Bridge and Jarvis (1982) conducted comprehensive measurements for flow and bedload discharge at a bend in the South Esk River (Scotland) to study the interaction of flow and sedimentary processes in natural rivers. They also measured secondary circulation, water surface configuration and analysed bed material grain size distributions, bed shear stress distribution, and bed roughness coefficients. They found that flow resistance reaches a maximum in the range of two-thirds of the bankfull stages, and bed topography is in equilibrium with bedload transport.

Dietrich and Smith (1984) studied bedload transport at a meander in Muddy Creek. They found that the zone of maximum sediment flux has been shifted across the meander from the inside bank in the upstream part of the meander towards the pool. The H-S sampler was used for sampling along and across the meander.

Bridge and Best (1988) studied the interaction between turbulence, sediment transport and the bedform dynamics during transition from dunes to upper stage plane beds in a laboratory flume. Measurements for bedload discharge were conducted using the H-S sampler and for suspended load using depth-integrated sampler. They found that both loads increase progressivly over the transition.

Van Rijn and Gaweesh (1992) designed and tested a total sediment load sampler that has been able to measure the bedload and the suspended load simultaneously. The sampler has been tested for measuring sediment transport in different cross sections of sand bed of Nile River in Egypt.

Atkinson (1994) explained the theory for predicting the trapping efficiency of vortex tubes. The comparison with field measurements showed that the ratio of observed trapping efficiency to the predicted has perfect agreement in mean.

Bunte and Abt (2009) compared sampling results from bedload traps with the H-S sampler for a wide range of transport rates in different sites of gravel bed mountain streams. The differences in measured transport rates showed that the H-S sampler measured higher transport rates than bedload traps at low flow and similar or higher transport rates at high flows.

2.3. Sediment Rating Curve

The relationship between water flow and sediment discharge is called a sediment rating curve. It sums up many of the variables affecting the sediment discharge in a simplified relationship with water discharge. The accuracy of such a rating curve is determined by the availability of a sufficient number of measurements, time frame for measurements, curve fitting methods, and curve formula. There is a consensus from many authors on using power function as a regression equation for the sediment rating curve (WMO, 2003; Glysson, 1987).

Walling (1977a) derived several power function rating curves for the Creedy River in England, depending on season and discharge stage, he also analysed the error produced from using them.
Fenn et al. (1985) conducted an evaluation for using standard ordinary least square regression technique for deriving rating coefficients from different grouped observations according to seasons and climatic conditions.

Asselman (2000) used different fitting procedures to derive power function sediment rating curves for many locations along the Rhine River and its main tributaries; she related the differences in the rating curves coefficients to watershed characteristics.

Gaeuman et al. (2015) established fractional bedload rating curve for the Hayden Creek in Denver-USA using a maximum likelihood regression technique to investigate the physical interpretations for the variations in rating curve coefficients.

2.4. Morphodynamic Models

The concern about the predicted changes in river bed topography and morphology has increased the demand for using the mathematical models to simulate such sophisticated phenomena. The mathematical models are inexpensive compared with physical models, flexible to simulate different alternates, easier to use with competent computers and graphical interfacing, as well as they are capable of performing integral solution for multitask issues (hydraulics, morphology, environment, etc.). Many related studies and applications have been conducted over the last few decades and they offer at various levels of sophistication. Some of these studies were reviewed.

Khalaf (1999) built a 2D morphodynamic model to predict the changes in flow field and bed topography. The model has been applied to the Tigris River within Baghdad City. The results showed an increase in depth at a scour region, whilst no significant change was noticed in other regions.

Dorfmann and Knoblauch (2009) compared 2D (MIKE21C) and 3D (SSIIM) morphodynamic models applied to a large reservoir with ADCP velocity measurements. They found that the 3D model is highly sensitive to the bed roughness and the numerical discretization scheme.

Rüther et al. (2010) applied both 2D and 3D morphodynamic models to the Po River in Italy to investigate the model’s results for better understanding the effect of different scales on fluvial processes in rivers dominated by alternative bars. They found that both models overestimate the velocity and the sediment concentrations at the first part of the upstream bend. However, the results of the 3D model were slightly superior at the deviation of the velocity vectors on the surface and on the bed, the computation of the sediment concentration was also better, too.

Logan et al. (2010) applied a 2D unsteady and a 3D unsteady morphodynamic models (Delft2D and Delft3D) to the Colorado River’s reach, containing lateral separation zone to understand the relative roles of various exchange mechanisms between the main channel and eddies. They found that both models over predict the amount of sediment deposited in the lateral separation zone and in other locations along the channel margin.

Hobi (2010) applied a 3D morphodynamic model (SSIIM) to the Euphrates River, downstream of Kuffa Barrage to determine whether the morphodynamic model is able to predict sediment transport in the study reach for different scenarios of barrage operation. He proposed an operation scenario to reduce the trapped sediment at the barrage.

Elsaeed (2011) predicted local scour around bridge piers using the SSIIM model for low Froude flow. The results have shown good agreement with the results from a large scale flume experiments.

Sirdari et al. (2013) applied a 3D morphodynamic model (SSIIM) on river confluence in Malaysia to study the bed morphology and bedload transport within the junction.

Moghaddam and Rennie (2015) utilized spatially distributed velocity measurements using ADCP for improving calibration and validation of the 3D morphodynamic model (Delft3D) applied to clay-bed meandering rivers. They found that eddy viscosity is more useful for calibration purposes than bed roughness.
Zhang et al. (2015) applied a 3D morphodynamic model (SSIIM) model on non-uniform bed composition reservoir on the Upper Rhine River in Germany to investigate the sensitivity of the results to a set of parameters in the model as well as to investigate the accuracy of bed changes simulation. They observed influences in the sediment erosion and deposition results due to minor changes in the parameters.

Baranyas et al. (2015) applied a 3D morphodynamic model (SSIIM) model on medium-sized river confluence in Hungary to simulate the unsteady vortex shedding and the secondary currents by implementing nested grid approach.

2.5. The Tigris River

Several studies had been conducted on the Tigris River within Baghdad City. Some of them dealt with estimation of sediment transport exclusively whilst others extended to evaluation the changes in bed topography and morphology or evaluation of the flood capacity.

Geohydraulique (1977) conducted a training study of the Tigris River for the benefit of The Iraqi Ministry of Irrigation to improve the river flow conditions and specify the flood capacity of the river in Baghdad. In that study, land and bathymetric surveys were conducted for a reach of 59 km of the Tigris River within Baghdad City. The study also included field measurements for water levels and velocity distribution. As well as collecting suspended load samples and bed material samples on six occasions. The analyses of sediment samples showed that the concentration of the suspended sediment were relatively ‘weak’ and did not reach to 3 g/l throughout the whole period, whilst it was less than 0.2 g/l during low flow. The analyses of particle size distribution for bed material showed that the bed formed predominantly from sand with high silt percentage close to the banks. The flume experiments for bed sediment showed that they are always in motion in the form of ripples even in low discharge periods. A sediment discharge rating curve was established during that study for predicting sediment load because none of six bed sediment load predictors used could be validated for the river. The annual total sediment discharge was estimated as 11 million tons for the mean annual water discharge (1035 m$^3$/s). The results of 1-D gradually varied flow model specified the inundation levels for the discharges 4000 and 5500 m$^3$/s and their locations. Furthermore, the study stated that the river was unstable with erosion on the outer banks of meanders and sand bars of various dimensions will continue to appear.

Al-Ansari et al. (1979) applied nine prediction formulas of sediment discharge to the Tigris River at the Sarai Baghdad gauging station. They used the geometry and hydraulic conditions obtained from Geohydraulique (1977) and direct suspended sediment measurements for the period 1969-1975. They proposed Yang formula (1973) as the closest to the actual measurements where it was 4.6 million ton/year in average.

Al-Ansari and Toma (1984) studied the morphology of the Tigris River between the Al-A’ameh and Al-Sarafiya Bridges (4.7 km reach length). They conducted a bathymetric survey for the river bed and collected bed material samples along the reach and from the islands. They found that the changes in the geometry took the sinuosity form. The bed material was a mixture of coarse, medium and fine sand with a trace of clay whilst the clay presence increased for the islands and the banks. The analyses of bed material showed that the bed sediment is very well sorted and has fine skewness with kurtosis ranging from platy to meso. The average annual sediment discharge at Sarai Baghdad was an estimated 23.6 million ton/year according to field measurements and historical observations. They found the Straub-DuBoys formula was the closest to the field measurements.

Al-Ansari et al. (1986) observed the suspended and solute loads of the Tigris River at Sarai Baghdad continuously for the period 1983-1985. They concluded that the average daily discharges of suspended and solute loads were 30000 ton and 40000 ton respectively. The studied period was considered as a dry period especially in 1984 where the average daily discharge was 571.3 m$^3$/s whilst for the whole period it was 728.8 m$^3$/s.
Khalaf (1988) studied sediment transport in the Tigris River in Baghdad. He measured suspended load at two gauging stations and collected bed material samples from these stations. He combined his measurements with the available historical records for suspended load starting from the year 1953 to find out the best prediction formula between fourteen sediment estimation formulas. He concluded that Laursen’s and modified Yang’s formulas with slight adjustments were giving the best predictions for the sediment load. He also recommended using Inglis-Lacey for extrapolating sediment load beyond the measured range of sediment due to the normality of the distribution that it follows and the highest correlation coefficient that it has. He attempted to derive a new prediction formula based on dimensional analysis concept. His formula is mainly related to the effective shear stress.

Al-Ansari and Al-Sinawi (1988) studied the effect of the agricultural activity near the Tigris River banks upstream from Baghdad on the suspended sediment concentrations in the river and their periodicity. They collected suspended load samples continuously at 2hr intervals at two stations (at the north of Baghdad and at the centre) for a period of 7 days. They concluded that the observed suspended load at the first station was deposited along the reach between the two stations as proved from the decrease of the concentrations and diminishing the cycles of the short periodicity at the second station.

The University of Technology (1992) in Iraq conducted a training study for the Tigris River in Baghdad similar to the study of Geohydraulle (1977). They considered two bathymetric surveys conducted in 1988 and 1991. A 2-D morphological model was applied to determine the velocity distribution and water surface profile that can be used to specify the locations of river banks that require protection, and specified the types of protection to be considered, as well as specifying depositional zones and the formation of islands. According to the model results, they concluded that the Tigris River was still unstable with the fixity coefficient increasing between years 1976 and 1991 from 2.5 to 3.1 respectively. The flood of 1988 brought considerable quantities of fine sediments, so that, depositions contained fine sand, silt and clay which were found in some locations. They recommended a list of locations that require protection from erosion either by stones or sheet piles. Also, they recommended the using of groins of 50-80m length for depositional locations.
Field and Lab Work

Simulating water flow and sediment transport in natural rivers required collecting sets of data from the field and treating them in the lab. Field data are related to river geometry, river hydrodynamic and sediment data. Brief explanations for these data are as follows:

1. River geometry included conducting bathymetry and land survey for the riverbed topography and floodplain respectively.
2. River hydrodynamic data included measuring the velocity distribution and water levels.
3. Sediment data included sampling of bed sediment, bedload and suspended load concentration.

Lab work can be described as all the work other than field work. In this case it included laboratory analyses of the collected samples water or sediment, treating surveying data, and model preparation and operation. In addition, treatment of the data was performed as:

1. Converting bathymetry and land survey to geometry map or DEM map.
2. Determining grain size distribution for the bed sediment as well as specific weights.
3. Preparing distribution map for bed sediment fractions.
4. Analysing collected sediment and water samples by determining weights and concentrations.
5. Calculating sediment transport rates from treated sediment samples.

3.1. Field Work

3.1.1. Bathymetry and Land Survey

Field work included conducting bathymetry and land survey for the Tigris River’s reach of 50 km length inside Baghdad. The survey was conducted in three stages as follows:

1. Installing 25 bench marks on the river banks along the study reach using DGPS based on UTM-WGS84 coordinates system as well as linking them with the Iraqi national triangulation network (known as “Polservice” according to the Polish firm that established these points).
2. Water surface elevations were levelled at river banks using total station.
3. Surveying the upper banks of the river from the crest to the base of the protected part at an average spacing of 200m between the sections, whilst for the lower banks, the unprotected part from the base of protection to the water surface, the average spacing between cross sections was 100m along the northern part of the river’s reach. On the southern reach of the river, it was difficult to conduct the land survey along the whole reach due to either security prevents or site difficulties such as private properties. The as-built drawings for the protected banks and previous survey were used with the assistance of the satellite images to determine river banks’ topography. Leica and TOPCON total stations were used for the land survey.
4. Bathymetric survey was conducted using echo sounder sonar with GPS external antenna at an average spacing of 100m between the cross sections. Landmarks that can be recognized from satellite images were used as guidance for the ends of the cross sections. Measurements of water levels were associated with the bathymetry at the beginning and the end of the surveyed reach segments every working day to convert the sonar water depths to bed elevations. Intensive readings for water depth were taken at the boundaries of the islands (Fig 3.1).
Chapter Three

3.1.2. Hydrodynamic Measurements

Hydrodynamic measurements are essential data to be used with simulation models. They are used in the calculations of sediment rate and in the operation and calibration of the hydrodynamic model. The following measurements were conducted at 27 cross sections of different morphology along the reach of the Tigris River:

1. Water discharge and velocity distribution were measured using SonTek RiverSurveyor M9 ADCP with repetitions for more accuracy (Fig 3.2).
2. Additional measurements for flow velocities were measured at certain depths of shallow water using FP111 Global Water Flow Probe whilst deeper flow was limited to ADCP readings.
3. Total dissolved solids were measured at each cross section of ADCP. They are required for using of ADCP.

Figure 3.1: Bathymetric surveys’ path along the northern reach of the Tigris River in Baghdad.
3.1.3. Sediment Measurements

Investigating sediment transport in the river is essentially conducted for studying the changes in the river bed. This investigation included the components of sediment load as well as the possible local sources of sediment. Measuring the total load of sediment requires measuring the two components of sediment, bedload and suspended load. The bedload can be measured using manually operated portable samplers such as Helley-Smith sampler (Diplas et al., 2008). Different techniques can be used for measuring the suspended sediment along the water column, depth-integrating sampling or point-integrating sampling (Edwards and Glysson, 1998). Bed sediment is the main source of transported sediment in supply-limited rivers. Sampling of bed material can be done by scooping.

Three kinds of sediment samples were collected in the field as follows:

1. Bed sediment samples: 57 samples were collected at the 23 cross sections along the study reach using van Veen grab of size 3.14 litres.

2. Bedload samples: 462 bedload samples were collected (Fig 3.3) using the Helley-Smith sampler, entrance of opening 3" x 3" (76 mm x 76 mm) and 3.5 exit/entrance expansion ratio, at the quartiles of 27 cross sections (Fig 3.4). Five repetitions of sampling time 60 sec were conducted at the sampling points to overcome temporal variation in bedload with the presence of ripples and sand dunes. Separation time between repetitions was 3 minutes. Zero-time samples were collected at sampling points along the northern reach of the Tigris River to examine the initial effect and the scooping effect of the sampler on the bed. For zero-time sampling, the sampler was lowered until the nozzle was in touch with the bed then the sampler was raised immediately (Van Rijn and Gaweesh, 1992).

3. Suspended load samples: 335 suspended load samples of 500ml volume were collected based on point-integrating procedure on the quartiles of 27 cross sections using suction pumps. Suction velocity was higher than the settling velocity of all particle sizes of bed material. The intake nozzle of pumping system was installed on the top of the HS sampler (Van Rijn and Gaweesh, 1992) but
with single intake nozzle. The number of samples at each water column was selected depending on its height; the longer the height was, the higher the number of samples, as listed in Table (3.1).

The selection of sampling cross sections was decided according to their morphological conditions such as existence of meander, island, sandbar and side bar within the river’s reach.

Figure 3.3: A bedload sample trapped in the PVC net bag of the Helley-Smith sampler.

Figure 3.4: Locations of the sites of sediment sampling along the Tigris River reach in Baghdad.
Table 3.1: Number of suspended load samples on water column depending on the water depth (Al-Ansari, 2005).

<table>
<thead>
<tr>
<th>Depth of water (h)</th>
<th>No of samples</th>
<th>Depths of samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.6m</td>
<td>1</td>
<td>0.6h</td>
</tr>
<tr>
<td>0.6m ≤ and &lt; 3m</td>
<td>2</td>
<td>0.2h, 0.8h</td>
</tr>
<tr>
<td>3m ≤ and &lt; 6m</td>
<td>3</td>
<td>0.2h, 0.6h, 0.8h</td>
</tr>
<tr>
<td>6m ≤</td>
<td>6</td>
<td>0h, 0.2h, 0.4h, 0.6h, 0.8h, 1h</td>
</tr>
</tbody>
</table>

3.2. Lab Work

Part of the lab work was exclusive to the measurements of the northern reach of the Tigris River a length of 18 km that extended from Al-Muthana Bridge to the north of the city until Sarai Baghdad gauging station at the centre of Baghdad (Fig 3.5). So, the results of the lab work were analysed and presented for the northern reach of the river whilst the southern reach will be included in future works.

![Figure 3.5: Cross sections of sediment sampling and ADCP measurements along the northern reach of the Tigris River.](image)

3.2.1. Bathymetry and Land Survey

Water depths obtained from the bathymetric survey were converted to bed elevations using the measurements of water levels during the survey. The coordinates of surveying cross sections were adjusted according to the guidance of the landmarks used during the survey. Bed elevations were combined with river bank elevations extracted from the land survey and the new elevations data set was used to build a DEM file for river geometry (Fig 3.6). River bed elevations are ranged between 13.5 and 28.76 m.a.s.l. The deepest locations are on the outer banks of meanders in the part directly downstream from the peaks of meanders. The bed elevation map showed deposition near water surface at level between 27 and 28.76 m.a.s.l.
3.2.2. Hydrodynamic Data
The range of measured water discharges at 16 cross sections along the northern reach of the Tigris River (Fig 3.5) was between 445.1 and 651.8 m$^3$/s. The average cross section velocities were in the range between 0.51 and 0.76 m/s as shown in Table (3.2). Velocity profiles at sampling points were determined from the readings of ADCP according to the location of the sampling point on the cross section. ACDP readings were sometimes coarse and the variations of velocity along water column were sharp and diverged from the logarithmic profile, so repetitions of ADCP readings were used to smooth the velocity profile at the sampling points.

3.2.3. Sediment Samples
Particles density was determined for bed sediment samples using water pycnometer. The range of densities was between 2.68 and 2.78 kg/litre. The mean particles density was considered as 2.73 kg/litre.

Particle size distributions of the bed sediment samples were analysed using sieving and a hydrometer. Fine sand was the bed dominant size and 9 particle sizes were recognized in the whole reach. They ranged from medium sand to clay according to Wentworth (1922) grain size classification (Fig 3.7). Mean particle diameters ($d_{50}$) were determined as well as $d_{90}$ (the particle size when 90% of the particles are finer) (see Table 3.2).
Figure 3.7: Cumulative curves of the bed sediment of the Tigris River for the sampling points CS2-L and CS7-C respectively.

Table 3.2: Hydraulic data from ADCP measurements and the mean particles size from bed sediments.

<table>
<thead>
<tr>
<th>C.S.</th>
<th>Cross-sectional Area ($m^2$)</th>
<th>Top Width (m)</th>
<th>Hydraulic Radius (m)</th>
<th>Water Discharge ($m^3/s$)</th>
<th>Water Velocity (m/s)</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{90}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>664.9</td>
<td>180.03</td>
<td>2.98</td>
<td>457.381</td>
<td>0.688</td>
<td>0.194</td>
<td>0.273</td>
</tr>
<tr>
<td>CS2</td>
<td>653.7</td>
<td>260.77</td>
<td>2.471</td>
<td>459.022</td>
<td>0.702</td>
<td>0.166</td>
<td>0.235</td>
</tr>
<tr>
<td>CS3</td>
<td>795.2</td>
<td>261.6</td>
<td>3.008</td>
<td>464.409</td>
<td>0.584</td>
<td>0.175</td>
<td>0.25</td>
</tr>
<tr>
<td>CS4</td>
<td>691.5</td>
<td>250.06</td>
<td>2.743</td>
<td>445.095</td>
<td>0.644</td>
<td>0.199</td>
<td>0.273</td>
</tr>
<tr>
<td>CS4-2</td>
<td>745</td>
<td>241.09</td>
<td>3.013</td>
<td>452.325</td>
<td>0.607</td>
<td>0.197</td>
<td>0.276</td>
</tr>
<tr>
<td>CS5</td>
<td>643.3</td>
<td>151.67</td>
<td>4.072</td>
<td>489.233</td>
<td>0.76</td>
<td>0.208</td>
<td>0.278</td>
</tr>
<tr>
<td>CS6-1</td>
<td>865.2</td>
<td>353.85</td>
<td>2.398</td>
<td>549.877</td>
<td>0.636</td>
<td>0.199</td>
<td>0.273</td>
</tr>
<tr>
<td>CS6-2</td>
<td>421.284</td>
<td>185.2</td>
<td>2.271</td>
<td>286.409</td>
<td>0.68</td>
<td>0.21</td>
<td>0.275</td>
</tr>
<tr>
<td>CS6-3</td>
<td>369.83</td>
<td>83.37</td>
<td>4.113</td>
<td>251.023</td>
<td>0.679</td>
<td>0.145</td>
<td>0.255</td>
</tr>
<tr>
<td>CS6-4</td>
<td>760.4</td>
<td>237.8</td>
<td>3.141</td>
<td>561.778</td>
<td>0.739</td>
<td>0.19</td>
<td>0.27</td>
</tr>
<tr>
<td>CS7</td>
<td>932.7</td>
<td>320.08</td>
<td>2.881</td>
<td>651.709</td>
<td>0.699</td>
<td>0.2</td>
<td>0.277</td>
</tr>
<tr>
<td>CS8</td>
<td>979</td>
<td>236.86</td>
<td>4.053</td>
<td>643.319</td>
<td>0.657</td>
<td>0.2</td>
<td>0.275</td>
</tr>
<tr>
<td>CS9</td>
<td>772.9</td>
<td>255.5</td>
<td>2.911</td>
<td>530.443</td>
<td>0.686</td>
<td>0.12</td>
<td>0.218</td>
</tr>
<tr>
<td>CS10</td>
<td>970.2</td>
<td>249.34</td>
<td>3.911</td>
<td>573.967</td>
<td>0.592</td>
<td>0.176</td>
<td>0.25</td>
</tr>
<tr>
<td>CS11</td>
<td>1128.40</td>
<td>213.99</td>
<td>5.279</td>
<td>578.375</td>
<td>0.513</td>
<td>0.143</td>
<td>0.243</td>
</tr>
<tr>
<td>CS13</td>
<td>720.6</td>
<td>114.9</td>
<td>5.889</td>
<td>529.965</td>
<td>0.735</td>
<td>0.197</td>
<td>0.276</td>
</tr>
<tr>
<td>CS14</td>
<td>711.4</td>
<td>137.84</td>
<td>4.976</td>
<td>522.226</td>
<td>0.734</td>
<td>0.135</td>
<td>0.213</td>
</tr>
</tbody>
</table>
From 462 bedload samples, 288 samples were collected at 16 cross sections along the northern reach of the Tigris River. 238 samples were of 60 second sampling time, 2 samples of 120 second and one sample of 300 second, whilst the remaining 47 samples were of zero-time sampling. The masses of the repeated samples at each sampling point were averaged and the masses of zero-time samples were subtracted from the averages according to its sampling point. The net average masses were reduced by 50% due to the trapping efficiency of the Helley-Smith sampler, where the calibration studies about the sampler mentioned that its trapping efficiency is about 175% for particles range between 0.25 and 0.5 mm (van Rijn, 2007) and it is about 200% for particles range between 0.125 and 0.25 mm (Emmett and Hubbell, 1977). The bedload masses ($w_{b}$) were calculated from the reduced average masses ($w_{av}$) after extracting the zero-time masses ($w_{0}$). The bedload masses were converted to sediment discharge ($q_{b}$) per unit width and the results were listed in Table (3.3).

<table>
<thead>
<tr>
<th>Sampling point</th>
<th>$w_{av}$ (g/m)</th>
<th>$w_{0}$ (g)</th>
<th>$w_{b}$ (g/m)</th>
<th>$q_{b}$ (g/s.m)</th>
<th>Sampling point</th>
<th>$w_{av}$ (g/m)</th>
<th>$w_{0}$ (g)</th>
<th>$w_{b}$ (g/m)</th>
<th>$q_{b}$ (g/s.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1-30</td>
<td>97.45</td>
<td>1.15</td>
<td>48.15</td>
<td>10.531</td>
<td>CS6-3-R</td>
<td>45.27</td>
<td>0.95</td>
<td>22.16</td>
<td>4.847</td>
</tr>
<tr>
<td>CS1-60</td>
<td>27.57</td>
<td>0.84</td>
<td>13.36</td>
<td>2.923</td>
<td>CS6-3-L</td>
<td>19.03</td>
<td>1.61</td>
<td>8.71</td>
<td>1.905</td>
</tr>
<tr>
<td>CS1-90</td>
<td>179.55</td>
<td>19.95</td>
<td>79.80</td>
<td>17.454</td>
<td>CS6-4-R</td>
<td>221.89</td>
<td>19.58</td>
<td>101.16</td>
<td>22.125</td>
</tr>
<tr>
<td>CS1-120</td>
<td>34.79</td>
<td>8.20</td>
<td>13.30</td>
<td>2.908</td>
<td>CS6-4-C</td>
<td>97.94</td>
<td>0.36</td>
<td>48.79</td>
<td>10.671</td>
</tr>
<tr>
<td>CS2-R</td>
<td>200.01</td>
<td>4.98</td>
<td>97.51</td>
<td>21.329</td>
<td>CS6-6-R</td>
<td>155.63</td>
<td>1.89</td>
<td>76.87</td>
<td>16.813</td>
</tr>
<tr>
<td>CS2-C</td>
<td>109.13</td>
<td>5.33</td>
<td>51.90</td>
<td>11.352</td>
<td>CS7-R</td>
<td>25.90</td>
<td>7.70</td>
<td>9.10</td>
<td>1.990</td>
</tr>
<tr>
<td>CS2-L</td>
<td>36.46</td>
<td>2.65</td>
<td>16.91</td>
<td>3.698</td>
<td>CS7-C</td>
<td>54.53</td>
<td>0.94</td>
<td>26.79</td>
<td>5.860</td>
</tr>
<tr>
<td>CS3-R</td>
<td>72.92</td>
<td>1.07</td>
<td>35.92</td>
<td>7.857</td>
<td>CS7-L</td>
<td>73.00</td>
<td>2.69</td>
<td>35.16</td>
<td>7.689</td>
</tr>
<tr>
<td>CS3-C</td>
<td>74.51</td>
<td>15.75</td>
<td>29.38</td>
<td>6.426</td>
<td>CS8-R</td>
<td>101.47</td>
<td>2.94</td>
<td>49.26</td>
<td>10.775</td>
</tr>
<tr>
<td>CS3-L</td>
<td>40.38</td>
<td>0.34</td>
<td>20.02</td>
<td>4.379</td>
<td>CS8-C</td>
<td>268.51</td>
<td>3.73</td>
<td>132.39</td>
<td>28.956</td>
</tr>
<tr>
<td>CS4-R</td>
<td>0.73</td>
<td>0.15</td>
<td>0.29</td>
<td>0.063</td>
<td>CS8-L</td>
<td>143.86</td>
<td>6.70</td>
<td>68.58</td>
<td>15.000</td>
</tr>
<tr>
<td>CS4-C</td>
<td>14.52</td>
<td>0.12</td>
<td>7.20</td>
<td>1.574</td>
<td>CS9-R</td>
<td>307.05</td>
<td>69.91</td>
<td>118.57</td>
<td>25.934</td>
</tr>
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<td>CS4-L</td>
<td>109.14</td>
<td>39.20</td>
<td>34.97</td>
<td>7.649</td>
<td>CS9-C</td>
<td>70.86</td>
<td>7.80</td>
<td>31.53</td>
<td>6.896</td>
</tr>
<tr>
<td>CS4-2-Rx</td>
<td>54.22</td>
<td>3.80</td>
<td>25.21</td>
<td>5.514</td>
<td>CS9-L</td>
<td>0.35</td>
<td>0.05</td>
<td>0.15</td>
<td>0.033</td>
</tr>
<tr>
<td>CS4-2-R</td>
<td>102.10</td>
<td>3.10</td>
<td>49.50</td>
<td>10.827</td>
<td>CS10-R</td>
<td>148.11</td>
<td>2.44</td>
<td>72.83</td>
<td>15.930</td>
</tr>
<tr>
<td>CS4-2-C</td>
<td>0.79</td>
<td>0.16</td>
<td>0.32</td>
<td>0.069</td>
<td>CS10-C</td>
<td>159.32</td>
<td>21.58</td>
<td>68.87</td>
<td>15.063</td>
</tr>
<tr>
<td>CS4-2-L</td>
<td>25.58</td>
<td>0.70</td>
<td>12.44</td>
<td>2.721</td>
<td>CS10-L</td>
<td>37.79</td>
<td>0.54</td>
<td>18.62</td>
<td>4.073</td>
</tr>
<tr>
<td>CS5-R</td>
<td>123.59</td>
<td>0.51</td>
<td>61.54</td>
<td>13.461</td>
<td>CS11-R</td>
<td>1.60</td>
<td>0.25</td>
<td>0.67</td>
<td>0.147</td>
</tr>
<tr>
<td>CS5-C</td>
<td>176.37</td>
<td>3.73</td>
<td>86.32</td>
<td>18.880</td>
<td>CS11-C</td>
<td>70.67</td>
<td>8.95</td>
<td>30.86</td>
<td>6.750</td>
</tr>
<tr>
<td>CS5-L</td>
<td>42.02</td>
<td>0.50</td>
<td>20.76</td>
<td>4.540</td>
<td>CS11-L</td>
<td>472.17</td>
<td>140.09</td>
<td>166.04</td>
<td>36.316</td>
</tr>
<tr>
<td>CS6-1-R</td>
<td>49.34</td>
<td>4.05</td>
<td>22.65</td>
<td>4.953</td>
<td>CS13-R</td>
<td>0.08</td>
<td>0.01</td>
<td>0.03</td>
<td>0.008</td>
</tr>
<tr>
<td>CS6-1-C</td>
<td>56.88</td>
<td>3.30</td>
<td>26.79</td>
<td>5.860</td>
<td>CS13-C</td>
<td>444.83</td>
<td>40.04</td>
<td>202.40</td>
<td>44.269</td>
</tr>
<tr>
<td>CS6-1-L</td>
<td>173.56</td>
<td>5.47</td>
<td>84.05</td>
<td>18.383</td>
<td>CS13-L</td>
<td>330.72</td>
<td>14.17</td>
<td>158.27</td>
<td>34.618</td>
</tr>
<tr>
<td>CS6-2-R</td>
<td>112.80</td>
<td>23.02</td>
<td>44.89</td>
<td>9.819</td>
<td>CS14-R</td>
<td>222.24</td>
<td>8.82</td>
<td>106.71</td>
<td>23.340</td>
</tr>
<tr>
<td>CS6-2-L</td>
<td>0.71</td>
<td>0.14</td>
<td>0.29</td>
<td>0.063</td>
<td>CS14-C</td>
<td>245.58</td>
<td>42.02</td>
<td>101.78</td>
<td>22.262</td>
</tr>
<tr>
<td>CS14-L</td>
<td>83.002</td>
<td>8.74</td>
<td>37.13</td>
<td>8.121</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

From 335 suspended load samples, 212 samples were collected at 16 cross sections along the northern reach of the Tigris River. Water samples were filtered in the lab using filter paper of 4 µm retention. The moisture contents of filter papers were determined before use to neutralize them. The
retention weights of fine particles were determined and converted to sediment concentrations in (g/l) units.

Using ADCP velocity measurements gives the opportunity to measure the near bed velocity and even bed moving velocity (Atsuhiro et al., 2009). The unmeasured part of the velocity profile is the uppermost part between the water surface and the ADCP transducer which was about 11cm. On the other hand, the unmeasured zone of suspended sediment was the zone near the bed where the sampler could not take a sample. To compensate for the unmeasured zones for both, extrapolation techniques proposed by van Rijn (1993) were applied. According to van Rijn (1993), velocity on the water surface was assumed equal to the closest measured velocity on the water column, whilst the lower part of concentration profile was extrapolated by three methods and the average of them was considered (see Fig. 3.8). These three extrapolations are described as follow:

1. The sediment concentration on the bed was assumed to be equal to that in the closest sampling point.

2. The sediment concentration on the bed was calculated from the power formula $c = A \cdot Y^B$

where:
- $Y$: (h-z)/z = dimensionless vertical coordinate,
- h: water depth,
- z: height above bed,
- A, B are coefficients determined by applying a regression method on the measured concentrations of the first three sampling points above the bed.

   a. Selecting $B = 0.1$ to 5 varied by step 0.1

   a. Computing $A = \frac{1}{3} \sum \frac{(Y_i^B c_i)}{\sum (Y_i^B)}$ ...................................................... (3.1)

   b. Computing $T = \sum (AY_i^B - c_i)$ ................................................................. (3.2)

   b. Repeating the procedure for all the range of B. The A and B coefficients corresponding to a minimum $T$-value are selected as the "best" coefficients.

3. The sediment concentration on the bed was computed from the exponential formula $c = e^{Ez + F}$

where:
- z: height above bed
- E, F are coefficients determined by applying a linear regression method on the measured concentrations of the first three sampling points above the bed.

Figure 3.8: The complements of the velocity profile and the sediment concentration profile (van Rijn, 1993).
The depth-integrated suspended sediment load \( (g_s) \) per unit width was computed as:

\[
g_s = \sum_{i=1}^{N} \left( v_i c_i + v_{i-1} c_{i-1} \right) / 2 \quad \text{...............}(3.5)
\]

where:
- \( v_i \): flow velocity at height \( z_i \) above the bed,
- \( c_i \): sediment concentration at height \( z_i \) above the bed,
- \( N \): total number of points along the water column including extrapolated and interpolated values.

Figure (3.9) shows the velocity and concentration profiles at some cross sections. The rates of suspended sediment \( (g_s) \) per unit width at sampling points were listed in Table (3.4).

---

**Figures 3.9:** The velocity and the sediment concentration profiles at (A) CS1-60, (B) CS6-3-L, (C) CS7-C, (D) CS9-L.
Table 3.4: Calculations of suspended load rates (kg/s) per meter width at sampling points.

<table>
<thead>
<tr>
<th>Cross section</th>
<th>Right point</th>
<th>Center point</th>
<th>Left point</th>
<th>Extra point</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>0.898</td>
<td>0.867</td>
<td>0.800</td>
<td>0.455</td>
</tr>
<tr>
<td>C2</td>
<td>1.156</td>
<td>0.731</td>
<td>0.463</td>
<td>-</td>
</tr>
<tr>
<td>C3</td>
<td>0.121</td>
<td>0.137</td>
<td>0.136</td>
<td>-</td>
</tr>
<tr>
<td>C4</td>
<td>0.199</td>
<td>0.177</td>
<td>0.746</td>
<td>-</td>
</tr>
<tr>
<td>C4-2</td>
<td>1.129</td>
<td>0.656</td>
<td>0.744</td>
<td>0.357</td>
</tr>
<tr>
<td>C5</td>
<td>0.322</td>
<td>0.373</td>
<td>0.161</td>
<td>-</td>
</tr>
<tr>
<td>C6-1</td>
<td>0.109</td>
<td>0.065</td>
<td>0.312</td>
<td>-</td>
</tr>
<tr>
<td>C6-2</td>
<td>0.213</td>
<td>-</td>
<td>0.138</td>
<td>-</td>
</tr>
<tr>
<td>C6-3</td>
<td>0.093</td>
<td>-</td>
<td>0.729</td>
<td>-</td>
</tr>
<tr>
<td>C6-4</td>
<td>0.125</td>
<td>0.884</td>
<td>0.773</td>
<td>-</td>
</tr>
<tr>
<td>C7</td>
<td>0.607</td>
<td>0.124</td>
<td>1.694</td>
<td>-</td>
</tr>
<tr>
<td>C8</td>
<td>1.244</td>
<td>0.427</td>
<td>0.477</td>
<td>-</td>
</tr>
<tr>
<td>C9</td>
<td>0.862</td>
<td>0.590</td>
<td>0.117</td>
<td>-</td>
</tr>
<tr>
<td>C10</td>
<td>0.336</td>
<td>0.693</td>
<td>1.040</td>
<td>-</td>
</tr>
<tr>
<td>C11</td>
<td>0.743</td>
<td>0.473</td>
<td>1.141</td>
<td>-</td>
</tr>
<tr>
<td>C13</td>
<td>0.908</td>
<td>0.857</td>
<td>0.412</td>
<td>-</td>
</tr>
<tr>
<td>C14</td>
<td>0.739</td>
<td>1.051</td>
<td>1.640</td>
<td>-</td>
</tr>
</tbody>
</table>
Flood Capacity and Its Improvement

Several cities have been built on the banks of the Tigris since the dawn of civilization. Amongst these is Baghdad, the capital of Iraq. Parts of all of these cities (Mosul, Samara, Baghdad and Al-Kut) were inundated by the spring floods from the river in 1954, 1971 and 1988. To overcome this problem various hydraulic projects have been constructed along the Tigris and its tributaries in Mosul, Samara, Dokan and Darbandikhan. The control of the river was most efficient during the twentieth century after huge dams were built to trap some of the waters (Al-Ansari and Knutsson, 2011). Despite the presence of many hydraulic structures upstream of the city, parts of Baghdad were inundated in 1988.

During the last two decades growing islands have become noticeable features in the Tigris channel within Baghdad City, where the number of islands is increasing with time. In this contribution the impact of the human activities of dam building, bank lining and dumping of debris within the channel at Baghdad has led to changes in the geometry of the river and its ability to carry flood waters.

4.1. Bathymetric Surveys

Due to the recurrence of inundation in parts of Baghdad City, The Ministry of Water Resources, conducted several surveys for the river in 1976 (Geohydraulique, 1977), and 1991 (University of Technology, 1992). In 2008, the Ministry of Water Resources conducted a third survey to investigate the changes in river geometry due to the high depositional rates.

The survey conducted in 2008 covered 49km of the river’s reach, from Al-Muthana Bridge in the north to the confluence with the Diyala River in the south (Fig. 4.1). A total of 219 cross sections were surveyed at 250m intervals (some cross sections were conducted at smaller intervals especially at meanders), as shown in Figure (4.1).

The repeated surveys indicated that the average slope of the bed of the Tigris within Baghdad was substantially greater in 2008 (5 cm/km) than it was in 1976 (1.03 cm/km) and more than twice than what it was in 1991 (2.45 cm/km). The irregularities in the cross sections of the river reflect the variations in flow velocity controlling erosion or deposition. It is important to note that most of the suspended sediments formerly transported with flood events to the reach were now being trapped in the upstream reservoirs, so that the river was attempting to achieve a new stable regime (Morris and Fan, 2010).
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Figure 4.1: The cross sections of the Tigris River from the bathymetric survey of 2008.

4.2. Flow Simulation

4.2.1. HEC-RAS Simulation Model

One of the well-known simulation models for river flow is HEC-RAS (Hydrologic Engineering Centre – River Analysis System). This model can perform 1-D water surface profile calculations for steady/unsteady gradually varied flow in open channels. The equation that is used for computing the water surface profile is the energy equation (Eq. 4.1). It is solved by iterative procedure which is called the standard step method from one cross section to the next (HEC, 2010).

\[ Z_2 + Y_2 + \frac{a_2 v_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 v_1^2}{2g} + h_e \] ................................................................. 4.1

Where
\[ Z_1, Z_2 = \text{bed elevation of the channel at sections 1 and 2 respectively (m)} \]
\[ Y_1, Y_2 = \text{water depth at sections 1 and 2 respectively (m)} \]
\[ V_1, V_2 = \text{average velocity at sections 1 and 2 respectively (m/s)} \]
\[ a_1, a_2 = \text{velocity weighting coefficients at sections 1 and 2 respectively} \]
\[ g = \text{acceleration of gravity (m/s^2)} \]
\[ h_e = \text{energy head loss between section 1 and 2 (m), which is comprised of frictional losses and contraction/expansion losses as shown in Equation (4.2)} \]

\[ h_e = L \frac{S_f}{g} + C \left[ \frac{a_2 v_2^2}{2g} - \frac{a_1 v_1^2}{2g} \right] \] ................................................................. 4.2

End of the reach

Start of the reach
Flood Capacity and Its Improvement

Where

$L = \text{reach length between section 1 and 2 (m)}$

$S_f = \text{friction slope between section 1 and 2}$

$C = \text{expansion or contraction loss coefficient}$

The water discharge is computed in the model using Manning equation (Eq. 4.3)

$$Q = \frac{1}{n} AR^{2/3}S_f^{1/2}$$

Where

$Q = \text{water discharge (m}^3/\text{s)}$

$A = \text{cross-sectional area (m}^2)$

$R = \text{hydraulic radius (m)}$

$n = \text{Manning’s roughness coefficient}$

4.2.2. Input Data for the Model

The findings of the 2008 survey were used to establish a 1-D steady flow simulation for different scenarios of discharges using the HEC-RAS program to determine the corresponding water levels in the Tigris River. Additional data concerning the locations and dimensions of the bridges were provided for the model.

A range of high discharges, those considered in previous similar studies (Geohydraulique, 1977; University of Technology, 1992) were examined in this work. This range extended from 800 to 4000 m$^3$/s. The initial river discharge was 400 m$^3$/s at the Sarai Baghdad gauging station for the period of observing water levels (59 observation points) along the lower 15 km of the studied reach. This initial discharge and the observations of water levels were used for calibrating the model. In addition, the average monthly discharge (500 m$^3$/s) for the period 2002-2012 was added to the range of the examined discharges.

A modified rating curve of the Tigris River downstream of its confluence with the Diyala River (University of Technology, 1992) was used as downstream boundary condition as shown in Equation (4.4). The backwater effect was taken into consideration in the simulation by considering four values for the Diyala River discharge as lateral inflow to the Tigris River at the end of the river reach. Three of these discharges were taken from the historical records whilst the fourth discharge was the recent discharge of the Diyala River (5 m$^3$/s) that was assumed as the default value in calibration step.

$$Z = 2.67 \ln(Q) + 9.63$$

Where

$Z = \text{water level (m)}$

$Q = \text{water flow (m}^3/\text{s)}$

4.2.3. Model Calibration and Validation

The aim of calibration in this model is to define suitable values of roughness coefficients for the main channel and the floodplain to produce minimum error in computed water levels compared with those observed in field. This was achieved by iterating for different values of Manning’s ‘$n$’ until a good agreement between the computed water levels and those observed along the lower part of the river reach was achieved. Root Mean Square Error (RSME) was used to measure the
produced errors. The Manning’s ‘n’ coefficient of value 0.0285 for the main channel and 0.042 for the floodplain gave a minimum RSME value of 0.026m for calibration step (see Fig. 4.2).

Five discharges of the Tigris River ranging from 400 to 1300 m$^3$/s were used for model validation. The computed water levels were compared with the corresponding water level records in Sarai Baghdad during the period 2002-2012. The agreement between simulation and observations was good and the RSME did not exceed 0.046m.

Figure 4.2: The calculated water surface elevations for different discharges of the Tigris River with the calibration and validation data.

4.2.4. Prediction of Flood Capacity

The procedure of predicting the flood capacity started by applying the average monthly discharge of 500 m$^3$/s and the corresponding water profile was computed. Then the upstream discharge was increased in steps as those considered in previous studies. For each applied discharge, water surface profile was computed as shown in Figure (4.2). This procedure was continued until the bankfull flow condition was achieved. Then more increase in water discharge produced water levels higher than river banks levels at some locations causing inundation for the nearby areas. This procedure was applied with lateral inflow, as the Diyala River flow, of 5 m$^3$/s and assumed as base scenario.

Figure (4.2) shows that the discharges higher than 2700 m$^3$/s have risk of partial inundation at some locations along the northern reach of the Tigris River. The critical water surface elevation for inundation in the reach was 35 m at station 43000 m. For discharges greater than 3500 m$^3$/s the inundation could take place along approximately 9 km of the reach. For the southern part of the reach, the inundation is not expected to occur with discharges lower than 3500 m$^3$/s.

The calculated slopes of water surface for the base scenario varied from 6.03 to 6.84 cm/km for discharges between 400 and 1500 m$^3$/s respectively. For discharges between 2500 and 2700 m$^3$/s, the slopes were between 8.59 and 8.96 cm/km respectively, whilst it could reach 10 cm/km for the discharges of 3500 and 4000 m$^3$/s.
The used rating curve at the downstream boundary requires updating for high water levels to give more reliable estimations according to the new morphological conditions of the river.

Three more scenarios were applied using the same procedure with different lateral inflow of values 25, 50 and 100 m³/s respectively to investigate the effect of backwater on the inundation levels in the Tigris River. The differences in water surface profile for each scenario were calculated with respect to the base scenario and the results are listed in Table (4.1). These differences indicated that the backwater effect has no significant influence on water levels during high discharges.

Table 4.1: Differences in the water surface profiles (m) for all flow scenarios with respect to the base scenario.

<table>
<thead>
<tr>
<th>Tigris River flow m³/s</th>
<th>Lateral inflow 25 m³/s</th>
<th>Lateral inflow 50 m³/s</th>
<th>Lateral inflow 100 m³/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>0.040</td>
<td>0.102</td>
<td>0.209</td>
</tr>
<tr>
<td>500</td>
<td>0.038</td>
<td>0.087</td>
<td>0.186</td>
</tr>
<tr>
<td>800</td>
<td>0.030</td>
<td>0.067</td>
<td>0.142</td>
</tr>
<tr>
<td>1100</td>
<td>0.023</td>
<td>0.052</td>
<td>0.110</td>
</tr>
<tr>
<td>1300</td>
<td>0.019</td>
<td>0.044</td>
<td>0.095</td>
</tr>
<tr>
<td>1500</td>
<td>0.017</td>
<td>0.039</td>
<td>0.083</td>
</tr>
<tr>
<td>2500</td>
<td>0.010</td>
<td>0.023</td>
<td>0.049</td>
</tr>
<tr>
<td>2700</td>
<td>0.009</td>
<td>0.021</td>
<td>0.047</td>
</tr>
<tr>
<td>3500</td>
<td>0.008</td>
<td>0.020</td>
<td>0.045</td>
</tr>
<tr>
<td>4000</td>
<td>0.007</td>
<td>0.019</td>
<td>0.043</td>
</tr>
</tbody>
</table>

Comparing the base scenario for water levels at Sarai Baghdad with those obtained by previous studies showed that for the survey of 2008, water levels were lower than those of the 1976’s survey for low discharges but higher than those in high discharges. They were always lower than the water levels obtained in 1991 as shown in Figure (4.3).

Figure 4.3: Comparison of the calculated water levels at the Sarai Baghdad for 1976, 1991 and 2008 surveys with observations.

4.3. Dredging Operations

The Ministry of Water Resources (MoWR) in Iraq decided to dredge parts of the river as a treatment for the sedimentation problems in the Tigris River. The main aims of removing the deposited sediment by dredging were to recover the designed cross sectional area to maintain the
flood capacity of the river, maintaining water intakes and preventing clogging them, and improving the appearance of the river and its banks.

A dredging plan was drawn up by the Office of Executing Rivers Dredging Works (OERDW), one of the directorates of MoWR, which included removing the main obstacles in the northern reach of the river and about half of the southern reach (MoWR, 2013) as shown in Figure (4.4). Two islands were excluded from the plan, one in the northern reach and the second in the southern reach.

According to the bathymetry of 2008 and additional local surveys that were conducted by OERDW, the quantities of sediment to be removed were estimated to be about 3443000 m$^3$. The estimations neglected any new deposition because the rates of erosion and deposition were unknown.

In addition to the above plan, two more dredging plans were suggested to add to the OERDW dredging plan. The first suggested removing most of depositions within the upper banks and the stacks of the dredged materials along the northern reach and the first half of the southern reach as shown in Figure (4.5). The suggested locations in this plan included cleaning the narrowest sections which have significant influence on the river capacity and detention of water levels during high discharges as shown in Figure (4.6). The estimated quantities of sediment to be removed in this additional plan were about 1330000 m$^3$.

The second additional plan included the dredging of Dura Island, the biggest island in the southern reach of the river, to a depth of 2m under water and the right bank deposition just upstream.
from the island (Fig. 4.5). The fact that this island takes about 59% of the river width and bisects a large meander puts it under the spotlight to investigate the removal effect on the hydraulic performance of the river. The estimated quantities of sediment to be removed in this additional plan were about 610000 m$^3$.

![Figure 4.5: The suggested areas for dredging according to the first additional plan (cyan) and the second additional plan (red).](image1)

4.3.1. Implementation of Dredging Plans in Simulation Model

The OERDW dredging plan was compiled with reference to modifications on the dimensions of river geometry obtained from the bathymetry of 2008 and the additional plans were combined with the OERDW plan individually producing two more modified geometries for the river.
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The three modified geometries were used in the HEC-RAS model individually to investigate their influence on the hydraulic performance and flood capacity of the river. The same procedure that followed in section (4.2.4) was used to determine the associated water surface profiles for the range of discharges (400, 500, 800, 1100, 1300, 1500, 2500, 2700, 3500 and 4000 m³/s) that were used before. Some changes on the roughness coefficients were applied to the cross sections where removing obstacles were suggested, so that the effect of high roughness due to the presence of the obstacles was removed from these cross sections.

Water surface profiles were calculated along the Tigris River reach in Baghdad for the range of discharges and lateral inflow (5 m³/s) as shown in Figure (4.7).

4.3.2. Evaluation of Dredging Plans

In general, at the low discharges (400, 800 and 1100 m³/s), figure (4.7) shows a clear drop in water levels along the river reach and the water profiles are mostly identical for all the dredging plans. This drop will affect the functionality of the water intakes especially along the northern reach of the river, so more caution should be taken to keep the water intakes functional.

The effect of the second additional plan seemed very limited and took place only at the location of Dura Island and in its vicinity. The differences between water profiles of the OERDW plan and the first additional plan are limited on the northern reach of the river.

At high discharges (2500 and 3500 m³/s) there are convergences between the profiles of all dredging plans and the pre-dredging condition. However, there is a limited advantage for the first additional plan where its water profile was lower than other profiles. This indicates the effect of the bank deposition which is more significant on the flood capacity.

Calculated water levels at Sarai Baghdad gauging station were compared with those obtained by the previous studies (Geohydraulique, 1977; University of Technology, 1992) and with a summary of historical records for the last four decades as shown in figure (4.8), where some conclusions can be drawn as follows:

The records of water levels during 1970s were higher than the results of the dredging plans for the discharges 400, 500 and 800 m³/s and very close to the results of the discharges of 1100, 1300 and 1500 m³/s and significantly lower than the results for high discharges. For this reason, the dredging plans cannot achieve the water levels for the cases of floods in the 1970s, because the river banks were not protected during that period and river course was wider as shown in Figure (4.9).

The available records of water levels during 1980s were close to the results of the dredging plans for the discharges 500, 800 and 1100 m³/s, whilst they were higher for the discharges 2500 and 2700 m³/s. The effect of headwater dams that were constructed during that decade was clear from the values of the peaks. The records of water levels during 1990s were very close to those of the previous decade. The river seemed more tranquil. The results of the dredging plans were closer to the records of the discharges 400 to 1500 m³/s. The records of water levels during the 2000s were very close to those of the 1990s but they covered a narrower range of discharges 400 to 1300 m³/s.
Figure 4.7: The water surface profiles of different water discharges along the river reach for the pre and after dredging operations using the data of the bathymetric survey of 2008.
Figure 4.8: The computed water levels at the Sarai Baghdad for all dredging plans with the results of previous studies and the observations of the last four decades.

Figure 4.9: The cross section of the Sarai Baghdad gauging station for different periods.

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Sediment Transport in the Tigris River

Sediment is transported in natural rivers in two modes which are defined by three different criteria. Either by the type of movement, in contact with the bed as bedload or in suspension as suspended load, or by the method of measurement, measured load along the water column and on the bed or unmeasured load at the lower zone above the bed where neither bedload sampler nor point sampler can measure, or by the source of sediment, wash load which are the finer sediment those transport in suspension always and bed material load which can be suspended or move in contact with the bed (Julien, 2010). Suspended load represents the relatively finer portion that is transported in suspension by the effect of flow turbulence whilst bedload, the coarser portion, moves in contact with the river’s bed by sliding, rolling or saltation due to the boundary shear stress. An exchange may occur between suspended load and bedload, and between bedload and bed material depending on the size of the sediment particles, flow transport capacity, flow velocity, and boundary shear stress (Hickin, 1995; WMO, 2003). In addition, it is difficult to separate washload, which depends on the upstream supply of sediment rather than transport capacity, and the suspended particles from bed material. The d_{10} of the bed material is commonly used as a cut-off between the washload (d_i < d_{10}) and bed material load (d_i > d_{10}) (Julien, 2010).

The main sources of fluvial sediments are watershed erosion, stream erosion and human activities (Vanoni, 2006). Since sediment sources are unlimited and streams have sufficient sediment transport capacity, sediment transport will continue (Friend, 1993). The condition of limited supply of fine sediment is the case in most natural rivers (Hickin, 1995), so in such rivers, bed-material is the main source of sediment load.

Based on the foregoing, to estimate total load transport rate in the Tigris River and find the best estimation formula for the studied river reach, the following steps were considered:

1. Evaluating the situation of the river and whether it has limited sources of sediment and what are possible sources of sediment.
2. Investigating the characteristics of bed sediment, which is the main source of transported sediment in the river.
3. Determining the bedload and suspended load transport rates directly by using field measurements to calculate the total load accordingly.
4. Estimating sediment transport rate indirectly using mathematical formulas and determining the best formula relating to the field measurements.
5. Testing the ability of the mathematical formulas to represent the spatial variation of sediment load.
6. Improving spatial sediment rating curve.

5.1. Flow Regulation and its Effect on Sediment Supply

The flow of the Tigris River inside Baghdad is fully controlled by a system of dams and regulators on the main river and its tributaries upstream of Baghdad as shown in Figure (1.2). This regulation disturbed the delivery of sediment from the head water. High trap efficiency for particles by dams can reach up to 95.33% at Mosul Dam reservoir on the main river (Issa, 2015).
The only uncontrolled source of sediment that can be delivered to Baghdad is the area restricted from the lower sub-basin of the Adhaim tributary and the catchment between the Samarra Barrage and Baghdad (Figure 1.2), as well as the bed and banks erosions. The delivery of fine sediment from the Adhaim Tributary has not been measured, but a glance at the possible extra flow contribution (rather than the flow released from the Adhaim Dam), can give an indication for the estimated sediment delivery. The extra water flow contribution from the Adhaim Tributary sub-basin and Tharthar Lake back feed toward the Tigris River was determined using the mass balance concept. That contribution did not exceed 260 m³/s during 2004-2005, which was a moderate year compared with recent more dry years. As an average, the extra contribution was 8% of the average monthly discharge at the Sarai Baghdad for the same year. So, with all this evidence, the river was considered as being of a limited source of sediment supply before entering Baghdad, and bed sediment has to be considered as the main source of transported sediment.

5.2. Bed Sediment Characteristics

The laboratory analysis of bed sediment samples showed that the bed of the river reach is mainly covered by sand (Fig 5.1.a) whilst silt and clay cover small portions of the studied reach (Fig 5.1.b and Fig 5.1.c). The percentage of sand: silt: clay ratio was 90.74:6.86:2.4. The size distribution curves of bed sediment showed deviation from the straight line, generally at two points. The first lies between 3.8 to 4 phi (0.074-0.0625 mm) whilst the second at 1.8 phi (0.3mm) (see Fig. 3.7). Using Folk’s classification (Folk, 1980), 72% of the sediment was sand and 24% were silty sand whilst the remaining 4% was sandy silt. When USDA textural soil classification (USDA, 1987) was used, then the majority of the sediment (72%) was sand followed by loamy sand (20%), sandy loam (4%) and loam (4%). More recent and detailed classification of Blott and Pye (Blott and Pye, 2012) showed that 54.2% of the samples were very slightly silty very slightly clayey sand. This is followed by 15.2% very slightly clayey slightly silty sand, 11% very slightly silty sand, 6.5% slightly clayey silty sand, 6.5% very slightly clayey sand, 2.2% very slightly clayey silty sand, 2.2% slightly silty slightly clayey sand and 2.2% slightly clayey sandy silt. More details are available from (Al-Ansari et al., 2015b).

The bed sediment showed that the average median size within the reach was 2.49 phi (0.177mm) whilst the mean size was 2.58 phi (0.16mm). In addition, the sediments were moderately sorted. About 35% of the samples were well sorted where 24% were poorly sorted. The former samples were mainly located in places where the flow was not disturbed. The majority of the sediments were fine skewed (52.2%) whilst the remainders were strongly fine skewed or nearly symmetrical. As far as the kurtosis of the sediments are concerned, generally more than half of the samples (56.5%) were leptokurtic, whilst 17.4% were mesokurtic and 13% were extremely leptokurtic. Previous studies by Al-Ansari and Toma (1984 and 1985) showed that the sediments were coarser on the bed of the Tigris River. This is believed to be due to the construction of the Adhaim Dam in 1999, where coarse sediments in the valley downstream of the dam can’t be delivered as before, as well as the competence of the river is reduced due to the dam operation plan. For this reason the grain size of the bed decreased in size and the bedload transported is now of a size ranging from about 0.0625 to 1.0 mm in diameter (4-0 phi).
Figure 5.1: (a) The distribution of sand; (b) silt; (c) clay on the bed of the Tigris River at Baghdad.
Chapter Five

5.3. Bedload Transport Rate

The computations of bedload transport rate were conducted according to the Edwards and Glysson (1998) procedure using the results of bedload samples in table (3.3). The maximum bedload was 3.938 kg/s at CS8 associated with water discharge of 643.32 m³/s and minimum bedload was 0.682 kg/s at CS4 associated with a water discharge of 445.1 m³/s as shown in table (5.1). The average bedload was determined as 2.099 kg/s associated with 0.889 standard deviation. This variation in bedload was affected spatially, since the sampling procedure adopted along the sampling cross sections of the river reach were varied in their morphological characteristics, such as meanders, islands and bank depositions. An effort is still needed to establish a bedload rating curve considering the spatial variation of bedload and the morphology of the river.

Table 5.1: Measured bedload and suspended load rates and calculated total load rates along the northern reach of the Tigris River.

<table>
<thead>
<tr>
<th>C.S.</th>
<th>Gb (kg/s)</th>
<th>Gs (kg/s)</th>
<th>Cppm</th>
<th>Gb / Gs (%)</th>
<th>Gt (kg/s)</th>
<th>Qw (m³/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>1.268</td>
<td>112.430</td>
<td>246.68</td>
<td>1.13</td>
<td>113.698</td>
<td>457.38</td>
</tr>
<tr>
<td>CS2</td>
<td>2.590</td>
<td>190.310</td>
<td>415.99</td>
<td>1.36</td>
<td>192.900</td>
<td>459.02</td>
</tr>
<tr>
<td>CS3</td>
<td>1.341</td>
<td>29.100</td>
<td>62.89</td>
<td>4.61</td>
<td>30.441</td>
<td>464.41</td>
</tr>
<tr>
<td>CS4</td>
<td>0.682</td>
<td>92.160</td>
<td>207.79</td>
<td>0.74</td>
<td>92.842</td>
<td>445.10</td>
</tr>
<tr>
<td>CS4-2</td>
<td>1.142</td>
<td>136.330</td>
<td>302.44</td>
<td>0.84</td>
<td>137.472</td>
<td>452.33</td>
</tr>
<tr>
<td>CS5</td>
<td>1.599</td>
<td>37.580</td>
<td>77.10</td>
<td>4.25</td>
<td>39.179</td>
<td>489.23</td>
</tr>
<tr>
<td>CS6-1</td>
<td>2.901</td>
<td>44.620</td>
<td>81.44</td>
<td>6.50</td>
<td>47.521</td>
<td>549.88</td>
</tr>
<tr>
<td>CS6-2</td>
<td>0.924</td>
<td>31.130</td>
<td>109.09</td>
<td>2.97</td>
<td>32.054</td>
<td>286.41</td>
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<tr>
<td>CS6-3</td>
<td>0.190</td>
<td>30.330</td>
<td>121.27</td>
<td>0.63</td>
<td>30.520</td>
<td>251.02</td>
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<td>CS6-4</td>
<td>3.420</td>
<td>96.620</td>
<td>172.61</td>
<td>3.54</td>
<td>100.040</td>
<td>561.78</td>
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<tr>
<td>CS7</td>
<td>1.564</td>
<td>180.610</td>
<td>278.10</td>
<td>0.87</td>
<td>182.174</td>
<td>651.71</td>
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<tr>
<td>CS8</td>
<td>3.938</td>
<td>143.980</td>
<td>224.60</td>
<td>2.74</td>
<td>147.918</td>
<td>643.32</td>
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<tr>
<td>CS9</td>
<td>2.171</td>
<td>128.670</td>
<td>243.42</td>
<td>1.69</td>
<td>130.841</td>
<td>530.44</td>
</tr>
<tr>
<td>CS10</td>
<td>2.716</td>
<td>145.220</td>
<td>253.90</td>
<td>1.87</td>
<td>147.936</td>
<td>573.97</td>
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<tr>
<td>CS11</td>
<td>2.630</td>
<td>138.600</td>
<td>240.48</td>
<td>1.90</td>
<td>141.230</td>
<td>578.38</td>
</tr>
<tr>
<td>CS13</td>
<td>2.579</td>
<td>72.700</td>
<td>137.68</td>
<td>3.55</td>
<td>75.279</td>
<td>529.97</td>
</tr>
<tr>
<td>CS14</td>
<td>1.930</td>
<td>127.890</td>
<td>245.76</td>
<td>1.51</td>
<td>129.820</td>
<td>522.23</td>
</tr>
</tbody>
</table>

5.3.1. Bedload Prediction Formulas

Wide spectrums of bedload prediction formulas were proposed and developed by many researchers depending on their different approaches. For each approach, a specified concept was considered as motivation for deriving the formula and a certain number of parameters were controlled in the lab measurements to estimate the constants for the formula. Twenty bedload formulas were selected and applied on the study reach to find the best suitable formulas for the Tigris River in Baghdad. Brief descriptions for the selected approaches and their formulas are given below:
5.3.1.1. Shear stress approach

The movement of bed material particles starts when the criteria of incipient motion is exceeded. So, shear stress near the bed will entrain the sediment particles to motion as long as the shear stress is greater than the critical shear stress of the particles. The following formulas which belong to this approach were used in this work:

a. DuBoys formula (1935) (Graf, 1971)

b. Shields formula (1936) (Yang, 1996)

c. Kalinske formula (1947) (Yang, 1996)

d. Cheng-Simons-Richardson formula (1965) (Yang, 1996)


5.3.1.2. Energy slope approach

The bedload motion is initiated due to the portion of energy losses coming from the grain resistance (Yang, 1996). The following formulas were used from this approach:

a. Meyer-Peter formula (1934) (Vanoni, 2006)

b. Meyer-Peter and Muller formula (1948) (Graf, 1971)

5.3.1.3. Discharge approach

In natural rivers, critical unit discharge was used as an indication on starting bedload sediment motion when it is exceeded by water discharge (Talukdar et al., 2012). The following formulas which belong to this approach were used in this work:

a. Schoklisch formula (1934, 1943) (Yang, 1996)

b. Casey formula (1935) (Khalaf, 1988)

5.3.1.4. Probabilistic approach

Probability concepts were introduced in bedload prediction by the pioneering work of Einstein in 1942. The turbulent flow fluctuations are the driver for sediment entrainment rather than the flow forces exerted on the particles. Both of the entrainment and the deposition were expressed in probability terms (Yang, 1996).

a. Einstein bedload function (1950)

b. Einstein-Brown formula (1950)

5.3.1.5. Regression approach

Data driven models (regression, ANN) were used to explain the bedload transport process due to the limitations of defining this complex process into a precise formula (Talukdar et al., 2012). The following formulas were used within this approach:

a. Rottner formula (1959)

b. Yalin formula (1963) (Yalin, 1977)


5.3.1.6. Equal mobility approach

The flow forces act on the exposed particles causing mobilization with possibility of participation of the substrate particles into bedload movement at scour zones due to their exposure on the surface (Yang, 1996).


5.3.1.7. Power Concept

This approach has developed from the concept that there is a relationship between the available energy to the river with the rate of work done by the river to transport sediment (Yang, 1996). The following formula was used within this approach:

a. Bagnold formula (1966)

The mathematical equations of the twenty bedload formulas aforementioned were listed in Table (5.2).

Table 5.2: Bedload prediction formulas applied on the Tigris River in Baghdad.

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Bedload formula</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>DuBoys formula (1935)*</td>
<td>( g_s = \varphi \rho \tau_s (\tau_s - \tau_c) )</td>
<td>( \tau_s = \gamma RS )</td>
<td></td>
</tr>
<tr>
<td>Shields formula (1936)</td>
<td>( g_s = 10qS \frac{(\tau_s - \tau_c)}{(Gs - 1)^2 d_{so}} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kalinske formula (1947)</td>
<td>( g_s = \rho u_d d_{so} \tan \left( \frac{\tau_c}{\tau_s} \right) )</td>
<td>( \tau_c = 12d_{so} )</td>
<td></td>
</tr>
<tr>
<td>Cheng-Simons-Richardson formula (1965)</td>
<td>( g_s = K_f \left( \frac{\tau_s}{\tau_c} - 1 \right) )</td>
<td>( K_f = \frac{V}{u_c (\gamma_s - \gamma)} \left( \frac{d_e}{d_s} \right) )</td>
<td></td>
</tr>
<tr>
<td>d_s = \sqrt{\frac{18 \mu \rho g}{(\rho_s - \rho)g}}</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wong-Parker formula (2006)</td>
<td>( g_s = \phi \rho \sqrt{(Gs - 1)} \gamma d_{so}^3 )</td>
<td>( \phi = 3.97(\theta - 0.0495)^{1.5} )</td>
<td></td>
</tr>
<tr>
<td>Meyer-Peter formula (1934)</td>
<td>( g_s = 250q \gamma S - 45 d_{so} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Meyer-Peter and Muller formula (1948)</td>
<td>( g_s = 8 \frac{K_r}{\rho_s - \rho} \left( \frac{1}{\rho} \right) \left( \frac{kr'}{K'} \right) (\tau_s - \tau_c) )</td>
<td>( kr' = \frac{26}{d_{so}^{1.5}} )</td>
<td></td>
</tr>
<tr>
<td>Schoklisch formula (1934)</td>
<td>( g_s = \frac{7000}{\sqrt{d_s}} S^{3/2} (q - q_c) )</td>
<td>( q_c = \frac{1.944 \times 10^{-5}}{S^{5/2}} d_s )</td>
<td></td>
</tr>
<tr>
<td>Schoklisch formula (1943)</td>
<td>( g_s = 2500 S^{3/2} (q - q_c) )</td>
<td>( q_c = \frac{0.26(Gs - 1)^{5/2}}{S^{7/2}} d_{so}^{3/2} )</td>
<td></td>
</tr>
<tr>
<td>Casey formula (1935)</td>
<td>( g_s = 0.3675 \rho_s S^{3/2} (q - q_c) )</td>
<td>( q_c = \frac{6.5 \times 10^{-6}}{S^{7/4}} d_{so}^{3/2} )</td>
<td></td>
</tr>
<tr>
<td>Author(s)</td>
<td>Bedload formula</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------------------</td>
<td>---------------------------------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Einstein bedload function (1950)**</td>
<td>( g_n = \phi\rho_n \sqrt{(G_s-1)g}d_i )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Einstein-Brown formula (1950)</td>
<td>( F_i = \sqrt{\frac{2}{3} + \frac{36u^2}{(G_s-1)gd_i}} - \frac{36u^2}{(G_s-1)gd_i} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rottner formula (1959)</td>
<td>( g_r = \rho \sqrt{(G_s-1)gD} \left[ \frac{V}{\sqrt{(G_s-1)gD}} \left( 0.667 \left( \frac{d_50}{D} \right)^{0.3} \right) + 0.14 \right] - 0.778 \left( \frac{d_50}{D} \right)^{0.5} )</td>
<td></td>
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<tr>
<td>Yalin formula (1963)</td>
<td>( A_y = 2.45 \left[ \frac{\tau_c}{(\gamma - \gamma)d_i} \right]^{1/2} \left( \frac{\rho}{\rho_s} \right)^{0.4} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Van Rijn formula (1984)</td>
<td>( g_r = 0.053 \rho \sqrt{(G_s-1)g}d_i \left( \frac{\theta}{\theta_c} - 1 \right)^{1.5} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Julien formula (2002)</td>
<td>( g_s = 18 \rho \sqrt{g}d_{50}^{0.5} \left( \theta \right)^{0.5} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carmenen-Larson formula (2005)</td>
<td>( g_s = \phi \rho \sqrt{(G_s-1)g}d_i )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wilcock formula (2001)</td>
<td>( g_s = \phi \rho \frac{d_{50}^{1.5}}{(G_s-1)g} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wilcock-Crowe formula (2003)</td>
<td>( g_s = \phi \rho \frac{d_{50}^{1.5}}{(G_s-1)g} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bagnold formula (1966)**</td>
<td>( g_s = \frac{\gamma}{(\gamma - \gamma)} \sqrt{V} \left( \frac{e_s}{\tan \alpha} \right) )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* see Figure (7.6) in (Graf, 1971) is used to obtain \( \varphi_d \) and \( \tau_c \)
** Einstein procedure explained in details in (Graf, 1971) and Figure (7.13) is used to obtain \( \phi = fn(\psi) \)
*** Figure (6.5) in (Yang, 1996) is used to obtain \( e_s \) and \( \tan \alpha \)
5.3.2. Application of Bedload Prediction Formulas

Two kinds of data sets were required for applying the bedload formulas, physical properties of river bed sediment and hydraulic-geometric parameters of the study reach. These data sets were determined by direct measurements and analysis of sampled sediment. The results published by Al-Ansari et al. (2015b) contained most of the datasets, whilst other datasets were listed in Table 3.2.

The application results of the bedload formulas at sixteen cross sections along the study reach were compared with the measured bedload discharges in the same section and two indicators were used to measure the accuracy of the predicted bedload. The discrepancy ratio, which is the ratio of predicted bedload to measured one (van Rijn, 1993), was one of the indicators and the error percentage (Walling, 1977a) was the other. The comparisons of results are shown in Figure 5.2. Six zones of different discrepancy ratios were specified in the figure to explain the distribution of the results around the perfect agreement line.

Most of the formulas overestimated the bedload transport rate by more than 10 times and even 100 times relative to field measurements. Five formulas from four of the approaches predicted bedload discharges close to measurements. These formulas were Meyer-Peter (1934), Schoklitsch (1934, 1943), van Rijn (1984) and Einstein bedload function (1950) with average discrepancy ratios of 0.5, 1.51, 0.47, 1.18 and 4.06 respectively. The predictions of van Rijn (1984) and Schoklitsch (1934) formulas are distributed on both sides of the perfect agreement line. Whilst both of Meyer-Peter and Schoklitsch (1943) formulas are mainly bounded between the perfect line and discrepancy ratio \( z \). Some results of Einstein (1950) were in the area between the perfect agreement and \( r = 8 \).
Table (5.3) shows the accumulated percentages of the predicted bedload discharges according to each range of the discrepancy ratio. The higher percentage of predicted bedload within the closer range of discrepancy ratio 0.75 - 1.25 (Error% = -25 ~ +25) was equally between Schoklitsch (1934) and van Rijn formulas and the results approximately continued in this manner until the third zone of discrepancy ½ - 2 (Error% = -50 ~ +100). At this range, more than 76% of Schoklitsch (1934) and 53% and of van Rijn predictions were located within the range. The percentages of the other three formulas didn’t exceed 24% for the discrepancy range ½ - 2 (Error% = -50 ~ +100).

To clarify the behaviour of the bedload predictors at different cross sections, which having varied morphological characteristics, the formulas were applied for a range of discharges between 400 and 700 m³/s at some sections along the reach. Figure (5.3) showed that Einstein’s formula was over-predicting in all sections and it showed multiple points of change in the slope at cross sections CS1, CS6-1, CS7 and CS9 depending on the water flow, whilst at sections CS6-4, CS11 and CS14, the formula curves were smoother. The Meyer-Peter and Schoklitsch (1943) formulas were always under-predicted. The Schoklitsch (1934) and van Rijn formulas alternated between the measurements being under and over depending on the characteristics of the cross section.
Table 5.3: Accumulative percentages of predicted bedload according to the ranges of discrepancy ratio.

<table>
<thead>
<tr>
<th>Formulas</th>
<th>0.75 ~ 1.25</th>
<th>0.67 ~ 1.5</th>
<th>½ ~ 2</th>
<th>¼ ~ 4</th>
<th>½ ~ 8</th>
<th>-25 ~ +25</th>
<th>-33 ~ +50</th>
<th>-50 ~ +100</th>
<th>-75 ~ +300</th>
<th>-87.5 ~ +700</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyer-Peter 1934</td>
<td>5.88</td>
<td>5.88</td>
<td>17.65</td>
<td>76.47</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Schoklitsch 1934</td>
<td>23.53</td>
<td>47.06</td>
<td>76.47</td>
<td>82.35</td>
<td>88.24</td>
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<tr>
<td>Chang et al. 1965</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Bagnold 1966</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>van Rijn 1984</td>
<td>23.53</td>
<td>41.18</td>
<td>52.94</td>
<td>94.12</td>
<td>94.12</td>
<td>0</td>
<td>0</td>
<td>0</td>
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</tr>
<tr>
<td>Wilcock 2001</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>17.65</td>
<td>47.06</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Julien 2002</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td>0</td>
</tr>
<tr>
<td>Wilcock-Crowe 2003</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>29.41</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Camenen-Larson 2005</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wong-Parker 2006</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>11.76</td>
<td>58.82</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Annual bedload quantities were estimated for the period 2009-2013 to be 36 thousand tons (minimum) in 2009 and ranged up to 50 thousand tons (maximum) in 2013 according to the van Rijn (1984) formula. The average annual transport rate for the period 2009-13 was 42.6 thousand tons.
Figure 5.3: Application of the bedload formulas for a range of discharges at different cross sections with the field measurements.
5.4. Total Load Transport Rate

The computations of suspended load transport rate ($G_s$) were conducted using the results of suspended load samples in table (3.4) according to the Edwards and Glysson (1998) procedure. The maximum load was 190.3 kg/s at CS2 associated with a water discharge of 459 m$^3$/s and the minimum load was 29.1 kg/s at CS3 associated with water discharge of 464.4 m$^3$/s as shown in Table (5.1).
The ratios of bedload to suspended load were also mentioned in Table (5.1). The maximum percentage ratio was 6.5% and the minimum percentage ratio was 0.63%. These ratios indicate that suspended load is the dominant mode of sediment transport along the study reach.

The suspended sediment discharges \((G_s)\) were combined with the bedload discharges \((G_b)\) to determine the total sediment loads \((G_t)\) at 16 cross sections along the northern reach as shown in Table (5.1). The maximum total load was 192.9 kg/s at CS2 whilst the minimum total load was 30.441 kg/s at CS3.

### 5.4.1. Sediment Rating Curve

Prediction of sediment load is of prime importance for river engineering and geomorphology (Recking, 2009). It affects the driving fluvial processing (Cook et al., 2013). The sediment rating curve is the common predictor used for estimating sediment transport as well as other prediction formulas. The sediment rating curve can be used for reconstructing long-term sediment transport records or compensating that which is missing in existing sediment transport records (Walling, 1977b; Asselman, 2000) as well as for using as a boundary condition in morphodynamic models.

The usual procedure to establish a sediment rating curve is by collecting sediment samples of a wide range of discharges at a certain cross section on the river reach, and then using one of the regression techniques to determine the best coefficients of the rating equation which is usually of power function form (Walling, 1977a; Fenn et al., 1985). Such a rating curve may be a good representation of sediment transport at the sampling location, but there is no guarantee that it will be as good as it is, where it to be used for other locations of different morphology.

Looking for a sediment rating curve that is spatially reliable along a river reach of complicated morphology is preferable for river engineering and modelling especially when there is evidence of sedimentation processes occurring along that river reach. Such a spatial sediment rating curve can be used as a sediment inflow or outflow boundary condition in morphodynamic models at any cross section along the reach. Instead of repeating sediment measurements at one location for a period of time, sediment samples will be collected at several locations along the river’s reach and this will represent the spatial variance in the topography, such as meandering, a riffle-pool, or an island, for that river reach on the condition that there is no tributary, distributary or regulator along that river reach.

The sediment rating curve was established previously for the Tigris River inside Baghdad using the suspended load measurements at three gauging stations (Sarai Baghdad, north of Baghdad and south of Baghdad) when none of the dams were constructed yet on the main river in Iraq (Geohydraulique, 1977, Al-Ansari et al., 1979). After the start of the Mosul Dam operation on the Tigris River in 1986, a modification was applied on the sediment rating curve (Khalaf, 1988) depending on more measurements in north of Baghdad gauging station. It should be mentioned however, that historical records were still used. All these rating curves took the form of power function. After adding the current measurements of the total load to the previous measurements, a new sediment rating curve (Eq. 5.1) was established with determination coefficient \((R^2)\) of 0.7963 as shown in Figure (5.4).
Figure 5.4: The total load rating curve along the northern reach of the Tigris River in Baghdad.

\[ G_t = 0.0002Q_w^{2.1312} \]

Where

- \( G_t \): total sediment discharge (kg/s)
- \( Q_w \): water discharge (m³/s)

The errors produced from applying the sediment rating curve at the same sampling locations are shown in Figure (5.5). A 26.66% of the estimated load have errors ranging between +10% and -10%, and an accumulated 46.66% of the estimations have errors ranging between -25% and +25%, and an accumulated 66.66% of the estimations have errors ranging between -50% and +50%, and an accumulated 80% of the estimations have errors ranging between -100% and +100%, then a 20% of the estimated load have errors greater than +100%.
5.4.2. Total Load Prediction Formulas

Brief descriptions for the approaches used for deriving the total load formulas are given below:

5.4.2.1. Unit stream power concept

The rate of work being done by a unit weight of water in transporting sediment must be directly related to the rate of work available to a unit weight of water (Yang, 1996). Yang emphasized the power available per unit weight of fluid to transport sediments.

a. Yang1973 formula (Yang, 1996)
b. Maddock1973 formula (Maddock, 1976)
c. Maddock1976 formula (Maddock, 1976)
d. Yang1979 formula (Yang, 1996)

5.4.2.2. Shear stress approach

The movement of bed material particles will start when the criteria of incipient motion is exceeded. So, shear stress near the bed will entrain the sediment particles into motion as long as the shear stress is greater than the critical shear stress of the particles. The following formulas which belong to this approach were used in this work:

a. Laursen1958 formula (Vanoni, 2006)
b. Chang-Simons-Richardson1965 formula (Yang, 1996)
c. Brownlie1981 formula (Julien, 2010)

5.4.2.3. Probabilistic approach

Probability concepts were introduced in bed sediment load prediction by the pioneer work of Einstein in 1942. The turbulent flow fluctuations are the driver for sediment entrainment
rather than the flow forces exerted on the particle. Both the entrainment and the deposition were expressed in probability terms (Yang, 1996).

a. Einstein1930 bedload function (Graf, 1971)
b. Garde-Dattatri1963 formula (Garde, 2006)
c. Colby1964 formula (Yang, 1996)
d. Graf-Acaroglu1968 formula (Graf, 1971)
e. Toffaleti1969 formula (Yang, 1996)
f. Simons-Li-Fullerton1981 formula (Julien, 2010)
g. Guo-Julien2004 formula (Julien, 2010)

5.4.2.4. Regression approach

Data driven models (regression, ANN) were used to explain the bedload transport process due to the limitations of defining this complex process into a precise formula (Talukdar et al., 2012). The following formulas were used within this approach:

a. Shen-Hung1972 formula (Yang, 1996)
c. Van Rijn1984 formula (Van Rijn, 1993)

5.4.2.5. Power Concept

This approach has developed from the concept that there is a relation between the available energy to the river with the rate of work done by the river to transport sediment (Yang, 1996). The following formula was used within this approach:

a. Bagnold1966 formula (Yang, 1996)
c. Ackers-White1973 formula (Yang, 1996)
d. Ackers-White1990 formula (Van Rijn, 1993)

5.4.2.6. Regime approach

This approach was developed based on the regime theory where the data used for establishing its relationships was taken from large stable irrigation canals (Vanoni, 2006).

a. Inglis-Lacey1968 formula

The mathematical equations of the twenty aforementioned bedload formulas were listed in Table (5.4).
Table 5.4: Total load prediction formulas applied to the Tigris River in Baghdad.

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Total load formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Einstein1958 bedload function</td>
<td>$g_t = \sum g_i (P_{0.0} - P_{0.1}) + 1$</td>
</tr>
<tr>
<td></td>
<td>$\phi = fn(\psi)$.</td>
</tr>
<tr>
<td></td>
<td>$\psi = \frac{(\rho_s - \rho)g}{\rho RS}$</td>
</tr>
<tr>
<td></td>
<td>$P_e = -\log \left(\frac{30.2D}{\Delta} \right)$</td>
</tr>
<tr>
<td>Laursen1958 formula **</td>
<td>$g_t = 0.01q_y \sum P_{0.0} \left( \frac{d_{m0}}{D} \right)^{7/6} \left( \frac{\tau_s}{\tau_m} - 1 \right) \left[ \frac{u_m}{\omega_x} \right]$</td>
</tr>
<tr>
<td></td>
<td>$\tau_s = \frac{\rho V^2 (d_{m0})^{1/3}}{D}$</td>
</tr>
<tr>
<td></td>
<td>$\tau_m = 0.039 (\gamma_s - \gamma) d_{m0}$</td>
</tr>
<tr>
<td>Garde-Dattatri1963 formula</td>
<td>$g_t = 16 \left( \frac{\tau_s}{(\gamma_s - \gamma) d_{m0}} \right)^{3/2} \gamma_s d_{m0}$</td>
</tr>
<tr>
<td>Colby1964 formula ***</td>
<td>$g_t = \frac{1}{f(\gamma_s - \gamma)} \left[ \frac{u_m}{\omega_x} \right]$</td>
</tr>
<tr>
<td>Chang-Simons-Richardson1965 formula ****</td>
<td>$g_t = K_t \left( \frac{V}{\gamma_s - \gamma} \right) + 1 + R_t \left( \frac{V}{\gamma_s - \gamma} \right)$</td>
</tr>
<tr>
<td></td>
<td>$K_t = fn(\psi_d)$</td>
</tr>
<tr>
<td></td>
<td>$\psi_d = \frac{(\rho_s - \rho)g}{\rho RS}$</td>
</tr>
<tr>
<td>Bagnold1966 formula</td>
<td>$g_t = \frac{\gamma}{(\gamma_s - \gamma)} V \left( \frac{c_s}{\omega_x} + 0.01 \frac{V}{\omega_x} \right)$</td>
</tr>
<tr>
<td></td>
<td>$\tan \alpha = \frac{d_{m0}}{(\gamma_s - \gamma) d_{m0}}$</td>
</tr>
<tr>
<td>Engelund-Hansen1967 formula</td>
<td>$g_t = 0.05 \rho_s \frac{V^2}{(\gamma_s - \gamma) d_{m0}^2}$</td>
</tr>
<tr>
<td>Graf-Acaroglu1968 formula</td>
<td>$C_y R \sqrt{(\frac{\gamma_s - \gamma) d_{m0}}{R} = 10.39 \left( \frac{(\gamma_s - \gamma) d_{m0}}{R} \right)^{2.52}}$</td>
</tr>
<tr>
<td>Inglis-Lacey1968 formula</td>
<td>$g_t = 0.562 \left( \frac{\omega_x}{\omega} \right)^{3/5} \frac{V^2}{\omega} \frac{V^3}{d_{m0}}$</td>
</tr>
<tr>
<td></td>
<td>$\omega = \frac{GM}{d_{m0}}$, $M = \left( \frac{2d_{m0}}{D} \right)^{1+\eta_{-0.7562}}$</td>
</tr>
<tr>
<td>Toffaleti1969 formula</td>
<td>$g_t = \sum g_i + g_{m0} + g_{m1} + g_{m2}$</td>
</tr>
<tr>
<td></td>
<td>$g_{m1} = M_1 \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}}$</td>
</tr>
<tr>
<td></td>
<td>$g_{m2} = M_2 \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}}$</td>
</tr>
<tr>
<td></td>
<td>$g_{m3} = M_3 \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}}$</td>
</tr>
<tr>
<td></td>
<td>$g_{m4} = M_4 \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{11.24} \right)^{2+\eta_{-0.7562}} \left( \frac{D}{2.5} \right)^{2+\eta_{-0.7562}}$</td>
</tr>
<tr>
<td></td>
<td>$Z_i = \frac{\omega_x V}{C_s RS}$, $Z_i = 1.5 \eta_{-0.7562}$, $\eta_{-0.7562} = 0.1198 + 0.00048 \Gamma$</td>
</tr>
<tr>
<td></td>
<td>$C_s = 260.67 - 0.66 \Gamma$</td>
</tr>
</tbody>
</table>

51
Table 5.4: Continued …

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Total load formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shen-Hung 1972 formula</td>
<td>$\frac{Y}{\omega} = \left(0.62 10^{-3} \frac{V}{d_{w}}\right)^{1.35}$</td>
</tr>
<tr>
<td>Ackers-White 1973 formula</td>
<td>$C_7 = 1 - 0.562 \log d_<em>, \quad \log C_7 = 2.86 \log d_</em> - (\log d_*)^2 - 3.53$</td>
</tr>
<tr>
<td>Yang 1973 formula</td>
<td>$\begin{align*} C_\omega &amp;= \frac{2.5}{\log \frac{V}{d_\omega}} + 0.66 \ 1.15 \leq \frac{V_{\omega} d_{\omega}}{d_<em>} &amp;&lt; 70 \quad V_{\omega} = 2.05 \end{align</em>}$</td>
</tr>
<tr>
<td>Maddock 1973 formula</td>
<td>$C_{SW} = 10^{2 \frac{PS}{\phi(d)}}$</td>
</tr>
<tr>
<td>Maddock 1976 formula</td>
<td>$C_{SW} = 10^{2 \frac{PS}{\phi(d)} - \frac{60(Gs-1)g d_{sw}^3}{\phi(d)^2 \frac{D}{d_<em>}}} \left(\frac{G(Gs-1)g d_{sw}^3}{d_</em>^3}\right)^{1.61}$</td>
</tr>
<tr>
<td>Yang formula (1970)</td>
<td>$\log C_{SW} = 5.165 - 0.153 \log \frac{V_{\omega} d_{\omega}}{d_<em>} - 0.297 \log \frac{V_{\omega} d_{\omega}}{d_</em>} + 1.78 - 0.363 \log \frac{V_{\omega} d_{\omega}}{d_<em>} - 0.48 \log \frac{V_{\omega} d_{\omega}}{d_</em>} \log \frac{PS}{\phi(d)}$</td>
</tr>
<tr>
<td>Simons-Li-Fullerton 1981 formula</td>
<td>$g_s = \left( D_{sw} C_s^{2.5} \right) \rho_s$</td>
</tr>
<tr>
<td>Brownlie 1981 formula</td>
<td>$C_{SW} = 7115 \frac{V - V_{sw}}{\sqrt{(Gs-1)g d_{sw}^3}}^{1.978} \left(\frac{R}{d_*}\right)^{3.101}$</td>
</tr>
<tr>
<td>Karim-Kennedy 1988a formula</td>
<td>$\begin{align*} C_{41} &amp;= \log \frac{V}{\sqrt{(Gs-1)g d_{sw}^3}} \ C_{42} &amp;= \log \frac{D}{d_<em>} \ C_{43} &amp;= \log \frac{V - V_{sw}}{\sqrt{(Gs-1)g d_{sw}^3}} \end{align</em>}$</td>
</tr>
<tr>
<td>Van Rijn 1984 formula</td>
<td>$\begin{align*} G_s &amp;= g_s + G_s \quad g_s = 0.053 \frac{\sqrt{1 - (G_s-1)g d_{sw}^3}}{d_<em>^{1.3}} + \rho_s \frac{FVDC_s}{d_</em>} \ T &amp;= \frac{\theta}{\theta_<em>} - 1 \quad \theta = \rho_s \frac{V}{C_s^3} \end{align</em>}$</td>
</tr>
</tbody>
</table>

$$C_s = 0.015 \frac{d_{sw}}{a} T^{1.5} \quad F = \left( \frac{F}{a} \right)^2 - \frac{(a/F)^2}{\left[1 - (a/F)^2 \right]^{(1.2 - Z)}}$$
Table 5.4: Continued …

<table>
<thead>
<tr>
<th>Author(s)</th>
<th>Total load formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ackers-White1990 formula</td>
<td>$C_Y \frac{D}{T(\frac{u_c}{V})} = C_1 \left(\frac{F_r e - 1}{C_3}\right)$</td>
</tr>
<tr>
<td></td>
<td>$F_r = \frac{u_c}{\sqrt{(G_t - 1) \rho g w}} \sqrt{\frac{V}{32 \log(10D/d_w)}}$</td>
</tr>
<tr>
<td></td>
<td>$C_1 = 1 - 0.562 \log d_s$</td>
</tr>
<tr>
<td></td>
<td>$C_3 = 0.23 + 0.14$</td>
</tr>
<tr>
<td></td>
<td>$C_1 = \frac{6.83}{d_s} + 1.67$</td>
</tr>
<tr>
<td></td>
<td>$g_s = g_s \left[1 + I_1 \ln \left(\frac{30D}{d_s} + I_2\right)\right]$</td>
</tr>
<tr>
<td></td>
<td>$g_t = g_s \left[1 + \left(\frac{u_c}{\omega}\right)^{1.1}\right]$ for $\frac{u_c}{\omega} &lt; 2$</td>
</tr>
<tr>
<td></td>
<td>$I_1 = 0.216 \frac{E^{Z-1}}{(1-E)^2} J_1(Z)$</td>
</tr>
<tr>
<td></td>
<td>$I_1 = 0.216 \frac{E^{Z-1}}{(1-E)^2} J_2(Z)$</td>
</tr>
<tr>
<td></td>
<td>$J_1(Z) = -\frac{nz^2}{\sin(nZ)} - F_1(Z)$</td>
</tr>
<tr>
<td></td>
<td>$J_2(Z) = -\frac{nz^2}{\sin(nZ)} \left[\pi \cot(nZ) - 1 - \frac{\pi^2}{Z} - \frac{Z}{6} \right] - F_2(Z)$</td>
</tr>
<tr>
<td></td>
<td>$F_1(Z) = \left(\frac{1-E}{E^{Z-1}} - Z \sum_{k=1}^{Z} -\frac{1}{k} \right)$</td>
</tr>
<tr>
<td></td>
<td>$F_2(Z) = F_1(Z) \left(\ln E + \frac{1}{Z-1} - Z \sum_{k=1}^{Z-k} \frac{-1}{Z-k} \left(J_1(Z-k) - J_1(Z-k-1)\right)\right)$</td>
</tr>
</tbody>
</table>

* Einstein’s procedure explained in detail in (Graf, 1971), see Figure 7.13 in (Graf, 1971) for the value of $\Phi=fn(\psi)$ and Figure 8.10 and 8.11 for the values of $I_1$ and $I_2$.

** see Figure 2.103 in (Vanoni, 2006) on pp. 122 for $\int \frac{u_c}{\omega} df$ value.

*** see Figure 6.12 pp. 172 and Figure 6.13 pp. 173 in (Yang, 1996) for the values of $k_1$, $k_2$, and $k_3$.

**** see Figure 5.10 pp. 135 and Figure 5.11 pp. 136 in (Yang, 1996) for the values of $I_1$ and $I_2$.

^^ see Figure 6.5 pp. 152 in (Yang, 1996) for the values of $f_{n1}$ and $f_{n2}$.

*+ see Figure 3 in (Maddock, 1976) for the value of $\phi(d)$

**+ see Table 11.1 pp. 270 in (Julien, 2010) for the values of $C_{S1}$, $C_{S2}$ and $C_{S3}$

***+ $g_b$ is the Einstein bedload

where

$g_s$: suspended sediment discharge per unit width (kg/s.m)
$g_b$: bedload sediment discharge per unit width (kg/s.m)
$g_t$: total sediment discharge per unit width (kg/s.m)
$\tau_s$: bed shear stress (N/m²)
$\tau_c$: critical shear stress for sediment particles (N/m²)
$g$: gravity acceleration (m/s²)
Chapter Five

\[ D = \text{depth of flow (m)} \]
\[ R = \text{hydraulic radius (m)} \]
\[ S = \text{water surface slope (m/m)} \]
\[ q = \text{water discharge per unit width (m}^3\text{/s.m)} \]
\[ q_c = \text{critical water unit discharge for sediment particles (m}^3\text{/s.m)} \]
\[ G_s = \text{specific gravity for sediment mixture} \]
\[ T = \text{water temperature °F} \]
\[ \rho = \text{water density (kg/m}^3\text{)} \]
\[ \rho_s = \text{sediment density (kg/m}^3\text{)} \]
\[ \gamma = \text{water specific weight (N/m}^3\text{)} \]
\[ \gamma_s = \text{sediment specific weight (N/m}^3\text{)} \]
\[ \sigma = \text{water specific weight (N/m}^3\text{)} \]
\[ \sigma_s = \text{sediment specific weight (N/m}^3\text{)} \]
\[ d_{50} = \text{median diameter of sediment (m)} \]
\[ d_{90} = \text{the particle diameter of 90% finer of the sediment weight (m)} \]
\[ d_s = \text{representative diameter of sediment (m)} \]
\[ i_b = \text{fraction of bed material} \]
\[ \beta = \text{proportional factor of sediment momentum diffusion to fluid momentum diffusion} \]
\[ \kappa = \text{von Karman constant (0.4)} \]
\[ \omega = \text{settling velocity of sediment particle (m/s)} \]
\[ v = \text{kinematic viscosity (m}^2\text{/s)} \]
\[ u^* = \text{bed shear velocity (m/s)} = \frac{\tau_w}{\gamma} \]

5.4.3. Application of Total Load Prediction Formulas

The results of applying the twenty-two total load formulas along sixteen cross sections within the study reach are displayed in Figure (5.6) showing a comparison with the measured total load discharges in the same cross sections. Three zones of different discrepancy ratios were added to the figure to explain the distribution of the compared results around the perfect agreement line.

Many formulas predicted considerably higher or lower sediment discharges, whilst the results of three formulas were closer to the measurements. These formulas were Colby\text{1964} formula, Brownlie\text{1981} formula and Guo-Julien\text{2004} formula with average discrepancy ratios of 1.17, 0.74 and 1.42 (errors 17%, -26% and 42%) respectively. Some of other predictors have limited results within the discrepancy zone 0.5 ~ 2 (error -50% ~ +100%), such as Garde-Dattatri, Graf-Acaroglu, Einstein, Toffaleti and Chang et al., but the accumulated percentage of their results didn’t reach 50% within the zone as shown in Table (5.5).
Sediment Transport in the Tigris River

Figure 5.6: Comparison of the predicted total load discharges using different total load formulas with the measured total load in the Tigris River.

To explain the scattering behaviour of the total load formulas at different cross sections along the study reach, they were applied at these cross sections for a range of discharges between 400 and 700 m$^3$/s. Figure (5.7) shows the results of the predictors at eight cross sections (CS1, CS3, CS6-1, CS6-4, CS7, CS9, CS11 and CS14) and it seems from the figure that the behaviour of most of the total load formulas is consistent, with the exception of the Guo-Julien formula at CS1 and CS6-4, where it has two distinct slopes, a steep slope for low flow until 500 m$^3$/s then a mild slope for higher flow. The variation of hydraulic radius versus water discharge is not consistent at all cross sections nor is the flow velocity which leads to such results for the more sensitive formulas. According to the results in Figure (5.7), it is hard to state that there is a unique prediction formula that can predict total sediment discharge along the whole study reach with stable magnitude of error. It is noteworthy to state that, even using those formulas where results are compatible with field measurements in regular cross sections, there is still no guarantee of getting the same compatibility at irregular cross sections (meanders, sand bars, etc.).

The annual total load quantity was estimated using the Colby1964 formula for the period between 2009 and 2013 and was 2.85 million tons in 2009 and ranged up to 3.87 million tons in 2013 compared to 2.47 million tons in 2009 ranging to 4.23 million tons in 2013 according to the total load rating curve. The average annual transport rate for the period 2009-13 was 3.21 million tons.
Table 5.5: Accumulative percentages of predicted total load according to the ranges of discrepancy ratio.

<table>
<thead>
<tr>
<th>Formulas</th>
<th>Accumulative of data percentage in each range of the discrepancy ratio or corresponding error percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.75 ~ 1.25</td>
</tr>
<tr>
<td></td>
<td>-25 ~ +25</td>
</tr>
<tr>
<td>Laursen 1958</td>
<td>5.88</td>
</tr>
<tr>
<td>Engelund-Hansen 1967</td>
<td>0</td>
</tr>
<tr>
<td>Garde-Dattatri 1963</td>
<td>23.53</td>
</tr>
<tr>
<td>Graf-Acaroglu 1968</td>
<td>17.65</td>
</tr>
<tr>
<td>Yang 1973</td>
<td>0</td>
</tr>
<tr>
<td>Yang 1979</td>
<td>0</td>
</tr>
<tr>
<td>Ackers-White 1973</td>
<td>0</td>
</tr>
<tr>
<td>Ackers-White 1990</td>
<td>0</td>
</tr>
<tr>
<td>Toffaleti 1969</td>
<td>5.88</td>
</tr>
<tr>
<td>Shen-Hung 1972</td>
<td>0</td>
</tr>
<tr>
<td>Maddock 1973</td>
<td>0</td>
</tr>
<tr>
<td>Maddock 1976</td>
<td>0</td>
</tr>
<tr>
<td>Bagnold 1966</td>
<td>0</td>
</tr>
<tr>
<td>Colby 1964</td>
<td>0</td>
</tr>
<tr>
<td>Chang et al. 1965</td>
<td>5.88</td>
</tr>
<tr>
<td>van Rijn 1984</td>
<td>0</td>
</tr>
<tr>
<td>Guo-Julien 2004</td>
<td>17.65</td>
</tr>
<tr>
<td>Einstein 1950</td>
<td>17.65</td>
</tr>
<tr>
<td>Simons et al. 1981</td>
<td>0</td>
</tr>
<tr>
<td>Brownlie 1981</td>
<td>29.41</td>
</tr>
<tr>
<td>Karim-Kennedy 1983</td>
<td>0</td>
</tr>
<tr>
<td>Inglis-Lacey 1968</td>
<td>5.88</td>
</tr>
</tbody>
</table>
Figure 5.7: Application of the total load formulas for a range of discharges at different cross sections with the values of the total load rating curve.
5.5. Spatial Distribution of Bedload and Suspended Load

The variance in the topography and morphology in the Tigris River was reflected in the spatial distribution of sediment loads and velocity field. Figure (5.8) shows the distribution of the measured suspended load concentrations and bedload discharges per unit width at sampling points along the study reach as well as bed shear stresses. The data series are not in the sequence of display in the figure and separation lines were used to specify cutting in the data series and also to specify the relative parts of each data series to a certain cross section. The following description for the spatial distribution of sediment loads was associated with the measured velocity distribution using ADCP at the sampling time.

CS1: The section is run section type. Some stagnation was on the extreme left side because it is located in the shade of bank deposition. Sediment concentration was higher towards the right side whilst bedload was oscillating across the section. Velocity was uniform except on the extreme left side.

CS2: Bank deposition was growing on the left side where flow velocity was low. Sediment concentration was the highest in all of the section whilst bedload was higher towards the right.

CS3: The section is located directly downstream of a large bank deposition. Its left side is hidden by the deposition and the velocity was higher on the right side. Both bedload and suspended concentration were low in the section and the bedload was little higher on the right side.

CS4: The section is located between a small island to the upstream and an acute meander to the downstream. The right side was stagnant because it is hidden behind the island and the velocity increased towards the left side. Both bedload and suspended concentrations were higher towards the left side. Eddies were noticed on the right side.

CS4-2: The section is pool type, deeper on the right side. It’s located at the centre of an acute meander. On the deeper side, velocity was not at its highest, higher velocities were closer to the centre. Sediment concentration was uniform across the section whilst bedload was higher in the centre.

CS5: The section is pool type, deeper on the right side. Velocity and bed shear were higher on the right side; even so, sediment concentration was higher in the centre of the section and also the bedload due to the effect of the secondary flow, since the section is within the downstream half of an acute meander.

CS6-1: The section is pool type; deeper on the left side. Even so, the velocity distributed uniformly across the section. Bedload was higher on the left side.

CS6-2: The section is the inner branch of a meander that is bisected by an island. The left side of the section was a trench produced by excavators, so the velocity was low by comparison with the right side. Both bedload and suspended concentration were relatively higher on the right side where the velocity was higher.

CS6-3: The section is the outer branch of the meander. The flow was turbulent on the most outer side that increased the sediment concentration even with lower velocity. Bedload was low at all.

CS6-4: The centre of the section was close to the tail of a large island, so the turbulence of the secondary flow at the confluence of the two branches was the reason behind the high sediment concentration.
Figure 5.8: The measurements of the suspended load concentrations and the bedload discharges with the calculated bed shear stresses at the sampling points along the Tigris River study reach.
CS7: The section is of a riffle type. The velocity on the left side was higher, so the sediment concentration was higher also. Whilst on the right side, although the bed shear was relatively high, bedload was low because the velocity was also low.

CS8: The section is pool type; deeper on the right side, but the velocity was neither high nor was the sediment concentration. The highest bedload was in the centre due to the effect of the secondary flow.

CS9: The main flow was on the right side whilst the left side was stagnant, so bedload was lower on the left.

CS10: The section is pool type, deeper on the left side. The bedload was lower due to higher bed shear. The velocity on the left side was not the higher, so that sediment concentration was not the higher also.

CS11: The section is the riffle type. Although the right side has the same depth as the left, but bedload was much higher on the left because the right side was stagnant and recently dredged.

CS13: The section is pool type, deeper on the right. The bedload was lower on the right due to higher bed shear, whilst bedload was higher in the centre due to the effect of secondary flow.

CS14: The section is pool type, deeper on the left. The bedload was lower on the left due to higher bed shear and higher flow velocity.

5.6. Adjusting the van Rijn Formula

In spite of the Colby (1964) formula was the closer to the measurements; however, this formula has not been considered in the morphodynamic model, which was used (see the next chapter). In addition, the Colby formula is not programmable. The preferable formula in the morphodynamic model is the van Rijn (1984) formula (Olsen, 2014) and the model gives the ability to adjust the parameters in the formula in case it is required. Trial and error iterations were conducted to determine the best values of the parameters involved within the formula, so it can give good agreement with the field measurements. The final form of the adjusted near bed sediment concentration equation in the van Rijn formula is shown in Equation (5.2). The agreement with the measurements for van Rijn and Colby formulas is shown in Figure (5.9).

\[
C_a = \frac{0.3}{a} \left( \frac{d_{50}}{a} \right)^{1.3} \left( \frac{t^2}{a} \right) \] ................................. (5.2)

All the variables in Eq. (5.2) were explained in Table (5.4). The produced errors from the adjusted van Rijn formula were of the same magnitudes of the errors of the Colby formula in average (see Fig. 5.9).
Figure 5.9: Comparison of the predicted total load discharges using the adjusted van Rijn formula and the Colby formula with the measured total load in the Tigris River.
Three-Dimensional Hydro-Morphological Modelling

Estimating erosion and sedimentation rates and their locations along the river reach are important for preparing future plans. These types of estimations require a good knowledge of and about the effective variables that control the processes. Discharge, flow velocity, bed material, sediment supply and channel slope are the major effective variables that control the erosion and sedimentation processes.

After the great development in computational methods and computer capabilities, simulation models became a powerful tool for simulating flow and consequent changes in river system using some of the well-known equations such as the Navier-Stokes equations which govern the relationships between the control variables in the treated phenomena. Moreover, many simulation models were amalgamated many models (such as the hydrodynamic model, the sediment transport model and the morphology model) in order to maintain integrity in the subject of erosion and sedimentation.

The reliability and the accuracy of the simulation models’ performances to reproduce the field conditions of real world problems, can be improved by calibration and validation processes to adjust to the parameters and the algorithms of the model and re-examining the results for different site conditions.

6.1. Theoretical Background

6.1.1. Water Flow Model

The principles of conservation of mass and momentum were used to derive the continuity equation (Eq. 6.1) and momentum equation (Eq. 6.2) (Navier-Stokes equations) for the instantaneous movement of the water-sediment mixture for incompressible and constant density flow. They are discretized with a finite volume approach and solved implicitly on non-staggered grid. SIMPLE (Semi-Implicit Method for Pressure-Linked Equations) algorithm is used for pressure correction (Olsen, 2014).

\[ \frac{\partial u_i}{\partial x_i} = 0 \]  \hspace{1cm} \text{(6.1)}

\[ \frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = \frac{1}{\rho} \frac{\partial}{\partial x_i} \left( -P \delta_{ij} - \rho \bar{u}_i \bar{u}_j \right) \]  \hspace{1cm} \text{(6.2)}

Where

- \( u_i \) is the flow velocity component in \( i^{th} \) direction
- \( t \) is time
- \( x_i \) is the \( i^{th} \) axis
- \( \rho \) is water density
- \( P \) is the pressure
- \( \delta_{ij} \) is the Kronecker delta

The term \( -\bar{u}_i \bar{u}_j \) is Reynolds stress term, modelled using eddy-viscosity concept (Eq. 6.3)
Chapter Six

\[ -\overline{u_i u_j} = \vartheta_T \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) - \frac{2}{3} k \delta_{ij} \] ................................. (6.3)

The eddy-viscosity is represented as
\[ \vartheta_T = C_\mu \frac{k^2}{\varepsilon} \] ................................. (6.4)

Where
\[ k \] is the turbulent kinetic energy
\[ \varepsilon \] is the dissipation rate turbulent energy
\[ C_\mu = 0.09 \] (Wu, 2008)

6.1.2. The \(k-\varepsilon\) turbulence model

The standard (linear) \(k-\varepsilon\) turbulence model was used in SSIIM which is modeled as shown in equations (6.5) and (6.6)

\[ \frac{\partial k}{\partial t} + u_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j} \left( \frac{\partial k}{\partial x_j} \right) + P_k - \varepsilon \] ................................. (6.5)

\[ \frac{\partial \varepsilon}{\partial t} + u_j \frac{\partial \varepsilon}{\partial x_j} = \frac{\partial}{\partial x_j} \left( \frac{\varepsilon}{\sigma_k} \frac{\partial k}{\partial x_j} \right) + \frac{\varepsilon}{k} C_{\varepsilon_1} P_k - C_{\varepsilon_2} \frac{\varepsilon^2}{k} \] ................................. (6.6)

Where
\[ P_k = \vartheta_T \frac{\partial u_i}{\partial x_j} \left( \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) \] ................................. (6.7)
\[ C_{\varepsilon_1} = 1.44, C_{\varepsilon_2} = 1.92, \sigma_k = 1.0 \text{ and } \sigma_\varepsilon = 1.3 \] (Wu, 2008)

6.1.3. Sediment transport

Sediment transport is governed by mass balance equation (Eq. 6.8).

\[ \frac{\partial c}{\partial t} + \vartheta_T \frac{\partial u_i}{\partial x_i} \frac{\partial c}{\partial x_i} + \omega \frac{\partial c}{\partial x_i} = \frac{\partial}{\partial x_i} \left( \Gamma_T \frac{\partial c}{\partial x_i} \right) \] ................................. (6.8)

Where
\[ \omega \] is the settling velocity of sediment particles
\[ \Gamma_T = \frac{\partial c}{\partial x_i} \] is the turbulent diffusivity of sediment
\[ Sc = 1.0 \] is Schmidt number

6.1.4. Wall laws

The empirical formula for rough walls derived by Schlichting in 1979 as shown in equation (6.9) was used as default in the model

\[ \frac{u}{u_*} = \frac{y}{k} \ln \left( \frac{2y}{\kappa} \right) \] ................................. (6.9)

Where
\[ y \] is the distance to the wall
\[ k \] is the roughness height
\[ \kappa \] is von Karman constant
\[ u_* \] is the bed shear velocity
6.2. Application

6.2.1. Preparation of SSIIM Model

The using of SSIIM model required preparing several issues before starting to run the model. These issues can categorize as following:

1. Geometry
   a. Geometry grid
   b. Bathymetry of the river
2. Hydrodynamic
   a. Initial water surface profile
   b. Boundary conditions
   c. Bed particles sizes and roughness
3. Morphology
   a. Bed sediment fractions
   b. Sediment boundary condition and transport
4. Calibration and validation
   a. Water levels
   b. Flow velocities
   c. Sediment concentrations

6.2.1.1. Model Geometry

A geometry grid was prepared for the river bed and lower banks. The size of the grid was chosen in a way to make a balance between the accuracy of the grid to represent the details of the complicated topography of the river as well as satisfying the stability requirements of the model solution and between computational time’s consumption. The horizontal size of the grid was 90000 cells and the length to width ratio of the grid cells was not larger than 3. The bathymetric and land surveys’ data was converted to geometry points and interpolated over the constructed grid as shown in Figure (6.1).

6.2.1.2. Hydrodynamic and Morphology

An initial water surface profile was considered from previous simulation for 1-D flow in the river for a discharge close to the average monthly flow (Ali et.al., 2012), then the model adjusted the water profile for the current hydraulic conditions using free surface algorithms. The upstream boundary condition was dependant on water inflow whilst the downstream boundary condition was dependant on specifying the water level corresponding to water discharge.

The results of the analysis for the bed material samples were used to prepare a distribution map for bed sediment fractions for all bed grid cells. The bed roughness was either calculated from Manning’s roughness coefficient \( n \) as shown in equation (6.10) or from \( d_{90} \) of particle size distribution and bedform height based on the sediment fractions map for bed grid cells as shown in equation (6.11).
Figure 6.1: The initial bed elevations (m.a.s.l.) of the Tigris River.
\[ k_s = (26n)^6 \] \hspace{1cm} (6.10)
\[ k_s = 3d_{90} + 1.1\Delta \left(1 - e^{\frac{2d_{90}}{\lambda}}\right) \] \hspace{1cm} (6.11)

Where

- \( n \) is Manning roughness coefficient
- \( d_{90} \) is the particle diameter where 90% of sediment are finer
- \( \Delta \) is bedform height which is calculated according to van Rijn (1984)
- \( \lambda \) is the bedform length = 7.3 * flow depth

Settling velocities for the sediment fractions in river bed were determined using Rubey settling velocity for the sizes between 300\( \mu \)m and 50\( \mu \)m, for smaller sizes, Stokes settling velocity was used as shown in Table (6.1). The total load rating curve (Eq. 5.1) was used to specify the sediment inflow.

<table>
<thead>
<tr>
<th>Particle size (mm)</th>
<th>Settling velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>0.041709</td>
</tr>
<tr>
<td>0.15</td>
<td>0.01748</td>
</tr>
<tr>
<td>0.075</td>
<td>0.005159</td>
</tr>
<tr>
<td>0.05</td>
<td>0.002344</td>
</tr>
<tr>
<td>0.0135</td>
<td>0.000134</td>
</tr>
<tr>
<td>0.008</td>
<td>4.71E-05</td>
</tr>
<tr>
<td>0.006</td>
<td>2.65E-05</td>
</tr>
<tr>
<td>0.004125</td>
<td>1.25E-05</td>
</tr>
<tr>
<td>0.0015</td>
<td>1.66E-06</td>
</tr>
</tbody>
</table>

### 6.2.2. Calibration and Validation

Calibration is an important process for any type of models that can improve the reliability of the model to represent the processes in the phenomena that is simulated. Calibration process includes the adjustment of some or all of the parameters those assumed in the govern equations by comparing the simulation results to the field or the lab measurements. By evaluating the size of the differences (errors) between results and measurements, one can accept or refuse the model’s results, the minimum differences is indicative of a higher quality of simulation. Another set of measurements needs to be used to validate the model using the same calibrated parameters in previous step.

In morphodynamic models, calibration can include adjusting several parameters, those assumed initially according to the experiences in the application field. The calibration parameters considered for this work are listed below with their selected values:

1. Bed roughness (\( n=0.02, n=0.025, n=0.0286, n=0.04, 3d_{90}/\lambda \)bedform)
2. Water surface profile algorithm (back water, pressure field)
3. Turbulence model (standard k-\( \varepsilon \), local k-\( \varepsilon \))
4. Simulation time step (6hr, 1hr, ½hr)
5. Discretization scheme (pow, sou)
6. No. of internal iterations (100, 150, 1000)
7. Sediment formula (suspended load, total load)
8. Active layer thickness (0.1m, 0.2m, 1m)
All field measurements were conducted during a range of water flows between 450 and 645 m$^3$/s. Furthermore, the average monthly discharge during the last decade was 522 m$^3$/s. So, the flow 530 m$^3$/s is the closest discharge to the monthly average and was selected to be used for model calibration and a higher discharge, 645 m$^3$/s, was selected for the validation of the model.

Calibration processes were repeated 39 times for different combinations of calibration parameters. Of these, 28 trials were designed for hydrodynamic part or parameters; such as bed roughness, a water surface algorithm, a turbulence model, the computational time step, a discretization scheme and number of internal iterations, whilst the remaining 11 trials were used for the sediment concentration part; such as the entrainment algorithm, the sediment formula and the active layer thickness.

Root mean square error (RMSE) was used as indication of the differences between the simulation results and measurements. For the first group of parameters, RMSE were calculated for water levels and velocity profiles, whilst for the second group, RMSE was calculated for sediment concentration profiles. The field measurements for CS9 and CS13 were used in the calibration processes because they were measured at the same flow rate, 530 m$^3$/s. The RMSE for the whole calibration trials was listed briefly in tables (6.2) and (6.3).

### Table 6.2: RMSE values for water level (m) and velocity profile (m/s) for calibration of hydrodynamic parameters

<table>
<thead>
<tr>
<th>WSP algorithm</th>
<th>WL</th>
<th>velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>BW</td>
<td>0.149</td>
<td>0.108</td>
</tr>
<tr>
<td>PF</td>
<td>0.104</td>
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</table>

<table>
<thead>
<tr>
<th>Turbulence model</th>
<th>WL</th>
<th>velocity</th>
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<tr>
<td>local k-$\varepsilon$</td>
<td>0.102</td>
<td>0.106</td>
</tr>
<tr>
<td>standard k-$\varepsilon$</td>
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<td>0.105</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Discretization scheme</th>
<th>WL</th>
<th>velocity</th>
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</thead>
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<tr>
<td>POW</td>
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<td>0.105</td>
</tr>
<tr>
<td>SOU</td>
<td>0.104</td>
<td>0.108</td>
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<table>
<thead>
<tr>
<th>Time step</th>
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<th>velocity</th>
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<tr>
<td>6</td>
<td>0.104</td>
<td>0.105</td>
</tr>
<tr>
<td>1</td>
<td>0.104</td>
<td>0.105</td>
</tr>
<tr>
<td>$\frac{1}{4}$</td>
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<td>0.105</td>
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<table>
<thead>
<tr>
<th>Roughness</th>
<th>WL</th>
<th>velocity</th>
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</thead>
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<tr>
<td>$n=0.02$</td>
<td>0.104</td>
<td>0.105</td>
</tr>
<tr>
<td>$n=0.025$</td>
<td>0.102</td>
<td>0.116</td>
</tr>
<tr>
<td>$n=0.0286$</td>
<td>0.140</td>
<td>0.127</td>
</tr>
<tr>
<td>$n=0.04$</td>
<td>0.190</td>
<td>0.166</td>
</tr>
<tr>
<td>$3d_{90}$</td>
<td>0.104</td>
<td>0.102</td>
</tr>
<tr>
<td>$3d_{90}+\text{bedform}$</td>
<td>0.104</td>
<td>0.102</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Internal iterations</th>
<th>WL</th>
<th>velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.104</td>
<td>0.105</td>
</tr>
<tr>
<td>150</td>
<td>0.104</td>
<td>0.107</td>
</tr>
<tr>
<td>1000</td>
<td>0.104</td>
<td>0.102</td>
</tr>
</tbody>
</table>
An overview of the calibration results in table (6.2) shows that most of the results are approximately identical. However, some of these parameters and algorithms were given smaller RMSE and they have to be considered for the validation process. Explanation for the importance of these parameters and their effects can be discussed in following points:

1. Water surface profile algorithm, pressure field, gave better results than the backwater algorithm for both water levels and velocity profiles.

2. Both turbulence models, local k-ε and standard k-ε, gave approximately similar results. The local k-ε model is based on the water velocity and its algorithm was not explained clearly in the documents of the SSIIM model, so the standard k-ε is preferred.

3. Although the second order upwind (SOU) scheme has an advantage over the first order power law (POW) scheme because it takes into consideration the first order derivative of the partial differential equation, whilst the POW scheme ignores it, the POW scheme gave less RMSE for the velocity than the SOU scheme. However, the SOU will be considered in the following steps because of its order of accuracy and to reduce the possible false diffusion in the convection term, as well as the difference between RMSE for both of them is negligible.

4. There is no apparent effect of the time step size on the calibration results, which may return to the steady state flow that was considered in the calibration. However, as soon as the sediment transport process starts and erosion and sedimentation processes develop, the importance of the time step size will arise due to developing the changes in bed and the associated changes in water depth and velocity vector. The smaller the time step is the more stable the solution and the longer the time of computations are. A one-hour time step will be considered in the next steps unless stability requirements impose smaller one.

5. Roughness is one of the most important parameters in the set. Many of results’ magnitudes and accuracies depend on the bed roughness value; such as the water surface profile and velocity profile. Bed roughness either computed from Manning’s roughness coefficient as in Eq. (6.10) or from bed sediment grain size and bed form height as in Eq. (6.11). The later one gave a smaller RMSE, so it will be used in the next steps.

6. At each time step, there are number of internal iterations for resolving the system of equations to reduce the residuals of the variables. Increasing number of internal iterations is required to develop the convergence in the solution; on the other hand, it will increase the computational time effectively if the grid has huge number of cells. Internal iterations between 200 and 500 were considered depending on stability requirements.

The other set of parameters are those that affect the sediment concentration profile (table 6.3) these were given a wider variance of RMSE values. Discussion of their values and the corresponding RMSE are outlined in the following points:

1. An algorithm that can invoke the SSIIM model to convert the computed sediment concentrations to sediment entrainment rates for the bed cells is by using the sediment formula (Olsen, 2014). Using this algorithm gave RMSE significantly lower values than those computed without the entrainment algorithm.

2. Applying sediment load according to the sediment rating curve improved the concentration profile and reduced the RMSE. Otherwise, the computation of the concentration profile will be based only on the erosion rate from the bed.
3. The Van Rijn bedload formula (van Rijn, 1993) and suspended load formula (van Rijn, 1984) can be used separately or combined in the SSIIM model. Combining them together will compute the total sediment load. The RMSE relating to total load computation was less than the one for suspended load alone.

4. The river bed can be simulated by two layers, active and inactive. The active layer is the top one and it has a constant thickness during simulation time. The active layer interacts with the flow and any erosion or sedimentation will compensate to/from the inactive layer. Zhang et al. (2015) found that the thickness of the active layer has to be considered as a calibration parameter. It often equated to half of the average bedform height (Mosselman, 2012) or even more up to 1.5 times the sand dune height (Tuijnder, 2010). The calibration processes showed that it was the active layer thickness 0.1m that gave smaller RMSE.

| Table 6.3: RMSE values for sediment concentration profile (mg/l) for calibration of sediment parameters |
|-------------------------------------------------|----------------|
| **entrainment** | **concentration** |
| yes | 0.094 |
| no | 3.104 |
| **Sediment load** | 0.057 |
| yes | 0.057 |
| no | 0.094 |
| **Sediment formula** | 0.084 |
| Suspended load | 0.057 |
| Total load | 0.057 |
| **Active layer thickness** | 0.037 |
| 0.1 | 0.037 |
| 0.2 | 0.052 |
| 1.0 | 0.057 |

As a result of the calibration processes, the easting and northing components of the calibrated velocity profiles at CS9 and CS13 are shown together with the measured profiles in Figures (6.2) and (6.3) respectively. The calibrated sediment concentration profiles are shown in Figure (6.4) together with the measured concentration profiles.

The simulated depth-averaged velocity distribution of the Tigris River is shown in Figure (6.5), where the velocity ranged between 0.07 m/s at stagnant locations close to the banks and 1.35 m/s at some locations along the deep route. The velocity field is affected by the precision of the geometry shape, where dispersion can be noted at certain locations which can be attributed to the discretization in the bathymetric survey and to the flow disturbance due to dredging operations.

The measurements of ADCP for the depth-averaged velocity profiles were compared with the simulated velocity field and it was found that the flow directions were compatible in the most of the measurements as shown in Figure (6.6). However, a few incompatibilities were found at some locations of circulation and they might be attributed to the false diffusion related to the discretization scheme (Dorfmann and Knoblauch, 2009).
Figure 6.2: The calibrated and the measured velocity profiles of CS9.
Figure 6.3: The calibrated and the measured velocity profiles of CS13.
Figure 6.4: The calibrated and the measured sediment concentration profiles of CS9 and CS13.
Figure 6.5: The depth-averaged velocity distribution (m/s) in the Tigris River after the calibration for water discharge 530 m$^3$/s.
Figure 6.6: The simulated velocity vectors on the water surface (black arrows) and on the riverbed (green arrows) with the measurements of the depth-averaged velocity by ADCP (blue lines).

The validation process was applied to a water flow of 645 m$^3$/s using the chosen set of parameters and algorithms from the calibration step. A new water surface profile was computed for the new water flow. The inflow of sediment discharge was set according to the water flow. To examine the model validation, the new simulation results were compared with another set of
measured which included water levels, velocity profiles and sediment concentration profiles of CS7 and CS8. The RMSEs were determined for the validation step as shown in table (6.4).

Table 6.4: RMSE values for water surface profile (m), velocity profile (m/s) and sediment concentration profile (mg/l) for validation process

<table>
<thead>
<tr>
<th>Validation</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water level</td>
<td>0.046</td>
</tr>
<tr>
<td>Velocity</td>
<td>0.101</td>
</tr>
<tr>
<td>Concentration</td>
<td>0.085</td>
</tr>
</tbody>
</table>

The results in table (6.4) show that the values of RMSE for the validation process were of the same magnitudes for calibration processes. Accordingly, the model can be considered valid for simulating water surface, velocity field and sediment concentrations for Tigris River reach.

### 6.3. Future Prediction

To predict the future changes in any river system accurately, knowledge about the future changes in the controlling variables; such as water flow, extra sources of sediment supply, human activities, and so on, must be satisfied. This will guide the model user to adjust the parameters at the appropriate simulation time to keep the model results on track. Otherwise, the future prediction will be limited to the present knowledge of the river conditions. Significant changes can occur to the controlling variables which should be taken into consideration; otherwise unrealistic results might be produced by the model.

After satisfying calibration and validation processes, the SSIIM model can be used for predicting the future changes of river bed topography and the velocity field for the northern reach of the Tigris River in Baghdad. The aims of predicting future changes of river topography and morphology are to set guidelines for:

1. Predicting locations of future sedimentations and the expected rate of deposition, which can help in drawing future dredging plans and deciding whether to maintain the functionality of the water intakes or to improve the flooding capacity of the river.
2. Finding out locations of future erosion which can help in warning of the possibilities of failure in river banks and in bridges piers.
3. Changes in navigation routes can be found, which can help to warn of expected shallow routes.
4. Changes in river ecosystem and biodiversity can be predicted.

According to the available data and information about the study reach, some assumptions have to be set for simulating the future predictions as follows:

1. River flow was assumed steady and equal to the discharge that was used in the calibration (530 m³/s).
2. The corresponding inflow of sediment discharge was assumed constant with the same size fractions.
3. The thickness of active and inactive bed layers were assumed to be 0.1m and 10m thick respectively along the study reach and of the same sediment fractions. The thickness of the
inactive layer was considered according to the thickness of depositions found in a previous study (Geohydraulique, 1977).

4. Bridge piers will not be considered in the river geometry to avoid the disturbance in the flow field, as well as computations of local souring around bridge piers, requiring a finer grid which is beyond the scope of current work.

Using a constant water discharge at 530 m$^3$/s, which is close to the average monthly discharge for the last 13 years, simulations of future predictions can help in establishing a comparison between the locations of deposition and their patterns from the simulation results of bed changes and deposition locations those are recognized from the available aerial images in Google Earth and from other sources for the study reach during the period of 2002 to 2012. This comparison has a degree of importance because of the continuity of the dredging operations along the river in Baghdad that makes continuous monitoring for the river bed changes hard to conduct by researchers.

Long term prediction of three dimensional morphodynamic models for a large scale study reach would consume long computational time and monitoring efforts due to high potential computations and instability problems. So, the prediction period will be limited to 14 months.

6.4. Results and Discussion

The disequilibrium behaviour is the usual behaviour in a river that is responding to the local or global hydrologic changes, such as climate changes or human activities (regulation and damming). Such a river tends to adjust the channel dimensions and slope continuously by reducing channel width and increasing flow depth since both discharge and sediment load are decreased (Hickin, 1995).

The sediment capacity of the Tigris River inside Baghdad is reduced, where the average cross sectional velocity was around 0.7 m/s. Also, the water surface slope is within the range of 6.5 to 6.8 cm/km, corresponding to the discharges of the period (2009-2013), while it is steeper (14 cm/km) between Samarra and Baghdad as shown in the document of MoWR (2012).

6.4.1. Bed Changes

Since the flow discharges and sediment loads were reduced by the headwater regulation system, so the Tigris River in Baghdad tends to deposit part of the eroded sediment upfront on the shallow part of the cross section having lower velocity and, on the other hand, it deepens the incised route to fit its current hydrologic condition leaving the former wide section as a floodplain for the newer river as shown in Figure (6.7). Higher depositions were distributed between circulation zones and meanders’ inner banks, where the flow velocity is low.

The results of future predictions for the changes in the river bed showed that the Tigris River behaved as an under-fit river. The exception in the Tigris River inside Baghdad is that the river is confined by protected banks, where the bases of the banks were filled by stones, so the margin of sinuosity is limited.
Figure 6.7: The predicted bed elevations (m.a.s.l.) of the Tigris River.
The depth of the incision at some locations seemed to be exaggerated, where it reached the whole bed sediment layer as shown in Figure (6.8) around station 2000m and 10000m. This is attributed to the high depth of the bed sediment layer which is composed of easily erodible material. So, investigations about the real depth of the loose sediment layer and the characteristics of the strata underneath are required. Figure (6.8) shows the changes in thalweg line elevations. It gives an indication of the potential threats of the river banks’ collapse since erosion is taking place below the protection base level especially in peaks of the meanders as shown in Figure (6.9). Small parts of the protection had already collapsed on the outer bank meander at CS13 (Fig. 6.9.C), which prompted the MoWR to drive in some sheet piles to stabilize the river bank. The high erosion in the beginning of the reach in Figure (6.8) may attribute to some bugs in the SSIIM model, where similar erosion has been noticed in some tutorials of the model.

![Figure 6.8](image1.png)

**Figure 6.8:** The initial and the predicted elevations of the thalweg line of the Tigris River.

![Figure 6.9](image2.png)

**Figure 6.9:** The predicted locations of the potential threats of river banks collapse (points A, B and C in Fig. 6.7).

Some depositions took shape of longitudinal barriers parallel to the flow direction whilst the deposition behind the barriers towards the banks continues to develop as shown in Figure (6.10).
The front sides of the barriers with the relatively faster flow were built from the coarser sediments whilst in the back sides, the finer sediment was depositing.

![Figure 6.10: Barrier depositions parallel to the flow direction (points D and E in Fig. 6.7).](image)

The results of the model showed that the net deposition/erosion rate was 67.44 kg/s in average along the study reach and the total deposition quantity was 2.12 million tons annually.

6.4.2. Velocity Field

In general, the range of the predicted depth-averaged velocity increased, where it reached up to 1.49 m/s as shown in Figure (6.11). Specifically, the velocity increased in the incised route zone and decreased in the shallow part of the section. The flow pattern became oriented and smoother compared with the pre-prediction case.

6.4.3. Depositions in Reality

Figure (6.12) showed part of the depositions along the Tigris River in Baghdad. Google Earth photos captured on different dates, showed the size of the depositions relative to river width. The associated water discharges for the dates of the photos were added to the figure to give an indication of the water levels when the depositions are uncovered. Considering the predicted velocity distribution in the figure, it can be concluded that re-deposition at same sites is possible. So the prediction from the model can be close to the reality.
Figure 6.11: The distribution of the predicted depth-averaged velocity (m/s) in the Tigris River for water discharge 530 m$^3$/s.
Figure 6.12: The predicted sites of depositions in the SSIIM model and the depositions in reality including the discharges of the river.
6.4.4. Development of Islands

Two islands are located within the study reach. The smaller one is located upstream of an acute meander (point D in Fig. 6.7). This island is undergoing dredging. As shown in Figure (6.10.D), depositions were built up touching the island and extended downstream forming a pond in the shade of the island. This pond is a result of depositing more fine sediment, which means developing the island again. The second island (Kura’at Island), which is the bigger, is at a bisecting meander. The origin of this island can be chute cut-off then it developed later to an island. Future predictions show that the front of the island is eroding whilst the width and the tail length of the island are increasing as shown in Figure (6.13). The growth rate is faster towards the outer side of the meander and it gives an indication of possible contact with the outer bank in the future, since the right branch of the meander is getting deeper and might develop to accommodate most of the flow over the whole cross section.

6.4.5. Navigation Routes

Figure (6.14) show the predicted water depths along the Tigris River. Water depth was within the range of 2 to 4 m except for the incised route zone where the depth reached up to 15m. So navigation for small boats is still possible even in the zones close to the banks where the draught of small boats is less than about 1.2m. Caution should be taken seriously by passenger boats or ferries when navigating close to the banks. Port areas are either to be kept clean from depositions by dredging or should be extend into deeper water.
Figure 6.13: The predicted development of the Kura’at Island.
Figure 6.14: The predicted water depths (m) in the Tigris River of water discharge 530 m$^3$/s.
Conclusions and Recommendations

7.1. Conclusions

The results of the implementation of the 1-D flow model on the Tigris River reach inside Baghdad for determining the flood capacity led to the following conclusions:

1. The variations in bed levels were less in the bathymetry of 2008 compared with those of 1976 and 1991. However, the average slope of the riverbed was steeper in 2008 than the previous surveys.
2. The calibration of the 1-D model showed very good agreement with the observations of the Sarai Baghdad gauging station also with the lower 15 km of the river reach.
3. The calculated water surface slopes varied from 6.03 to 6.84 cm/km during low discharges. For discharges between 2500 and 2700 m$^3$/s, the slopes were between 8.59 and 8.96 cm/km respectively, whilst it could reach 10 cm/km for the discharges between 3500 and 4000 m$^3$/s.
4. Inundation could take place along approximately 9 km of the reach with discharges greater than 3500 m$^3$/s.

The results of the 1-D flow model for evaluating the effect of the dredging plans on improving the flood capacity led to the following conclusions:

1. The differences between the dredging programs or plans are concentrated on the upper banks depositions which were not reflected on the water surface profiles for low discharges where they were almost identical for all dredging plans.
2. The application of the dredging plans can recover the water levels at Sarai Baghdad to the values of the 1970s for the discharges range from 1100 to 1500 m$^3$/s, but not for higher discharges.
3. The dredging plans can decrease the Sarai Baghdad water levels for less than the values of 1980s for the range discharges 2500 to 2700 m$^3$/s.
4. The expected water levels at Sarai Baghdad will be closer to the records during the last three decades for low discharges.
5. Caution should be taken to keep the water intakes functional due to the expected drop in water levels, especially along the northern reach of the river.
6. Dredging plans along the dredged part of the river reach will be associated with reductions in longitudinal velocities and more sedimentation is expected to take place at these locations.
7. The estimated quantities of sediment to be removed by OERDW plan was about 3443000 m$^3$. Fewer quantities of sediment were estimated to be removed by the first additional plan (1330000 m$^3$), and 610000 m$^3$ were the estimated sediment to be removed by the second additional plan.
The results of the analysis of bed sediment samples of the Tigris River led to the following conclusions:
1. The size of the bed sediment relatively decreased when compared with previous studies, due to reduction of flow competence and also the construction of a dam on Adhaim tributary in 1999.
2. The bed levels had increased compared to previous surveys. This is believed to be due to the decrease in the capacity of the river to transport sediment.
3. Dredging operations and obstacles (e.g. fallen bridges and islands) disturbed the flow of the river in several sites and consequently disturbed the characteristics of the sediment in the vicinity of such areas.

The results of the analysis of bedload samples of the Tigris River led to the following conclusions:
1. The implemented regulation scheme on the Tigris River has limited the sources of sediment supplying; it has also decreased the average water flow to 44% compared to previous period.
2. Limited local sources can supply fine sediment to the river reach during rainy seasons or high flow, whilst the main source of transported sediment in the Tigris River in Baghdad is the erodible bed material.
3. It was difficult to find a representative bedload formula due to the complicated geometry of the river.
4. The closest bedload prediction formula was van Rijn formula (1984).
5. Annual bedload quantities were estimated for the period 2009-2013 to be 36 thousand tons (minimum) in 2009 and ranged to 50 thousand tons (maximum) in 2013 according to the van Rijn (1984) formula. The average annual transport rate for the period 2009-13 was 42.6 thousand tons.

The results of the analysis of suspended load samples and total load calculations of the Tigris River led to the following conclusions:
1. The ratios of the measured bedload to suspended load indicated that the suspended load is the dominant mode in the total load with a minimum percentage of 93.5%.
2. The total load was ranged from 29.1 to 190.3 kg/s.
3. The total load prediction formula closest to the field measurements was Colby formula (1964).
4. Scattering in the results of total load predictors can be attributed to the spatial variance in topography whilst it has less effect on the sediment rating curve.
5. The associated errors from using the total load rating curve are within reassuring levels and less than the errors produced from most of the other twenty two total load predictors.
6. Using the spatial total load rating curve is preferable for morphologically complicated rivers.
7. The estimated annual transported quantities of total load were 2.47 (minimum) and 4.23 (maximum) million tons for 2009 and 2013 respectively. The average annual transport rate for the period 2009-13 was 3.21 million tons.
The results of implementation of the 3-D flow model on the Tigris River reach inside Baghdad for predicting the future changes in the riverbed led to the following conclusions:

1. The calibration of the SSIIM model succeeded in reproducing the water levels, velocity profiles and sediment concentration profiles at the cross sections used for this purpose. The validation process for other cross sections produced results of the same order of errors.
2. The SSIIM model has been found valid to be used for predicting future changes in the riverbed.
3. The future predictions showed the Tigris River behaved like an underfit river tending to adjust the channel dimensions and the slope by deepening an incised route that is more fit with its current discharges (water and sediment), and leaving the former wide section as a floodplain of the newer river.
4. Higher depositions were distributed between circulation zones and meanders’ inner banks, where the flow velocity is low.
5. The erosion along the thalweg line gave an indication of the potential threats of the river banks’ collapse and the bridge piers’ instability.
6. Some deposition took shape of longitudinal barriers, forming settling ponds for finer sediment to deposit behind the barriers towards the banks.
7. The net deposition/erosion rate was 67.44 kg/s as an average along the study reach and the total deposition quantity was 2.12 million tons annually.
8. The locations of depositions are compatible with those of the river in the real world.
9. The re-deposition in the model at the same real sites along the river indicated that sedimentation processes will continue in the river for the current hydrologic conditions and deepening of the incised route will also continue until the cross section of the river is adapted or the erosion reaches a stiffer bed layer that cannot be easily eroded.
10. The pond formed by deposition in the shade of the small island, is working on re-build the dredged part again.
11. The width and the tail length of the larger island (Kura’at Island) increased. A faster rate growth on the left side of the island may lead in the future to the connecting of the island with the near bank, and the right branch of the meander might develop to accommodate most of the flow over the whole cross section.
12. Small boats of small draught have the possibility to port close to the river banks. Areas around ports have to be kept clean from sediment to let passenger boats or ferries porting easily. Otherwise ports have to be extended to deeper water.

7.2. Recommendations for Future Works
According to the previous conclusions related to the Tigris River, the following issues are recommended to be considered as future work:

1. Estimating the flood capacity of the Tigris River and determining the possible inundation of the nearby areas using the dynamic wave routing (unsteady flow) considering the predicted flood wave due to the proposed scenario of the Mosul Dam’s failure whether using the current situation of the riverbed topography or the predicted topography of the near future.
2. Establishing a spatial bedload rating curve for the Tigris River in Baghdad.
3. Conducting a new bathymetric survey for the intact parts of the river reach to compare the real changes of the riverbed with the results of the SSIIM model, and proposing different set
of calibration parameters (such as particle settling velocity, characteristics of riverbed strata, sediment transport formula) in case of getting significant differences.

4. Simulating other scenarios of water flow of lower discharges to find out the predicted changes in the riverbed for more drought hydrologic conditions.

5. Extending the current work related to the estimation of the bedload and the total load for the southern reach of the Tigris River.

6. Extending the current work related to the future predictions of the riverbed changes for the southern reach of the Tigris River in Baghdad.

7. Studying the potential threats of the stability of the bridge piers and the protected banks due to the high predicted erosion along the thalweg line of the river.

8. Proposing using the technique of current deflection walls instead of the dredging and studying its efficiency of re-arranging the flow field to erode the bed sediment in applicable locations.


Atsuhiro, Y., Shintaku, S., Ejima, K., Fukami, K., and Kanazawa, H., 2009. DEVELOPMENT OF A SEDIMENT DISCHARGE MEASUREMENT SYSTEM WITH ADCP, 10th International Conference on Fluid Control, Measurements, and Visualization, August 17-21, Moscow, Russia.


Bridge, J.S. and Best, J.L., 1988. Flow, sediment transport and bedform dynamics over the transition from dunes to upper-stage plane beds: implication for the formation of planar laminae, Sedimentology 35: 753-763.


Ministry of Water Resources (MoWR), 2012. Distances, wave travel times and water surface slopes between gauging stations along Tigris and Euphrates Rivers. Personal communications with the staff of the National Center for Water Resources Management, Baghdad.


Wilcock, P.R., 2001. TOWARD A PRACTICAL METHOD FOR ESTIMATING SEDIMENT-TRANSPORT RATES IN GRAVEL-BED RIVERS. Earth Surface Processes and Landforms 26, 1395–1408.


World Meteorological Organization (WMO), 2003. MANUAL ON SEDIMENT MANAGEMENT AND MEASUREMENT. OPERATIONAL HYDROLOGY REPORT No. 47, WMO-No. 948.


Morphology of Tigris River within Baghdad City

Morphology of Tigris River within Baghdad City

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Abstract. In recent years, substantial changes have occurred in the morphology of the River Tigris within Baghdad City. Although huge volumes of sediment are being trapped in recently constructed headwater reservoirs, the number of islands in the Tigris at Baghdad is increasing. The debris of bridges destroyed in the wars of 1991 and 2003 and their subsequent reconstruction have enhanced the development of these islands. As a consequence the ability of the river to carry the peaks of flood waters has been reduced. This has led to potential increase of flooding in parts of the city.

The bed of the River Tigris has been surveyed on three occasions (1976, 1991, and 2008). The most recent survey was conducted by the Ministry of Water Resources, extended 49 km from the Al-Muthana Bridge north Baghdad to the confluence with the Diyala River south Baghdad. It yielded cross-section profiles at 250 m intervals. The data are used to predict the maximum flood capacity for the river using the one-dimensional hydraulic model for steady flow “HEC-RAS” modeling. Calibration of the model was carried out using field measurements for water levels along the last 15 km of the reach and the last 10 yr of observation at the Sarai Baghdad gauging station.

The model showed a significant predicted reduction in the current river capacity below that which the river had carried during the floods of 1971 and 1988. The three surveys conducted on the same reach of the Tigris indicated that the ability of the river to transport water has decreased.

1 Introduction

The River Tigris is 1850 km in length, rising in the Taunus Mountains of Eastern Turkey. The river flows for about 400 km through Turkey before entering Iraq. The total length of the river in Iraq is 1418 km. It drains an area of 473,103 km² which is shared by Turkey, Syria and Iraq, as shown in Fig. 1. About 58 % of the basin lies in Iraq, and no major tributary joins the Tigris south of Baghdad (Al-Ansari et al., 1986, 1987), but several canals draw water from the Tigris in this region for irrigation purposes. For this reason the mean annual daily flow of the river falls below the discharge at Baghdad (1140 m³ s⁻¹) at Kut and Amara, cities to the south.

The average annual flow discharge of the Tigris is 21.2 km³ yr⁻¹ (672 m³ s⁻¹) when it enters Iraq. Its main tributaries contribute a further 24.78 km³ yr⁻¹ (786 m³ s⁻¹) of water and some minor wadies from Iran carry about 7 km³ yr⁻¹ (222 m³ s⁻¹) directly into the southern marsh area (Al-Ansari and Knutsson, 2011).

Several cities have been built on the banks of the Tigris since the dawn of civilization. Among these is Baghdad, the capital city of Iraq. Parts of all of these cities (Mosul, Samara, Baghdad and Al-Kut) were inundated by the spring floods of the river in 1954, 1971 and 1988. To overcome this problem, various hydraulic projects have been constructed along the Tigris and its tributaries in Mosul, Samara, Dokan and Darbandikhan. The control of the river was most efficient during the twentieth century, after huge dams were built to entrap some of the waters (Al-Ansari and Knutsson, 2011). Despite the presence of many hydraulic structures upstream of the city, parts of Baghdad were inundated in 1988. For this reason the Ministry of Water Resources, which had conducted a previous survey of the river in 1976, undertook a second survey in 1991. In 2008 the Ministry of Water Resources made a third survey, extending from the Al-Muthana Bridge north of Baghdad to the Tigris-Diyala confluence in the south using land surveying instrument (total station) and echo sounder.

In the last century, the nature of the successions of high water and flood conditions and the interactions of the flows with the many control structures have induced erosion and
deposition of material on the river bed, as well as the growth and disappearance of islands, to the extent that it has been classified as an unstable river (Geohydraulique, 1977).

During the last twenty years growing islands have become noticeable features in the Tigris channel within Baghdad City, the numbers of islands increasing with time. In this contribution the impact of human activities in dam building, bank lining and dumping of debris within the channel at Baghdad has led to changes in the geometry of the river and its ability to carry flood waters.

2 Discharge of Tigris River for the period 1990–2010

Water flows of the Tigris and Euphrates Rivers entering Iraq have decreased annually in a dramatic way for the past two decades, due to the major water impoundment projects constructed; some remain under construction on these rivers in the neighboring countries, Turkey, Syria and Iran (Al-Ansari and Knutsson, 2011). In addition, the problem has become more severe due to the recent dry climatic period in Iraq. As a result the flow of the Tigris at Baghdad has fallen sharply (about 19% the trend of fall line according to the discharge values mentioned by Al-Shahrabaly, 2008). The average monthly discharge of the Tigris at Baghdad during the period 1960–2010 is shown in Fig. 2. Twenty years average discharge (671 m$^3$s$^{-1}$) decreased to (531 m$^3$s$^{-1}$) during 2000–2010 and it is less than half of the mean daily discharge of 1140 m$^3$s$^{-1}$ prior to 2005 and well below the flood discharges of 4480, 3050 and 1315 m$^3$s$^{-1}$ recorded in 1971, 1988 and 2005 respectively.

3 Previous studies

In the past, several studies have been conducted on the River Tigris. Among these NEDECO (1958) and Herza (1963) examined the hydraulic conditions controlling flows and the hydrological constraints, respectively. Later studies conducted by the Ministry of Irrigation were more related to the present research. The “Tigris River training project within Baghdad City” in 1977 was conducted with Geohydraulique, and a second study, in 1992, was linked with the University of Technology in Iraq. Suspended sediment samples were collected in both programs which were designed to improve the river channel by protecting the banks against water erosion in floods and raising the banks in places of expected overflows.

Similar river training studies have been conducted on many rivers worldwide. Marchi et al. (1996) evaluated river training works in the lower Po River of Italy. There, training activities had successfully reduced the overflows frequency as a consequence of protection and regulation works on the tributaries and also on the main river. The storage capacity of the river bed in floods was reduced due to a reduction of flood expansion areas in the upper and middle parts of the drainage basin.

Lammersen et al. (2002) investigated the impact of river training and retention measures on the flood peaks on the River Rhine in Germany. They found that weirs constructed along the upper reaches and other retention measures had successfully influenced the flood conditions along the river. The SYNHP hydrological model was used to describe the flood routing processes in the river by using single linear stores and this was used to evaluate the effects of retention measures in the upper reaches. The 1-D river flow model SOBEK was used to perform flow calculations for the middle and lower reaches, based on the Saint-Venant equations.
The models indicated that the river training activities led to an increase in peak flow. Korpak (2007) demonstrated the influence of river training on channel erosion in Polish Mountain Rivers. Using data from 53 yr of observations, he showed that debris dams and groynes built before 1980 had caused great changes in channel patterns and increased the channel gradient and the rate of river incision. He considered that although the measures to decrease river downcutting in alluvial deposits worked well, it had not been eliminated. Korpak noted that river training schemes distort the equilibrium of the channel systems and that most such projects were of limited success in the long term because they rarely considered the entire reaches of the rivers.

4 Control structures upstream Baghdad City

Four tributaries contribute to the Tigris River flows upstream of Baghdad (Fig. 1). A number of dams, barrages and regulators have been constructed on the river during and since the second half of the twentieth century (Fig. 3). To link these structures to the Tigris River surveys under examination, they can be classified according to three periods of installation. Prior to 1976 the Samara Barrage (1956) and the Dokan dam on the Lesser Zab tributary (1961) were the two main modifications to the river. During the second period, from 1976 to 1991, the Hemrin dam on the Diyala River has operated since 1981, and the Mosul dam on the Tigris began operating in 1986. The only significant major structure constructed since 1991 was the Adhaim dam, opened in 1999. No detail has been given for anticipated discharge of compensation waters from the 10.4 km³ capacity reservoir to be created by the Ilisu dam in Turkey (yet to be completed) and their potential impact on the water movements in the middle Tigris valley area.

5 Bridges on Tigris River within Baghdad City

The City of Baghdad is divided into two substantial areas by the River Tigris. These are connected by a number of bridges which disturb the flow of the waters. Prior to 1976, six bridges spanned the river in the north of the city. Six more bridges were constructed during the period 1976 to 1991, four more in the north and two in the southern part of the city. Only one additional bridge has been constructed linking the southern parts of the city since 1991. The geographic distribution of the bridges (Fig. 4), with ten towards the north and only three in the south of the city indicates that the resulting disturbance to river flows is greater in the north than in the south.

During the wars of 1991 and 2003, three major bridges (Jumhuriya, Sarafia and the suspension bridge) suffered a high level of damage, causing large pieces of concrete and structural steel to fall into the river. Although some of the larger pieces (steel members) of debris were removed from the river bed, much of the smaller material (concrete fragments that split from the steel structure) could not be removed and remains on the river bed.

The reconstruction procedures for the three bridges required the installation of a temporary bridge for the
6 Changes in river geometry

Three main islands were recognized in the 1976 survey, namely Suraidat (upstream the study reach), Um Al-Khanazer and Abu Rumail (see Fig. 7, the dashed lines highlight the depositional side of the islands), and two smaller islands. The first, Kureat, lay in the second meander of the study reach and the second about 9 km upstream from the Diyala River confluence.

Between 1976 and 1991 a recreation park was constructed on Suraidat Island and an access connected it to the left bank of the river, creating a small lagoon. A similar development at Um Al-Khanazer Island linked it to the right bank, and likewise a lagoon was created beside that bank. The river cross-sections of the 1991 survey revealed changes in the bed and banks of the river and there were indications of new islands (in Abu Nuwas and Dura) growing which had not been identified in the 1976 survey. These changes became more noticeable in the 2008 survey.

During the period 1976–1991 about 97% of the banks of the northern part of the river were subjected to artificial protection using rocks and concrete. The same was true in the southern part of the river, but to a lesser extent. By the end of 2002 about 66% of the banks of the reach had been protected to a level of 35–37 m above sea level in attempts to canalize the river course within the most populated areas and to avoid bank collapse during floods (Al-Ansari et al., 1979).
Seven samples of bed material were taken by van veen grab along 7 km in the northern part of the reach (between Al-A’ameh Bridge and AlShuhada’a Bridge) from the center line and on the quarters of transverse sections. The results of sieve analysis for three of the samples indicated that the major component is fine sand (finer than 0.3 mm diameter) as shown in Table 1. This is in agreement with Al-Ansari and Toma’s (1984) description of the bed sediments of the River Tigris in Baghdad.

The irregularities in the cross-sections of the river reflect the variations in flow velocity controlling erosion or deposition in new parts of the reach. It is important to note that most of the suspended sediments formerly transported to the reach were now being trapped in the upstream reservoirs, so that the river was attempting to achieve a new stable regime (Morris and Fan, 2010).

The repeated surveys have shown that the average slope of the bed of the Tigris within Baghdad was substantially greater in 2008 (5 cm km$^{-1}$) than in 1976 (1.03 cm km$^{-1}$) and more than twice that in 1991 (2.45 cm km$^{-1}$). The obstacles present in the river during the 2008 survey are listed in Table 2, with details of their location, length and type. Some are islands and others areas of bank accretion. Their positions are indicated in Fig. 10.

Between the 1976, 1991 and 2008 surveys reached up to 4 m (Fig. 9). The 1991 cross-section showed the most extreme changes in bed level. This is believed to be due to the survey having been conducted shortly after the 1988 major floods. The bed level variation in 2008 was the least and may be attributed to the fact that the survey was conducted 20 yr after the high flood of 1988 or alternatively was due to the river having suffered from low flow regime during the previous 20 yr.

The recent regional decrease in rainfall is leading to low water levels in the river reaches at Baghdad, and the waters are eroding only below the foundation levels of protection given to the upper banks. It is likely that this will lead to the collapse of parts of the protected banks in the future.

In addition to the variations in bed levels along the reach (Fig. 8), changes in elevation on any single cross section...
Table 2. Main observed obstacles in Tigris River within Baghdad City in 2008.

<table>
<thead>
<tr>
<th>Location</th>
<th>Type</th>
<th>Length (km)</th>
<th>Symbol (Fig. 10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kura’at</td>
<td>Bank deposition</td>
<td>1.4</td>
<td>A</td>
</tr>
<tr>
<td>Kadhimyaa</td>
<td>Bank deposition</td>
<td>0.6</td>
<td>B</td>
</tr>
<tr>
<td>Kadhimyaa</td>
<td>Island</td>
<td>1.0</td>
<td>C</td>
</tr>
<tr>
<td>Kadhimyaa</td>
<td>Bank deposition</td>
<td>1.2</td>
<td>D</td>
</tr>
<tr>
<td>Adhmuyah</td>
<td>Bank deposition</td>
<td>0.6</td>
<td>E</td>
</tr>
<tr>
<td>Adhmuyah</td>
<td>Bank deposition</td>
<td>0.8</td>
<td>F</td>
</tr>
<tr>
<td>Etisiyyah</td>
<td>Bank deposition</td>
<td>0.7</td>
<td>G</td>
</tr>
<tr>
<td>Sinuk-Jumbartuyah</td>
<td>Small islands</td>
<td>–</td>
<td>H</td>
</tr>
<tr>
<td>Abu Nuwas1</td>
<td>Island</td>
<td>0.6</td>
<td>I</td>
</tr>
<tr>
<td>Abu Nuwas2</td>
<td>Island</td>
<td>0.3</td>
<td>J</td>
</tr>
<tr>
<td>Jadriyah</td>
<td>Island</td>
<td>0.4</td>
<td>K</td>
</tr>
<tr>
<td>Dura</td>
<td>Bank deposition</td>
<td>1.5</td>
<td>L</td>
</tr>
<tr>
<td>Dura</td>
<td>Island</td>
<td>0.4</td>
<td>M</td>
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<tr>
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<td>Island</td>
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<td>N</td>
</tr>
<tr>
<td>Dura</td>
<td>Island</td>
<td>1.1</td>
<td>O</td>
</tr>
</tbody>
</table>

7 Methodology

7.1 River geometry

The survey conducted in the winter season of 2007–2008 by the Iraqi Ministry of Water Resources covered 49 km of the river, from the Al-Muthana Bridge in the north to the confluence with the Diyala River in the south. A total of 219 cross sections were surveyed at intervals of 250 m (some cross sections were conducted at lesser intervals especially at meanders), as shown in Fig. 11. The findings of this survey have been used in the present investigation to create a 1-D steady flow model, using the HEC-RAS program, with additional data on the locations and dimensions of the bridges.

7.2 Boundary conditions

The average discharge of the river at Baghdad calculated for the previous ten years and additional discharge figures considered in previous studies (Geohydraulique, 1977 and University of Technology, 1992) have been used in the model to define the upstream conditions, and a modified rating curve for the river below the Diyala confluence was used to define the downstream boundary for each of the upstream conditions.

7.3 Model calibration

Calibration of the model was achieved by using observed water level variations (59 observation points) along the lower 15 km of the studied reach on a single day when the discharge was 400 m$^3$ s$^{-1}$. The problems of calibration were extended to an attempt to define suitable values for the Manning coefficients for the main channel and the flood plain. This was achieved by iteration to give coincidence between the computed water surface levels and those observed. The minimum Root Mean Square Errors (RSME) of 0.026 m were obtained for the coefficient values of 0.0285 for the main channel and 0.042 for the flood plains. No precise data for the water consumption through the reach were available and an estimate of the lateral inflow/outflow was included within the average inflow from the Diyala River of 5 m$^3$ s$^{-1}$.

7.4 Model verification and application

A range of different scenarios were examined by increasing the discharge, starting from the average flow for the previous ten years, in order to determine the critical discharge that can cause inundation. For some of these discharges (from 500 to 1300 m$^3$ s$^{-1}$), water surface levels had been recorded at the Sarai Baghdad station during that ten year period. A new RSME was computed for these observations giving good coincidence (RSME = 0.046 m) as shown in Fig. 12.

8 Results and discussion

The procedure of increasing upstream discharge was continued so that areas that had been inundated could be detected. The discharges that were considered in this work started at 500 m$^3$ s$^{-1}$ and increased in the same discharge steps as those considered in previous studies (Geohydraulique, 1977 and University of Technology, 1992). Each of these discharges was repeated in the model for four scenarios. The difference in each scenario was the lateral inflow represented by the Diyala River. The lateral inflow for the initial (base) scenario was 5 m$^3$ s$^{-1}$, which is the known average inflow served in the Diyala, and it was also used for calibration purposes. The three other lateral inflows (taken from historical data for Diyala river) examined were 25, 50 and 100 m$^3$ s$^{-1}$. The effect of the backwater curve associated with each lateral
inflow was also checked. The average differences in water surface elevation for each scenario compared with the base condition are shown in Table 3. These differences indicate that the lateral inflow exerted no significant influence during periods of higher discharges.

The water surface elevations computed at the Sarai Baghdad station from the present study are plotted against those from previous studies (Geohydraulique in 1977 and University of Technology in 1992) in Fig. 13. The more recent water level predictions are lower than those of the 1976 study for low discharges but higher than those for high discharge. They are always lower than the levels recorded in 1991.

The plots in Fig. 12 indicate that discharges that are higher than 2700 m$^3$s$^{-1}$ could cause partial inundation in some areas in the northern part of the reach. The critical water surface elevation for inundation in the reach is 35 m, at station 43,000 m. For discharges greater than 3500 m$^3$s$^{-1}$ the inundation could take place along approximately 9 km of the reach. For the southern part of the reach under examination, the inundation is not expected to occur below a discharge of 35,000 m$^3$s$^{-1}$.

The water surface slopes for the base condition varied from 6.03 to 6.84 cm km$^{-1}$ for discharges between 400 and 1500 m$^3$s$^{-1}$, respectively. For discharges of 2500 and 2700 m$^3$s$^{-1}$, respectively the slopes were 8.59 and 8.96 cm km$^{-1}$, but reached 10 cm km$^{-1}$ for discharges of 3500 and 4000 m$^3$s$^{-1}$.

The rating curve used to define the downstream boundary condition needs modification for the high water stages to give more reliable estimates of the new geometry conditions in the river.

<table>
<thead>
<tr>
<th>Tigris Flow m$^3$s$^{-1}$</th>
<th>Lat. Flow 25</th>
<th>Lat. Flow 50</th>
<th>Lat. Flow 100</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>0.040</td>
<td>0.102</td>
<td>0.269</td>
</tr>
<tr>
<td>500</td>
<td>0.038</td>
<td>0.087</td>
<td>0.186</td>
</tr>
<tr>
<td>800</td>
<td>0.030</td>
<td>0.067</td>
<td>0.142</td>
</tr>
<tr>
<td>1100</td>
<td>0.023</td>
<td>0.052</td>
<td>0.110</td>
</tr>
<tr>
<td>1300</td>
<td>0.019</td>
<td>0.044</td>
<td>0.095</td>
</tr>
<tr>
<td>1500</td>
<td>0.017</td>
<td>0.039</td>
<td>0.083</td>
</tr>
<tr>
<td>2500</td>
<td>0.010</td>
<td>0.023</td>
<td>0.049</td>
</tr>
<tr>
<td>2700</td>
<td>0.009</td>
<td>0.021</td>
<td>0.047</td>
</tr>
<tr>
<td>3500</td>
<td>0.008</td>
<td>0.020</td>
<td>0.045</td>
</tr>
<tr>
<td>4000</td>
<td>0.007</td>
<td>0.019</td>
<td>0.043</td>
</tr>
</tbody>
</table>
Conclusions

The results of the three surveys and the operation of the model on the channel of the Tigris indicate the following:

1. Recent shortages in the flow have kept the water levels low on all of the river cross-sections so that the protected banks have had little value for flood protection; however, they have helped the river to reach a new stable regime.

2. Since the water is now eroding below the protected bank levels this will lead to the collapse of parts of these banks in the future.

3. The variations in the level of the river bed were less in the 2008 survey than during the surveys of 1976 and 1991.

4. The average slope of the river bed was steeper in 2008 than during the earlier surveys.

5. The bed obstacles during the 2008 survey were greater in number and occupied the most complicated locations than during the two earlier surveys.

6. The output from the model showed very good coincidence with the observed water surface levels at the Sarai Baghdad station and also along the lower 15 km of the reach examined.

7. The computed water surface slopes varied from 6.03 to 6.84 cm km$^{-1}$ during low flow conditions.

8. Inundation could take place along approximately 9 km of the reach surveyed with discharges greater than 3500 m$^3$ s$^{-1}$.

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Evaluation of Dredging Operations for Tigris River within Baghdad, Iraq

Evaluation of Dredging Operations for Tigris River within Baghdad, Iraq

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Abstract

River Tigris divides Baghdad, capital of Iraq, in two parts. The reach of the river within Baghdad is about 60 km long. Many islands and bars are obstructing the flow of the river within Baghdad. To overcome this problem, dredging operations started along most of Tigris River inside Baghdad City to remove many islands and side bars, which reduced the flooding capacity and the efficiency of water intakes. An examination for the dredging plan under process and two proposed additional plans was performed using the Hydrologic Engineering Centers River Analysis System software (HEC-RAS) for a 50 km long river reach to investigate whether they can recover the designed flooding capacity of the river or just improving it. Calibration and verification processes were implemented in the model using observed water levels at Sarai Baghdad gauging station and along the last 15 km of the river reach. Comparisons of computed water levels were conducted with those of previous studies and historical data. Some improvement of flood capacity was achieved based on the recorded data of the last three decades. Cautions about the water intakes should be considered to maintain their function with the expected drop in water level due to dredging operations.

Keywords
Baghdad; Dredging; Flooding Capacity; Tigris River

1. Introduction

Baghdad City is the capital of Iraq. The city is bisected into two parts from the north to the southeast by Tigris
River for a distance of 60 km, 50 km of which are located within the urban areas, and the rest is in rural parts (Figure 1).

Tigris River reach within Baghdad has compound meanders, single channel and alluvial plain characteristics. According to the meandering characteristics, Tigris River can be divided into two parts. The northern part, starts from Al-Muthana Bridge north of the city and ends at Sarai Baghdad gauging station in the centre of the city. This part is characterized by a series of small meanders. While the southern part, which is characterized by its large meanders, starts from Sarai Baghdad gauging station and ends at the confluence between Tigris and Diyala rivers to the south of the city (Figure 1).

The banks of the river within the northern part and about 49% of the southern part were been protected by stones and cement mortar. Thirteen bridges were been installed along the reach; three of which were damaged partially and reconstructed during the last 22 years [1].

Water flow of Tigris River entering Iraq was decreased dramatically during the last two decades due to the huge water projects constructed across the river in Turkey and Iran [2] [3]. The recent dry climatic period in Iraq and the region increased the severity of the problem. As a result, the flow of Tigris River at Baghdad dropped from 927 m³/s as the average monthly discharge (1960-1999) to about 520 m³/s (2000-2012) [4].

Tigris River has many growing islands, side and point bars along its reach inside Baghdad City (surrounded by red borders in Figure 1). The obstacles in the river course can be briefly summarized by the presence of 10 islands and 17 side bars. Dimensions, types and distribution of most of these obstacles were discussed in details by Ali [4].

The negative impacts of these obstacles can be classified under two groups. The first is that related to the hydraulic performance of the river. This includes changing of the river cross sections that reduces the flooding capacity of the river, decreasing water depth at the intakes of water pumping stations and approaching the river bed from the intakes mouths, the impossibility of navigation along the whole reach and limitation to discrete zones, threatening the banks protection stability at some locations due to deep eroding incisions in the river bed near these protections at the meanders. The second impact is related to the river environment and aesthetic such as increasing the turbidity, growing of reeds, water hyacinth and ceratophyllum demersum at stagnant locations, as well as the disfiguration of aesthetic view over the river and its banks.

An effective rapid action was required to consider preventing this deterioration in the performance of the river. Iraqi Ministry of Water Resources (IMoWR) executed dredging operations at specified locations along the river inside Baghdad to overcome the existing problems where at the end of 2012 a flooding wave of 3000 m³/s passed through the river inside Baghdad. This caused inundation of parts of the city where the design flooding capacity of Tigris River was reduced due to the obstacles within the river course [1]. When considering the dredging plan pressing questions arise: Is dredging the more efficient measure to be taken? And how efficient
the dredging operations from the river hydraulic performance point of view? To answer these questions, a simulation hydraulic model was required to investigate the improvements of the flooding capacity of Tigris River along the dredged reach inside Baghdad due to dredging operations at certain locations, as well as determining the associated changes in the hydraulic performance of the river.

In this research, three main questions are to be answered concerning the dredging operations which were executed by IMoWR. Firstly, having these dredging operations executed at certain locations improved the flooding capacity along the whole reach of Tigris River inside Baghdad? If not, then, is there a need for additional measures? Secondly, since the main slope of the river reach changed [1], is there a necessity to recover the design cross section at the dredged locations to get back the original designed flooding capacity? Thirdly, is there a future impact of the dredging operations on the hydraulic performance of the river if there are no other effective measures to consider, which preventing recurrence of forming the same islands or new islands?

2. Previous Studies and Surveys of Tigris River

Three surveys were conducted for Tigris River in different occasions. The first was conducted in 1976 by Geohydraulique [5], followed by University of Technology in 1991 [6], and finally by IMoWR in 2008. The first two surveys (1976 and 1991) are related to studies about training of Tigris River to improve its flooding capacity and prevent possible collapses of its banks due to erosions caused by high discharges. The former suggested banks protections to maintain the cross sections of the river during floods depending upon the geometry of the river as a result of the survey conducted at that time. In the later, recommendations were given to the protections details of the banks that were not protected at that time depending upon the geometry of the river in the 1991 survey. Consequently, the design cross sections of Tigris River are represented on the design drawing of the river’s banks protection project in the two phases discussed above.

The survey of 2008 showed significant changes in the geometry of Tigris River, as well as significant reduction in the flooding capacity of the river [1].

3. Dredging Process of Tigris River inside Baghdad City

IMoWR decided that dredging might be a major treatment process for most of the hydraulic problems in Tigris River. Such kind of work required significant financial and technical potentials where some were not available for IMoWR prior 2008. The main aims set by IMoWR for the dredging operations in Tigris River were:

- Removal of all the sediment deposited on the bed and banks at specified locations along the river to recover the design cross section to maintain the flooding capacity of the river [7].
- Maintaining water intakes and prevent them from clogging.
- Improving the appearance of the river and its banks.

In 2004, IMoWR did some institutional and administrative changes and replaced the company that was specialized in dredging work by an office called “Office of Executing Rivers Dredging Works (OERDW)”. The dredging work was executed at limited locations on the river for the period 2004-2007 using excavators and one dredger at that time. OERDW imported many dredgers and supplementary equipment in 2008. Most of the dredgers are of cutter suction type (Figure 2) and some of the backhoes type (Figure 3). Large staff was trained to execute this operation. The dredging extended over twenty sites along the river as well as purging of ten water intakes (Figure 4).

The work was carried out at some sites and is still in progress at different levels in other sites. In some early dredged sites, especially at water intakes, dredging was repeated due to the accumulation of sediment deposited with time.

Figure 4 shows the locations to be dredged from the main obstacles in the whole northern part of the river reach and about the half of the southern part as planned by OERDW. The remainder of the southern part of the river was not taken into consideration because it has fewer numbers of obstacles and passes through industrial and agricultural areas only.

The areas that were considered in the dredging plan are shown in Figure 5. There were many exceptions in the plan. One of the main obstacles in the northern part (Kura’at Island) which was ignored from the plan due to an investment decision which was taken by the local authorities considering the site as tourist area. The second exception was the biggest island along the river reach (Dura Island) at the middle of the southern part of the river, as well as the side bar just upstream that island (Figure 6).
Figure 2. A dredger of type cutter suction working on Tigris River (the photo was taken by the authors).

Figure 3. A dredger of type backhoe working on Tigris River (source [8]).

Figure 4. Dredging (yellow) and water intakes (red) sites along Tigris River (the raw image from Google Earth©).
Figure 5. The considered areas in the dredging plan of OERDW [6] (the raw image from Google Earth©).

Figure 6. The expected obstacles from OERDW dredging plan ((a) Kura’at Island; (b) Dura Island and the side bar; (c) Al-Rasheed Camp Island and D. PEPSI Factory Island) (the raw images from Google Earth©).
The estimated quantities of sediment to be removed were about 3,443,000 m$^3$ depending upon the topography of 2008 surveys and the additional surveys which were conducted by OERDW at the proposed dredging locations at different dates. New sediment depositions after the dredging operations were neglected in the estimations simply because the current rates of erosion and deposition were unknown.

The dimensions and the depths of dredging depended on the experiences of the engineers of OERDW. Sometimes the dredging does not match the dimensions of the design cross sections. This is due to the fact that they believe that dredging will not necessarily brings the geometry of the river to the designed flooding capacity at that location.

In addition to the plan above, two more dredging programs were suggested. The first, is dredging most of the sediment deposition locations, especially within the upper banks depositions and the stacks of the dredged materials, which were originally excluded from OERDW plan along the northern river reach and the first half of the southern river reach as shown in Figure 7.

The suggested locations that have more effect on the flooding capacity are those of the narrowest sections where the high detentions for the flow may occur at their sections which influence water levels upstream during high discharges as that at the suspension bridge and Dura power plant locations (Figure 8). It should be noted that, the suspension bridge was destroyed in 1991 and re-built with two auxiliary bridges of huge number of piles. This represented a source of additional disturbance for the flow which was associated with deposition of sediment. One of these auxiliary bridges was removed during 2011 and 2012 while the second remains (Figure 8(a)). On the other hand, Dura power plant location (Figure 8(b)) is the narrowest part of the river reach where its width doesn’t exceed 80 m. Furthermore, it is located at sharp meander with lined outer bank.

The estimated quantities of the sediment suggested to be removed according to this plan were about 1,330,000 m$^3$.

The second additional program included the dredging of Dura Island (see Figure 6(b)) in the southern part of the river reach. This is due to the fact that it takes about 59% of the river width and bisects a large meander. In addition, the outer arm of the meander has some obstacles represented by an old intake for a power plant on the left shore and small truss bridge to pass pipes of the new intake from the island to the power plant. The sediment quantity that was suggested to be removed was about 610,000 m$^3$.

4. Methods and Techniques

Fifty kilometres of Tigris River reach which are located within the urban areas was considered in this work starting from Al-Muthana Bridge to the north and ending at the confluence of Tigris-Diyala Rivers to the southeast. Cross sections of 2008 surveys of Tigris River were used to represent the geometry of the river. The average spacing between cross sections is about 250 m.

The Hydrologic Engineering Centers River Analysis System (HEC-RAS) software was used to perform one-dimensional simulation for a range of steady flows in Tigris River and obtain the associated water surface profiles. Calibration is an important step for such kind of mathematical models to get valid results. For this purpose, 59 water surface observations along the last 15 km of the river reach were used to calibrate Manning’s roughness coefficient along the river. These observations were measured for 400 m$^3$/s discharge which represents low discharge for the river. The upstream boundary condition was the same discharge for the measured water levels. While the downstream boundary condition was stage-discharge rating curves for Tigris River downstream the confluence. The contribution of Diyala River was represented by 5 m$^3$/s lateral inflow which represents the average annual flow. The best calibration was achieved by the mean of minimum Root Mean Square Errors (RMSE) between the observed data and computed water surface profile for several iterations of finding suitable Manning’s n values for the main channel and the flood plain. These observations were measured for 400 m$^3$/s discharge which represents low discharge for the river. The upstream boundary condition was the same discharge for the measured water levels. While the downstream boundary condition was stage-discharge rating curves for Tigris River downstream the confluence. The contribution of Diyala River was represented by 5 m$^3$/s lateral inflow which represents the average annual flow. The best calibration was achieved by the mean of minimum Root Mean Square Errors (RMSE) between the observed data and computed water surface profile for several iterations of finding suitable Manning’s n values for the main channel and the flood plain. These iterations culminated by obtaining Manning’s “n” of 0.0285 for the main channel and 0.042 for the flood plain with RMSE equalled to 0.026 m (Figure 9).

Verification process is the next step on the way of modelling. A set of recorded discharges with their water levels at Sarai Baghdad gauging station during the last 12 years were used in the verification process. These discharges (400, 500, 800, 1100 and 1300 m$^3$/s) were classified from low to moderate discharges according to the historical records. Good agreement was achieved between observed water levels for the recorded data and those computed by HEC-RAS for the same discharges. The RMSE for verification was 0.046m at Sarai Baghdad (Figure 9).
5. Dredging Simulation Results

Several modifications for the cross sections were implemented to consider the changes of the river geometry due to the dredging operations. Some changes in the roughness coefficient at the locations of the removed obstacles were considered so that the effect of the additional roughness due to the presence of the obstacles was removed. The first modification taken into consideration was the dimensions of the dredging program that was considered by the decision makers in OERDW, whether they were executed on the field or planned to be executed. The suggested two additional programs mentioned above were also implemented based upon the modification of OERDW plan. The modified geometries were supplied to HEC-RAS program with a range of steady flows to obtain the associated water surface profiles. This is to find out whether the dredging program recovered the design flooding capacity of the river, exceeded the required limits or needs additional dredging.

Water surface profiles were computed along the river reach for a range of discharges (400, 500, 800, 1100, 1300, 1500, 2500, 2700, 3500 and 4000 m$^3$/s) in Tigris River. This range of the selected discharges was examined for the same reach in previous works carried out by Geohydraulique [5], University of Technology [6].
and Ali et al. [1]. Therefore, it was preferable to examine the same discharges with the new river geometries. There was no need to change the downstream rating curve because it was validated in the calibration and verification processes as well as it is far away from the dredging sites.

Water surface profiles related to selected discharges (400, 800, 1100, 2500 and 3500 m$^3$/s) were plotted for all dredging plans as well as the pre-dredging condition in Figure 10. The selection of these discharges was based on different criteria, where the predominant discharge in Tigris River now a day is around 400 m$^3$/s, while a discharge of 800 m$^3$/s represents the moderate discharge for the river during the last decade. The University of Technology [6] concentrated on two discharges, 1100 and 3500 m$^3$/s, assuming that the first one is the operational discharge for the river and the second is the critical indication for the flooding capacity of the river and it was also the significant discharge in evaluating the flooding capacity for 2008 [1]. The discharge 2500 m$^3$/s was chosen for bridging the gap between 1100 and 3500 m$^3$/s discharges.

The drop in water levels was clear for the low discharges (400, 800 and 1100 m$^3$/s) along the river reach that was included in the dredging program and the profiles are mostly identical for all plans. That drop will affect the operation levels of the water intakes especially along the northern part of the reach, so more caution should be taken to keep the water intakes functional. The effect of the second additional plan seems to be very limited and took place only at the location of Dura Island and its vicinity. The differences between the first two dredging programs are restricted to the upper portions of the river cross sections, while the second additional dredging program was concerned with the removal of the island.

During high flooding discharges (2500 and 3500 m$^3$/s) there are convergences between the profiles of the dredging plans and the pre-dredging condition (Figure 10). It should be mentioned, however, there is a very little advantage for the first additional plan which indicates that the effect of the banks deposition is more significant on the flooding capacity of the river. Computed water levels at Sarai Baghdad gauging station were compared with those obtained from the previous works [1] [5] [6] and with a summary of historical records for the last four decades as shown in Figure 11. From this figure, the following conclusions were noticed:

- **1970s**: Many high peaks were recognized. They ranged between 2230 and 3890 m$^3$/s, but the maximum was 4480 m$^3$/s in 1971. This suggests that the patterns of erosion and deposition were not stable due to the oscillation between the amount of sediment brought by floods and flashing process.
Figure 10. Water surface profiles related to selected discharges for all dredging plans and pre-dredging condition.

The recorded water levels were higher than the results of the dredging plans for 400, 500 and 800 m$^3$/s discharges and very close to 1100, 1300 and 1500 m$^3$/s discharges and significantly lower for higher discharges. For this reason, the dredging plans cannot achieve the water levels for the flooding cases of the 1970s, because the river at that period was not protected at the banks and had milder bed slope than it is now [1].

- 1980s: The available water levels are only restricted for the periods between October 1987-September 1988 and October-December 1989. The maximum peak recorded was 3060 m$^3$/s at water level 35.03 m in April 1988. Other peaks (894 to 1998 m$^3$/s) were lower than the previous. The effect of head water dams that were constructed during that decade was clear on the values of the peaks. The available records were close to the results of the dredging plans for the 500, 800 and 1100 m$^3$/s discharges, while they were higher for the 2500 and 2700 m$^3$/s discharges. No records of water levels are available for the rest of the discharges in range.
Figure 11. Computed water levels at Sarai Baghdad gauging station for all dredging plans with the results of previous studies and the observations of the last four decades.

- **1990s**: The peaks were between 990 and 1825 m$^3$/s. The recorded water levels were very close to those of the previous decade. The river seemed more tranquil. The results of the dredging plans are closer to the recorded water levels which covered the discharges 400 - 1500 m$^3$/s from the examined range.

- **2000s**: The range of peaks in those records were the lowest (515 - 970 m$^3$/s) relative to the last four decades. The maximum peak did not exceed 1315 m$^3$/s in 2004. The recorded water levels were very close to those of 1990s but they covered a narrower range (400 - 1300 m$^3$/s) of the examined discharges.

It is axiomatically to know that since the river bed geometry changes, and then the velocity distribution will change also. What we don’t know whether this change will be positive or negative from erosion and sedimentation points of view. To solve this query, a two-dimensional hydrodynamic model is required to be used for the new river geometry to find the velocity distribution. In addition, the texture of the river bed material and the transported sediment are to be investigated. Generally speaking, since the river has these huge quantities of sediment deposited on its bed and banks as mentioned earlier which were coupled with decrease in water discharge, it is very logical to assume that the velocity reduction will lead to more sediment deposition.

A comparison was made to the changes in the longitudinal average velocities along the river reach. This comparison includes the velocities before implementing dredging operations, after implementing dredging operations and for the suggested two plans (Figure 12). This figure shows the negative and positive differences. When the average velocity of the river increases due to dredging with respect to the pre-dredging condition then, this was considered positive. This might keep the deposition rate as it is. Negative differences (which are the most dominant along the river reach) however, are exactly opposite to the first case. Repetition of dredging at some locations on the river can enhance this concept.

6. Conclusions

The conclusions about dredging operations carried out by OERDW on Tigris River within Baghdad after 2008 can be summarized as follows:

1) For low discharges of 400, 800 and 1100 m$^3$/s, there will be clear drop in water levels along those dredged parts of the river.
Figure 12. Velocity differences along the river reach between the average velocities of pre-dredging condition and the results of the dredging plans for the low discharges.
2) The differences between the dredging programs or plans are concentrated on the upper banks depositions which are reflected on the water surface profiles for low discharges where they were almost identical for all dredging plans.

3) For high discharges (the flooding cases) 2500 and 3500 m$^3$/s, the water surface profiles are very converge even with the pre-dredging condition.

4) In flooding cases, the first additional plan gave lower water profiles. This is an advantage for the first additional plan which can satisfy flooding cases.

5) The effect of the second additional plan was limited to the location of Dura Island and nearby areas.

6) The application of the dredging plans can recover the water levels at Sarai Baghdad to those of 1970s for the discharges range 1100 to 1500 m$^3$/s, but not for higher discharges.

7) The dredging plans can reduce Sarai Baghdad water levels for less than those of 1980s for the range discharges 2500 to 2700 m$^3$/s.

8) The expected water levels at Sarai Baghdad will be closer to what they were during the last three decades for low discharges.

9) Some caution should be taken to keep the water intakes functional due to the expected drop in water levels, especially along the northern part of the river reach.

10) Dredging plans along the dredged part of the river reach will be associated with reductions in longitudinal velocities and more sedimentation is expected to take place at these locations.

11) The estimated quantities of sediment to be removed by OERDW plan are about 3,443,000 m$^3$. Fewer amounts of sediment quantities for the first additional plan were estimated (about 1,330,000 m$^3$) and lesser for the second one (about 610,000 m$^3$).

12) The new sediment that will be deposited during the execution of any one of the dredging plans was not considered because the current rates of erosion and deposition are unknown.

Acknowledgements

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References


Flow of River Tigris and its Effect on the Bed Sediment within Baghdad, Iraq

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1 Introduction

Iraq is part of the Middle East and North Africa (MENA) region. It covers an area of 433,970 km² and is populated by about 32 million inhabitants (Figure 1). Baghdad City, the capital of Iraq, is bisected into two areas from the north to the southeast by the Tigris River for a distance of 60 km, 50 km of which are located within the urban areas, and the rest is in rural parts (Figure 2).

Within Baghdad, the Tigris River has a single channel characterized by compound meanders. Thirteen bridges have been installed along this reach to join the western and eastern parts of the city. A series of small meanders are noticed within the northern part of the river, which is located upstream from the center of Baghdad (Sarai Baghdad), and the banks of the river are protected by stones and cement mortar (Figure 2) [1, 2]. The southern part of the river is characterized by large meanders and about half of this river portion has its banks protected [1, 2].

Tigris River has 10 islands and 17 side and point bars along its reach inside Baghdad City (Figure 2) [1, 2]. These islands and bars are affecting the hydraulic performance of the river; this includes changing of the river cross sections, which reduces the flooding capacity of the river, and decreasing water depth at the intakes of water pumping stations, approaching the river bed from the intake mouths. In addition, there is the impossibility of navigation along the whole reach and limitation to discrete zones, and the threat of the banks’ protection stability at some locations due to deep eroding incisions in the river bed. Furthermore, these obstacles are also affecting the environment and aesthetic characteristics, such as increasing the turbidity, growing of reeds, water hyacinth and ceratophyllum demersum at stagnant locations, as well as the disfiguration of aesthetic view over the river and its banks [1, 2]. This has led Iraqi Ministry of Water Resources [3] to dredge parts of the river to attempt to overcome these impacts [2].

Three surveys have been conducted along Tigris River in the city of Baghdad. The first was conducted in 1976 by Geohydraulique [4], followed by University of Technology in 1991 [5], and finally by MoWR in 2008 [3]. It is notewor-
thy to mention that none of these surveys studied the sediment characteristics of the river in details.

In this research, sediment samples from the bed of the River Tigris at a reach extending from the center of Baghdad at Sarai Baghdad gauging station to about Muthana Bridge in the north were been analyzed for their characteristics. The breadth of the river within this reach ranges from 150 to 360 m.

2 River Tigris

The catchment area of the river is 473,103 km$^2$ and is distributed between Turkey, Syria, Iran and Iraq (Table 1) [8–11].

River Tigris enters Iraq four kilometers north of Fieshkhur, near Zakho city (Figure 3). The Tigris is joined by its first tributary inside Iraq, which is known as Khabur River. This tributary is 100 km long; its catchment area is 6,268 km$^2$ with an average discharge of 68 m$^3$/s. The Tigris River runs south for about 188 km in a hilly area to reach Mosul city. At Mosul the average, maximum and minimum discharges of the river are 668 m$^3$/s, 7,740 m$^3$/s (on 2/5/1972) and 85 m$^3$/s (in October, 1935), respectively. The elevation of the channel bed is 225 m above sea level [8, 9, 11].
Figure 3: Schematic diagram of Tigris River and its tributaries hydrological scheme [12].

Table 1: Catchment area of River Tigris.

<table>
<thead>
<tr>
<th>Countries</th>
<th>Tigris River Catchment area (km²)</th>
<th>Catchment area (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turkey</td>
<td>57614</td>
<td>12.2</td>
</tr>
<tr>
<td>Syria</td>
<td>834</td>
<td>0.2</td>
</tr>
<tr>
<td>Iraq</td>
<td>253000</td>
<td>58</td>
</tr>
<tr>
<td>Iran</td>
<td>140180</td>
<td>29.6</td>
</tr>
<tr>
<td>Total</td>
<td>473103</td>
<td>100</td>
</tr>
</tbody>
</table>

About 49 km south of Mosul toward Sharqat city, the Tigris joins its biggest tributary, the Greater Zab (Figure 3). Its total length is 437 km with a mean discharge of 450 m³/s. It supplies 28.7% of the Tigris water [8, 9, 11].

The Tigris River runs south and the Lesser Zab tributary joins the river Tigris (Figure 3). The total catchment area of this tributary is 22,250 km². The total length of the tributary is 456 km with a mean discharge of 227 m³/s. About 30 km downstream of this confluence the Tigris River crosses Fatha gorge. The river Tigris mean, maximum and minimum discharges at Fatha gorge are 1,349 m³/s, 16,380 m³/s (on 3/4/1969) and 200 m³/s (in October 1930), respectively [8, 9, 11].

The Adhaim tributary joins the Tigris 68 km south of Sammara Barrage (Figure 3). The tributary drains an area of 13,000 km² lying within Iraq. Its length is 330 km. The mean daily discharge is 25.5 m³/s. The banks of the river Tigris south of its confluence with the Adhaim tributary are below the maximum flood peak level by 3 m from the left, and by 1.8 m from the right [8, 9, 11].

Further to the south, the river reaches Baghdad. At Baghdad, the mean, maximum and minimum discharges are 1,140 m³/s, 7,640 m³/s (on 12/2/1941) and 163 m³/s (in October 1955), respectively. The slope of the channel is very low, i.e. 6.9 cm/km. It is noteworthy to mention that since 1990 the flow of the River Tigris at Baghdad is completely controlled by the regulating scheme at Sammara and the Adhaim dam.

About 31 km south of Baghdad, the last main tributary “Diyala” joins the Tigris (Figure 3). Diyala’s drainage basin is 31,896 km² with a mean daily discharge of 182 m³/s [8, 9, 11].

Downstream of the confluence of the Tigris–Diyala rivers, the Tigris channel is characterized by its large number of meanders. In addition, the river discharge steadily decreases downstream due to losses. These losses include evaporation, infiltration, and mainly water withdrawal through irrigation canals. There are many small streams running from Iran toward Iraq where water is discharged in the marshes [8, 9, 11].

The channel of River Tigris reaches its minimum width at Kasarah area south of Amarah city. At Qalaat Salih, the mean daily discharge of the river is 80 m³/s. Downstream this city the river joins the Euphrates River at Qurnah city forming Shatt Al-Arab River (Figure 3) [8, 9, 11].

3 Flow Characteristics of River Tigris at Baghdad

The average annual flow of the Tigris River is 21.2 km³ when it enters Iraq. Its tributaries contribute 24.78 km³ of
water and there are about 7 km³ of water brought by small valleys from Iran that drains directly toward the marsh area in the south. This figure greatly fluctuates from year to year (Figure 4).

Records show that the flow of the Tigris has been decreasing with time, where the mean annual flow for the period 1931–1973 was 21.3 billion cubic meters (BCM) and it dropped to 19.1 BCM for the period 1974–2005 [10]. The maximum and minimum annual flow recorded have been 43.1 and 6.5 BCM, respectively [10].

Tigris River hygrograph at Sarai Baghdad gauging station (Figure 5) showed that the maximum flow takes place during April and May. Furthermore, floods and drought are themselves of variable magnitude. Such variations are due to changing meteorological conditions. The period extending from October to February is referred to as a variable flood period, where discharges in the river fluctuate depending on intensity and duration of rainfall at its basin. This period is usually followed by what is known as steady flood period extending from March to April. Furthermore, Figure 5 shows that the hydrograph is becoming flatter since 1990. This is due to the effect of the dams that were constructed upstream Baghdad gauging station, and due to the draught period that is affecting the area as a result of climate change [11].

Long term (1931–2013) monthly discharge records (Figure 4) indicate that there is a general decrease of the flow. It is noticed that the discharge of the river was relatively high during the period 1931–1960, when it reached 1,207 m³/s. During this period, there were no dams constructed on the river. Following that period (1961–2000) some dams were constructed that have caused a relative decrease of the discharge to 927 m³/s. More detailed information in this context indicates that annual water flow of the River Tigris decreased by 5.8% (2.07 km³) between the 1950s and ’60s. This trend has continued with time due to, mainly, climate change and construction of dams. From 1980 onwards, the flow of the river has been at its lowest values, having reached 715 m³/s. This is due to the construction of dams in Turkey and Iraq, mainly. In addition, water from all the valleys within in Iran that were supplying water to the Tigris has been diverted for Iranian use. Furthermore, for the years 2000–2013, the discharge dropped to 522 m³/s. This represent more than 50% reduction of the mean monthly discharge of the previous period, and is well below the flood discharges of 4,680, 3,050 and 1,315 m³/s recorded in 1971, 1988 and 2005, respectively. The drop of the inclination of the trend line for the average monthly discharges is 22.5 m³/s per year for the last 24 years (Figure 6). Consequently, this indicates that the annual flow of the river was reduced by 59.3% during the last 60 years.

Figure 4: Mean monthly flow of the River Tigris at Baghdad for the period 1931–2010 [13].

Figure 5: Decadal hydrographs of River Tigris at Sarai Baghdad for the period 1930–2013.

Figure 6: Trend line for the average monthly discharges at Sarai Baghdad for the period 1989–2013 (Modified after [14]).

The studied reach at Baghdad is about 18 km long, extending from the center of Baghdad at CS14 at Sarai gauging station upstream, until CS 1 near Al-Muthana Bridge (Figure 7). The sampling points labeled according to the cross section number (e.g. CS-BM) refer to term bed-
Figure 7: Location of the studied cross sections and sampling points.

Figure 8: Bed level of Tigris River at Baghdad (above mean sea level).

material (BM), while the location of the sampling point within the cross section is referred as: R: right, C: center, L: left of the cross section. The breadth of the river varies from 150 to 350 m. The depth of water within the reach varies from 0.05 to about 15 m (Figure 8). The depth generally increases at the outer edges of the meanders.

It should be mentioned, however, that the flow of the River Tigris at Baghdad is highly influenced by two factors. The first is due to the effect of climate change on the flow [15–17]. Rainfall data from the northern part of Iraq [18–21] as well as the central part of Iraq [22] shows that the trend is decreasing with time, which reduces future flow [23]. The second factor affecting the flow is the construction of dams and barrages upstream from Baghdad. Mosul dam on the river at the northern part of Iraq is the first dam constructed in 1986 on the Tigris once it enters Iraq from Turkey. Another dam (Dokan) was constructed in 1959 on the Lesser Zab River. Then at Sammara (about 100 km north Baghdad), Tharthar project controls the flow of the Tigris River since 1956. In 1999 a dam was constructed on the Adhaim tributary. This implies that all sediments that are transported within the Tigris River are mainly transported from the area downstream Sammara City.

4 Materials and Methods

The studied reach starts at the center of Baghdad (Sarai gauging station) and extends about 18 km upstream from the station. For the current work, field surveying was conducted recently between May-2012 and January-2013 and included:

1. Installing 15 benchmarks on the banks of the river along the study reach. The DGPS device TOPCON GNSS GR3 was used for determining the coordinates of these benchmarks based on UTM-WGS84 coordinates system; also a transformation to the Iraqi na-
tional triangulation network (known as “Polservice” according to the Polish firm that established these points) was done. All the banks were lined by limestone and cement mortar. Surveying was done in the upper river banks from the crest of the stony protection to the water surface at an average spacing of about 200m between the sections along the northern part of the river reach. Leica Builder 405 and TOPCON GTS 225 total stations were used in these measurements with the same coordinates system of the benchmarks.

2. Sixty five cross sections were surveyed at the same locations considered using EAGLE SeaCharter 480DF sonar with GPS and WAAS external antenna. Water surface elevations were measured at the beginning and the end of the reach segment, which was surveyed every working day, to transform the water depths to bed elevations as well as the locations coordinates. Extensive surveying of water depth were done around existing islands.

3. Forty six sediment samples were collected from the bed of the River Tigris at 15 cross sections (Figure 7) using van Veen grab. In each cross section, one sample was taken from the left side, another from the right side, and one from the middle. Additional samples were taken near the islands and in the meanders. These samples were dried in the lab and prepared for particle size analysis. Particle size was determined by sieving the dried sediment samples. The portion of the samples that were less than 0.0625 mm was tested using hydrometer test. During these tests the dispersion used was sodium hexameta phosphate, which was prepared according to the British Standard 1377 [24]. Details of the procedures are reported by Folk [25]. Cumulative curves (% coarser) were drawn (Figure 9), and the statistical parameters were calculated according to Folk [25]. The percent of sand, silt and clay were also calculated from the cumulative curves.

4. The water depths of the bathymetric survey of 2012 were transformed to bed levels using observed water levels at the nearest benchmarks. Triangulated irregular network (TIN) was created for the bed levels (Figure 8) along the reach using ArcGIS 10.1.
The same ArcGIS technique was used to create TIN maps for the distribution of bed composition percentages of sand, silt and clay (Figures 10(a), 10(b), and 10(c)) as well as for the distribution of computed statistical parameters such as median, mean, sorting, skewness and kurtosis (Figures 12(a)–12(c) and 13(a)–13(c)).

5 Bed Sediment Characteristics

The bed of the river reach is mainly covered by sand (Figure 10(a)), while silt and clay cover small portions of the studied reach (Figures 10(b) and 10(c)). The percentage ratio of sand:silt:clay was 90.74:6.86:2.4. The size distribution curves of the sediment (Figure 8) showed deviation from straight line generally at two points. The first lies between 3.8 to 4 phi (0.074–0.0625 mm) while the second lies at 1.8 phi (0.3 mm). Using Folks classification [25], 72% of the sediments were sand and 24% were silty sand, while the remainder 4% was sandy silt (Table 2). When USDA [26] textural soil classification was used, then the majority of the sediment (72%) was sand, followed by loamy sand (20%), sandy loam (4%) and loam (4%) (Table 3). More recent and detailed classification of Blott and Pye [27] showed that 54.2% of the samples were very slightly silty very slightly clayey sand. This is followed by 15.2% very slightly clayey slightly silty sand, 11% very slightly silty sand, 6.5% slightly clayey silty sand, 6.5% very slightly clayey sand, 2.2% very slightly clayey silty sand, 2.2% slightly clayey slightly clayey sand and 2.2% slightly clayey sand silt (Table 4).

The sediment showed that the average median size within the reach was 2.49 phi (0.177 mm), while the mean size was 2.58 phi (0.16 mm). In addition, the sediments were moderately sorted. About 35% of the samples were well sorted, whereas 26% were poorly sorted (Table 5). The former samples were mainly located in places where the flow was not disturbed. The majority of the sediments were fine skewed (52.2%), while the remaining sediments were strongly fine skewed or nearly symmetrical (Table 6). As far as the kurtosis of the sediments is concerned (Table 7), generally, more than half the samples (56.3%) were leptokurtic, while 17.4% were mesokurtic, and 13% were extremely leptokurtic. Previous studies [28, 29] showed that the sediments were coarser on the bed of the Tigris River. This is believed to be due to the construction of Adhaim dam on Adhaim tributary in 1999, which has caused coarse sediments to become trapped within the Adhaim reservoir. That tributary is believed to be the main supplier of sediment in the river above Baghdad. For this reason, the grain size of the bed has decreased in size, and the bed load now transported are of the size ranging from about 0.0625 to 1.0 mm in diameter (4–0 phi). Part of the load is expected to be transported by dragging or rolling on the bed (about 1.0–0.25 mm in diameter), while the other part (0.25–0.07 mm) by saltation [30].

A TIN of flow depths (see Figure 11) was extracted from bed levels along the studied reach at 500 m³/s discharge. This value represents the flow of the river during the field work and is very close to the average monthly discharge (Figure 4). A value of 6.9 cm/km was used for water surface slope. The water mean depth at CS1 ranged between 2–6 meters, with the deep part of the channel close to the right side. This area is characterized by sediments with a median and mean size range between 2.2–2.6 phi (0.22–0.178 mm) and 1.5–3.5 phi (0.35–0.088 mm),

| Table 2: Folk soil classification for Tigris River bed samples [25]. |
|--------------------------|------------------|-------------|
| Folk class               | No. of samples   | Percentage %|
| Sand                     | 33               | 71.74       |
| Sandy silt               | 2                | 4.35        |
| Silty sand               | 11               | 23.91       |

| Table 3: USDA soil classification for Tigris River bed samples [26]. |
|--------------------------|------------------|-------------|
| USDA class               | No. of samples   | Percentage %|
| Sand                     | 33               | 71.74       |
| Loamy sand               | 9                | 19.57       |
| Sandy loam               | 2                | 4.35        |
| Loam                     | 2                | 4.35        |

| Table 4: Blott and Pye soil classification for Tigris River bed samples [27]. |
|--------------------------|------------------|-------------|
| Blott and Pye class      | No. of samples   | Percentage %|
| very slightly silty very slightly clayey sand | 25 | 54.2 |
| very slightly clayey slightly silty sand | 7 | 15.2 |
| very slightly clayey silty sand | 1 | 2.2 |
| slightly clayey silty sand | 3 | 6.5 |
| very slightly silty sand | 5 | 11 |
| very slightly clayey sand | 3 | 6.5 |
| slightly clayey clayey sand | 1 | 2.2 |
| slightly clayey sandy silt | 1 | 2.2 |
Figure 10: (a) Distribution of sand; (b) Distribution of silt; (c) Distribution of clay on the bed of Tigris River at Baghdad.
Table 5: Sorting of sediment samples from the bed of River Tigris within Baghdad.

<table>
<thead>
<tr>
<th>Type of sorting</th>
<th>No. of samples</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very well sorted</td>
<td>3</td>
<td>6.5</td>
</tr>
<tr>
<td>Well sorted</td>
<td>16</td>
<td>34.8</td>
</tr>
<tr>
<td>Moderately well sorted</td>
<td>10</td>
<td>21.7</td>
</tr>
<tr>
<td>Moderately sorted</td>
<td>3</td>
<td>6.5</td>
</tr>
<tr>
<td>Poorly sorted</td>
<td>11</td>
<td>23.9</td>
</tr>
<tr>
<td>Very poorly sorted</td>
<td>3</td>
<td>6.5</td>
</tr>
<tr>
<td>Extremely poorly sorted</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 6: Skewness of sediment samples from the bed of River Tigris within Baghdad.

<table>
<thead>
<tr>
<th>Type of skewness</th>
<th>No. of samples</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strongly fine skewed</td>
<td>17</td>
<td>37.0</td>
</tr>
<tr>
<td>Fine skewed</td>
<td>24</td>
<td>52.2</td>
</tr>
<tr>
<td>Nearly symmetrical</td>
<td>5</td>
<td>10.8</td>
</tr>
<tr>
<td>Coarse skewed</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Strongly coarse skewed</td>
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<td>0</td>
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</tbody>
</table>

Table 7: Kurtosis of sediment samples from the bed of River Tigris within Baghdad.

<table>
<thead>
<tr>
<th>Type of kurtosis</th>
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<th>Percentage</th>
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</thead>
<tbody>
<tr>
<td>Very platykurtic</td>
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<td>0</td>
</tr>
<tr>
<td>Platykurtic</td>
<td>2</td>
<td>4.4</td>
</tr>
<tr>
<td>Mesokurtic</td>
<td>8</td>
<td>17.4</td>
</tr>
<tr>
<td>Leptokurtic</td>
<td>26</td>
<td>56.5</td>
</tr>
<tr>
<td>Very leptokurtic</td>
<td>4</td>
<td>8.7</td>
</tr>
<tr>
<td>Extremely leptokurtic</td>
<td>6</td>
<td>13.0</td>
</tr>
</tbody>
</table>

respectively (Figures 12(a) and 12(b)). They are also moderately well sorted, fine skewed and leptokurtic to mesokurtic (Figures 13(a), 13(b) and 13(c)). Midway to CS2, water depth begins to decrease to 2–4 m and sometimes to 0.05–2.0 m (Figure 11). This is due to accumulation of sediment forming an island close to CS2 [1, 2]. In view of this, the deep part of the channel shifted toward the left, and this change of flow pattern caused disturbances for the sediments, whereby the median and mean changed from 2.2 to 3 (0.22–0.125 mm) and 1.5 to 4.4 phi (0.35–0.046 mm), respectively (Figures 12(a) and 12(b)). The depth of water from CS2 until CS5a was between 2–6 m (Figure 11). Further downstream, between CS5b, CS5a and CS5, the deep channel changes from right to the left and again to the right (Figure 8). The median and mean size of the sediments is within the range of 2.2–2.4 phi (0.22–0.17 mm) and 1.5–3.5 phi (0.35–0.088 mm), respectively (Figures 12(a) and 12(b)). The sorting of the sediments in this section of the river were poorly to well sorted, although the majority were well sorted (Figure 13(a)). The skewness of the sediment shows that they were either strongly fine skewed to fine skewed (Figure 13(b)). The kurtosis of the sediment was mainly very leptokurtic to leptokurtic (Figure 13(c)). It seems that the flow through the meander of the river at this part did not disturb the sediment.

The area between CS5 to CS6-5 showed that the deep part of the channel shifted from the right to the left at CS6-1, eventually an island appeared on the left side (Figure 11). The water depth in this part was about 4–6 m, but it should be mentioned that in the outer part of the meander it reached 8 m in some parts, while on the inner part of the meander it decreased to less than 2 m. The disturbance of flow due to the presence of the island within
the meander disturbed the characteristics of the sediment. The median and mean size was of the order of 2.2–3.6 phi (0.22–0.08 mm) and 1.5–3.5 phi (0.35–0.088 mm), respectively (Figures 12(a), 12(b)). The sorting of the sediments was mainly poorly to moderately well sorted, while they were strongly fine skewed to fine skewed, and very leptokurtic to mesokurtic (Figures 13(a)–13(c)). Between CS6-5 and CS7, the channel of the river is relatively straight. The deep channel is confined to the left side. The water depth in this part of the river was varying between 2–6 m. It seems that the flow was relatively not disturbed due to the effect the distribution of the sediment that had median and mean size of 2.2–2.4 phi (0.22–0.18 mm) and 1.5–2.5 phi (0.35–0.177 mm) respectively. In addition, the sediments were mainly well sorted to moderately well sorted, while they were fine skewed, and mainly leptokurtic.

Downstream CS7 the deep channel shifts from the left side toward the right at CS9. A meander exists in this area. At the outer part of the meander the depth reaches up to 12–14 m, while it reaches less than 2 m in the inner part of the meander (Figure 12). It should be mentioned, however, that the area on the left side of the river between CS 7 and CS8a was dredged, and the left bank of CS8 was scraped by excavators [1, 2]. Furthermore, a bridge exists downstream from CS8. The median size of the sediment grains in this area starts at 2.2 phi (0.22 mm) and gradually increases up to 3 phi (0.125 mm); mean starts at 2.3 up to 3.5 phi (0.18–0.088 mm). The sediments within the vicinity of CS7 are well sorted, and subsequently the sorting gradually deteriorates to poorly sorted at CS8a, then returns to well sorted when reaching CS8 and deteriorates again to poorly sorted at CS9 (Figure 13(a)). The skewness and kurtosis of the sediment in this part of the river were strongly fine skewed and leptokurtic to very leptokurtic, respectively (Figures 13(b)–13(c)). It seems that the variations in the characteristics of the sediment are due to the dredging operations and the existence of the bridge.

Figure 12: Distribution of the sediment (a) median grain size; (b) mean grain size of the bed of the Tigris River in Baghdad.
Figure 13: Distribution of the sediment (a) sorting; (b) skewness; (c) kurtosis of the bed of the Tigris River in Baghdad.
The last section of the reach extends from CS9 to CS14 at Sarai Baghdad gauging station (Figures 8 and 11). Two meanders exist within this section. It can be noticed that the deep part of the channel generally follows the outer side of the meanders always and reaches up to 14 m in depth. Midway between CS9 to CS11 dredging operations took place on the right bank side, then on the left side until CS13. It is noteworthy to mention that a bridge, just 200 m upstream from CS11, was knocked down in 2003 [1, 2]. The sediments in this part of the river have a median diameter of 2.4–3 phi (0.18–0.125 mm) in the first meander, which then drops to 2.2 phi (0.22 mm) along the downstream meander (Figure 12(a)). The mean sediment size is entirely between 2.5 to 3.5 phi (0.177–0.088 mm) in the first meander between CS9 and CS11, while it ranges from 1.5 to 3.5 phi (0.35–0.088 mm) in the second meander (CS11–CS14) (Figure 12(b)). Moderately to moderately well sorted sediments are noticed in the first meander, and the sorting decreases to poorly sorted in the second meander (Figure 13(a)). As far as the skewness of the sediments is concerned, they were mainly strongly fine skewed in the first meander, changing to fine skewed in the second meander (Figure 13(b)). Finally, the kurtosis of the sediment within the first meander was mainly leptokurtic, while it changed to mesokurtic downstream from the second meander (Figure 13(c)). In this reach, it is believed that dredging operations and the collapse of the iron bridge caused diversions of the flow that had their effect on the distribution of sediment and their characteristics.

At Sarai gauging station, the cross section of the river changed with time (Figure 14). It seems that the cross sectional area has been decreasing with time since 1971. It is believed that the decrease of the quantity of flow (Figure 4) is causing a decrease in the capacity and competence of the river. Sections taken at Sarai gauging station since 1976 indicate that 34% of the area of the cross section was reduced when a 2012 survey is considered.

Previous work at Sarai gauging station in Baghdad [31] indicates that the average annual sediment discharge at the station was 4.6 million tonnes during the period 1969/70-1974/75. Later, Al-Ansari and Toma [29] calculated the annual sediment discharge for the period 1958–1985; an average of about 2.36 million tonnes was transported. In addition, during March, April and May 66% of the load is usually transported [28, 29, 32, 33]. It should be mentioned, however, that the average mean daily discharge at that period was 1,160 m³/s and it has dropped to 522 m³/s recently. This implies that the load being currently transported is less than it used to be, especially before 1999, when the construction of the Adhaim dam took place.

Sediments of the bed of the River Tigris were studied through a reach about 18 km long starting from the center of Baghdad at Sarai gauging station upstream to Al-Muthna Bridge. It was noticed that fine sand covers the bed of the river and the sand:silt:clay ratio was 90.74:6.86:2.4. The average median size within the reach was 2.49 phi (0.18 mm) while the mean size was 2.58 phi (0.15 mm). In addition, the sediments were moderately sorted, fine skewed and leptokurtic. The size of the bed sediment decreased relative to previous studies. This is due to reduction of flow and the construction of a dam on Adhaim tributary in 1999, which used to be the main sediment supplier to the Tigris River before entering Baghdad. The effect of climate change and the construction of dams is reflected in the flow of the river, where the discharge of the Tigris River for the period 2000–2013 (522 m³/s) has decreased by about 57% and 44% since 2000 compared to the period 1931–1959 (1,207 m³/s) and 1960–1999 (927 m³/s), respectively. This suggests that the reduction of annual flow from the 1940s to 2000s reached 59.3%. The bed level has increased compared to previous surveys. This is believed to be due to the decrease in the capacity and competence of the river to transport sediment; the cross sectional area at Sari gauging station, as an example, has been reduced by 34% since 1976.

It was noticed that dredging operations and obstructions (e.g. fallen bridge and islands) have disturbed the flow of the river in several sites. This has disturbed the characteristics of the sediment in the vicinity of such areas.

6 Conclusions

Figure 14: Cross section of River Tigris at Sarai gauging station at different periods.
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References


[10] ESCWA (Economic and Social Commission for Western Asia), Inventory of Shared Water Resources in Western Asia, Salim Daboud printing Co., Beirut, 2013.


Spatial Measurement of Bedload Transport in Tigris River

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Spatial Measurement of Bedload Transport in Tigris River

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Abstract Using Helley-Smith sampler, 288 bedload samples were collected from 16 cross sections along 18 km reach length of Tigris River within Baghdad. The spatial distribution of sampling along the reach took into consideration the variance of river topography where 7 meanders, 2 islands and several bank deposits characterize the geometry of the river. The implemented regulation schemes on Tigris River have reduced 44% of water discharges compared to previous period. The spatial variance in topography was effectively scattering the results of the applied twenty bed load formulas. The study results indicated that the complicated geometry of the river reach makes finding a unique representative bedload formula along the study reach rather difficult, and there is no grantee to have good agreement with measurements in the irregular cross sections (meanders, sand bars, etc.). The closest bedload prediction formulas were van Rijn¹⁹⁸⁴. The annual transported quantities of bedload were estimated to be 30 thousand tons (minimum) in 2009 and 50 thousand tons (maximum) in 2013.

Keywords: bedload sampling, spatial bedload, Helley-Smith sampler, meandering river, sand bed, prediction formula, Tigris River.

1. Introduction

Natural rivers transport sediment in two modes, suspended load and bedload. The former is finer and it is transported in suspension whether its source is wash load or bed sediment. While the latter “bedload” is the coarser fraction and it moves in contact with the river’s bed by sliding, rolling and saltating according to the boundary shear stress. An exchange occurs between suspended and bed loads, and between bedload and bed material depending on the sizes of sediment particles, transport capacity, flow velocity, and boundary shear stress (Hickin 1995, WMO 2003).

The main sources of fluvial sediments are watershed erosion, stream erosion and human activities (Vanoni 2006). Since sediment sources are unlimited and streams have sufficient sediment transport capacity, sediment transport will

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continue (Friend 1993). Limited Supply condition of the fine sediment is the case in most natural rivers. When the river transport capacity reduces, certain range of sediment sizes becomes heavy to be kept transported even as bedload, then the river becomes competence-limited case (Hickin 1995). River bed material is the source of one of the sediment load components which is called bed-sediment load. This load’s component is transported as bed load or even in suspension and it should be distinct from the wash load as sediment source (Hickin 1995, WMO 2003).

Prediction of bedload is of primary importance for river engineering and geomorphology (Recking 2009). Its effect on developing the bed forms, driving fluvial incision and knick point propagation (Cook et al. 2013).

It is difficult to measure bedload directly because the measuring sampler performance is affected by several parameters such as hydraulic efficiency, sampler orientation, bed forms, bed material and so on (Gaudet et al. 1994; Hickin 1995). However, a lot of measurements were conducted in labs and natural streams (Helley and Smith 1971, Gaweesh and Van Rijn 1994, Bunte and Abt 2009, Atkinson 1994; Strahlhofer 2010) using different samplers, such as manually operated portable samplers, vortex tube, pit and trough (Diplas et al. 2008).

In this work, an attempt has been made to calculate the bedload transport rate of the northern part of Tigris River within Baghdad directly using field measurement and to predict sediment bed load indirectly using mathematical formulas.

2. Tigris River

Tigris River bisects Baghdad City, the capital of Iraq, in two parts (Fig. 1) for a distance of 50 km within urban zone and 10 km within rural zones (Ali 2013, Ali et al. 2012, 2014). The northern part of Tigris River reach, which is considered in this work, of 18 km length extends from Al-Muthana Bridge to the north to Sarai Baghdad gauging station at the center of Baghdad (Fig. 1.B). This river reach has single thread, compound meanders, and alluvial plain characteristics. The river banks are protected against erosion by aligned stones and cement mortar between levels 29 and 37 m.a.s.l. Recently, the dominant water levels in the reach are below the protection levels (Ali 2013, Ali et al. 2012, 2014).

During the last two decades many new islands, side depositions and point bars appeared in the Tigris River’s reach within Baghdad (Fig. 1.B). These sedimentations in the river course has its impact on the hydraulic performance of the river, such as reducing its flood capacity, impeding navigation and reducing the efficiency of water intakes of water treatment plants, as well as the environmental and aesthetic impacts (Ali et al. 2012, 2014).

Previous study about sediment transport and river training (Geohydraulique
1977) which was conducted on the river reach in Baghdad, mentioned Tigris River sediment is “bed-load”. Since it found the suspended load concentrations never reach 3 g l⁻¹ in high water and never exceed 0.2 g l⁻¹ in low water periods. This study was not based on real field measurements for the bedload. Bed material samples and suspended load samples were collected only in that study. Therefore, it is important to measure the bedload discharge in Baghdad since no measurements were previously performed.

2.1 Hydrology of Tigris River in Iraq

The flow of the river is fully controlled in Baghdad by a system of dams and regulators constructed on the main river and the tributaries upstream of Baghdad (Fig. 2)(Al-Ansari 2013& 2016). These regulating schemes have decreased the average monthly discharge of the river 44% according to the records of the Sarai Baghdad gauging station, it has been 522 m³s⁻¹. The Tigris River hydrograph at Sarai Baghdad (Fig. 3) shows the main delivery events of sediment into Baghdad have been vanished.

Sediment transport rates are affected in the course of Tigris River upstream of Baghdad due to the trapping of sediment within the reservoirs of the headwater.

The only uncontrolled source of sediment that can be delivered to Baghdad is the area restricted from the lower sub-basin of the Adhaim tributary and the catchment between the Samarra Barrage and Baghdad (Fig. 2), as well as the bed and banks erosions.

The delivery of fine sediment from the Adhaim Tributary has not been measured, but a glance at the possible extra flow contribution (rather than flow released from the Adhaim Dam), can give an indication for the estimated sediment delivery. The extra water flow contribution from the Adhaim Tributary sub-basin and Tharthar Lake back feed toward the Tigris River was determined using the mass balance concept. The contribution did not exceed 260 m³s⁻¹ during 2004-2005, which was a moderate year compared with recent more dry years as shown in figure (4). As an average, the extra contribution was 8% of the average monthly discharge at the Sarai Baghdad for the same year.

2.2. River Geometry and Bed Composite

The morphology and the bed sediment of the river were investigated several times inside Baghdad by Geohydraulique (1977), Al-Ansari and Toma (1984), Khalaf (1988), University of Technology (1992) and Al-Ansari et al. (2015). Rapid changes in the bed material from gravel in Samarra to fine sand in Baled (50 km to the downstream of the Samarra Barrage and 130 km to the upstream of Baghdad) (Hardy et al. 1956). The riverbed was already sand bed between Baled and Baghdad even before the construction of the Samarra Barrage in 1956.
The geometry of the study area consists of a series of 7 meanders (Fig. 5) of radii of curvature are ranging from 475m to 1245m (University of Technology 1992). Along the second meander (CS4, CS4-2 and CS5) an island is noticed directly upstream of CS4 and the river cross section transfers from rille at CS4 to pool at CS4-2 and CS5 and the higher velocity zone is also transferred from the inner bank of CS4 to the center of CS4-2 then to the outer bank of CS5. This change in the velocity field gives an indication about the attempts of the river to shift the peak of the meander to the downstream of its’ current location. This can explain the high depositions on the outer bank between CS4 and CS4-2.

The same velocities distribution is repeated at the 4th and 5th meanders (CS8 and CS10 respectively), where the higher velocities are in the centers of the sections and large depositions existed in the front halves of the concave banks.

At the third meander (CS6-1, CS6-2, CS6-3 and CS6-4), a large island mediates the meander. The sections CS6-1 and CS6-4 are of pool type while the sections CS6-2 and CS6-3 are of run type. Depositions are existed along the inner bank of the meander. Mean velocities in both branches are equal while the top width of the right branch is 2.2 times the left branch.

Using the van Veen grab, 46 bed-material samples were collected along the northern reach in Baghdad. The particle size distribution was analysed using the sieves and the hydrometer. It was noticed that fine sand dominant the riverbed. The average median size was 0.178 mm (Al-Ansari et al. 2015b). The size of the bed sediment relatively decreased compared to earlier investigations (see Al-Ansari and Toma 1984). In addition, the sediments were moderately sorted, fine skewed and leptokurtic (Al-Ansari et al. 2015b).

3. Bedload Sampling

A Helley-Smith sampler was manufactured of 3" × 3" (76 mm × 76 mm) opening size and 3.5 exit/entrance expansion ratio. Other dimensions of the sampler were taken from van Rijn (2007). Two exceptions were considered, the weight of the sampler and the size of the mesh bag. The original weight of the sampler was reduced to 15kg, which makes it easier handling in small boat without winch. No problem in the performance was expected to be due to the reduced weight since the average velocity in the river did not reach 1 m s⁻¹. Furthermore, avoiding the oversampling that may occur due to the scooping effect (Gaweesh and Van Rijn 1994, Bunte and Abt 2009) is more valuable. Larger mesh bag of 2200 cm² surface area was used to maintain the sampling efficiency in case the bag is filled for more than 40% of volume and to avoid clogging the bag openings by close size particles and organic materials (Druffel et al. 1976, Beschta 1981, Van Rijn and Gaweesh 1992).

At the quartiles of 16 cross sections (Fig. 5), 288 bedload samples were collected. Sampling times were 60s of 238 samples, 120s of 2 samples, and 300 s of a single sample. The separation period was 3 min between 5 sequences.

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samples at each sampling point to determine the time-averaged rate of sediment transport. Zero-time samples were collected at the sampling points to overcome the initial and scooping effect of the sampler on the bed (Van Rijn and Gaweesh 1992). Velocity profile measurements were conjugated with bedload sampling in all cross sections using SonTek RiverSurveyor ADPC.

The mean weight was determined for the repetitions of the same sampling point. The weights of zero-time samples were subtracted from the means. A reduction factor of 0.5 was applied to the modified means considering the trapping efficiency as 200% for fine sand. Table (1) shows the measured bedload discharge for each cross section along the reach with the bed sediment properties and the hydraulic-geometric parameters those are associated or existed during measurements.

The maximum bedload transport rate was 3.938 kg s\(^{-1}\) at CS8 associated with a water discharge of 643.5 m\(^{3}\) s\(^{-1}\) and the minimum was 0.6822 kg s\(^{-1}\) at CS4 associated with a water discharge of 449 m\(^{3}\) s\(^{-1}\). The average bedload transport rate was 2.099 kg s\(^{-1}\) with 0.889 standard deviation.

The scattering in some of bedload measurements can be attributed to the spatial variation in river topography along the study reach where it is an influencing factor, as well as, bed sediment size, particle size distribution and bed shear stresses (Julien 2010) in case there is no external source of disturbance.

4. Spatial Distribution of Bedload

The variance in the topography and morphology in the Tigris River was reflected in the spatial distribution of bedload and velocity field as shown in figure (6). This figure shows the distribution of the measured bedload discharges per unit width at sampling points along the study reach as well as bed shear stresses. The data series are not in the sequence of display in the figure and separation lines were used to specify cutting in the data series and also to specify the relative parts of each data series to a certain cross section. The following description for the spatial distribution of bedload was associated with the measured velocity distribution using ADCP at the sampling time.

CS1: The section is a run section type. Some stagnation was on the extreme left side because it is located in the shade of bank deposition. Bedload was oscillating across the section. Velocity was uniform except on the extreme left side

CS2: Bank deposition was growing on the left side where flow velocity was low. Bedload was higher towards the right.

CS3: The section is located directly downstream of a large bank deposition. Its left side is hidden by the deposition and the velocity was higher on the right side. The bedload was low in the section, however it was little higher on the right side.
CS4: The section is located between a small island to the upstream and an acute meander to the downstream. The right side was stagnant because it is hidden behind the island and the velocity increased towards the left side. The bedload was higher towards the left side. Eddies were noticed on the right side.

CS4-2: The section is pool type, deeper on the right side. It’s located at the center of an acute meander. On the deeper side, velocity was not at its highest, higher velocities were closer to the center. The bedload was higher in the center.

CS5: The section is pool type, deeper on the right side. Velocity and bed shear were higher on the right side. However, the bedload was higher in the center of the section due to the effect of the secondary flow, since the section is within the downstream half of an acute meander.

CS6-1: The section is pool type; deeper on the left side. The velocity distributed uniformly across the section. The bedload was higher on the left side.

CS6-2: The section is the inner branch of a meander that is bisected by an island. The left side of the section was a trench produced by excavators, so the velocity was low by comparison with the right side. The bedload was higher on the right side where the velocity was higher.

CS6-3: The section is the outer branch of the meander. The flow was turbulent on the most outer side. Bedload was low at all.

CS6-4: The center of the section was close to the tail of a large island, so the secondary flow at the confluence of the two branches was the reason behind the high bedload on both sides.

CS7: The section is of a riffle type. The velocity on the left side was higher, while on the right side, although the bed shear was relatively high, the bedload was low because the velocity was also low.

CS8: The section is pool type; deeper on the right side. However, the velocity was neither high nor was the bedload. The highest bedload was in the center due to the effect of the secondary flow.

CS9: The main flow was on the right side while the left side was stagnant, so bedload was higher on the right and lower on the left.

CS10: The section is pool type, deeper on the left side. The bedload was lower due to higher bed shear, which may suspend the bed sediment.

CS11: The section is the riffle type. Although the right side has the same depth as the left, but bedload was much higher on the left because the right side was stagnant and recently dredged.

CS13: The section is pool type, deeper on the right. The bedload was lower on the right due to higher bed shear, which may suspend the bed sediment. The bedload was higher in the center due to the effect of secondary flow.
CS14: The section is pool type, deeper on the left. The bedload was lower on the left due to higher bed shear and higher flow velocity, suspend the bed sediment.

5. Bedload Prediction Using Formulas

Wide spectrums of bedload predicting formulas were proposed and developed by many researchers depending on different approaches. For each approach, a specified concept was considered as motivation for deriving the approach’s formula and a certain number of parameters were controlled in the lab measurements to estimate the formula parameters.

5.2. Approaches of Bedload Formulas

Twenty bedload formulas were selected and applied on the study reach to predict the bedload discharge to find the best suitable formulas. Brief descriptions for the used approaches are given below:

5.2.1. Shear stress approach

The movement of bed material particles will start when the criteria of incipient motion is exceeded. So, shear stress near the bed will entrain the sediment particles to motion as long as the shear stress is greater than the critical shear stress of the particles. The following formulas which belong to this approach were used in this work:

a. DuBoys1935 formula (Graf 1971)
b. Shields1936 formula (Yang 1996)
c. Kalinske1947 formula (Yang 1996)
d. Cheng-Simons-Richardson1965 formula (Yang 1996)
e. Wong-Parker2006 formula (Wong and Parker 2006)

5.2.2. Energy slope approach

The bedload motion is initiated due to the portion of energy losses coming from the grain resistance (Yang 1996). The following formulas were used from this approach:

a. Meyer-Peter1933 formula (Vanoni 2006)
b. Meyer-Peter-Muller1948 formula (Graf 1971)

5.2.3. Discharge approach

In natural rivers, critical unit discharge was used as an indication on starting bedload sediment motion when it is exceeded by water discharge
(Talukdar et al. 2012). The following formulas which belong to this approach were used in this work:
   a. Schoklisch_{1934, 1943} formula (Yang 1996)
   b. Casey_{1935} formula (Khalaf 1988)

5.2.4. Probabilistic approach
Probability concepts were introduced in bedload prediction by the pioneer work of Einstein in 1942. The turbulent flow fluctuations are the driver for sediment entrainment rather than the flow forces exerted on the particle. Both of the entrainment and the deposition were expressed in probability terms (Yang 1996).
   a. Einstein_{1940} bedload function
   b. Einstein-Brown_{1940} formula

5.2.5. Regression approach
Data driven models (regression, ANN) were used to explain the bedload transport process due to the limitations of defining this complex process into precise formula (Talukdar et al. 2012). The following formulas were used within this approach:
   a. Rottner_{1955} formula
   b. Yalin_{1963} formula (Yalin 1977)
   c. Van Rijn_{1984} formula (van Rijn 1993)
   d. Julien_{2002} formula (Julien 2002)
   e. Camenen-Larson_{2005} formula (Camenen and Larson 2005)

5.2.6. Equal mobility approach
The flow forces act on the exposed particles causing mobilization with possibility of participation of the substrate particles into bedload movement at scour zones due to their exposure on the surface (Yang 1996).
   a. Wilcock_{2001} formula (Wilcock 2001)
   b. Wilcock-Crowe_{2003} formula (Wilcock and Crowe 2003)

5.2.7. Power Concept
This approach has developed from the concept that there is a relation between the available energy to the river with the rate of work done by the river to transport sediment (Yang 1996). The following formula was used within this approach:
   a. Bagnold_{1956} formula
5.3. Application of Bedload Formulas

Two kinds of datasets were required for applying bedload formulas, physical properties of river bed sediment and hydraulic-geometric parameters of the study reach. Sediment characteristics were determined from the size analysis of the bed materials samples. The hydraulic-geometric parameters included; water depth, cross sectional area, top width, wetted perimeter, hydraulic radius, water surface slope and water discharge. These datasets were extracted from field measurements in the sampled cross sections. The results published by Al-Ansari et al. (2012) contained most of the datasets, whilst other datasets were listed in table 1.

The results of the bedload formulas at sixteen cross sections along the study reach were compared with the measured bedload discharges in the same section and two indicators were used to measure the accuracy of the predicted bedload. The discrepancy ratio, which is the ratio of predicted bedload to measured one (van Rijn 1993), was one of the indicators and the error percentage (Walling 1977) was the other. The comparisons of results are shown in figure (7). Six zones of different discrepancy ratios were specified in the figure to explain the distribution of the results around the perfect agreement line.

Most of the formulas overestimated the bedload transport rate by more than 10 times and even 100 times relative to field measurements. Five formulas from four of the approaches predicted bedload discharges close to measurements. These formulas were Meyer-Peter1938, Scholllitsch1934, 1943, van Rijn1984 and Einstein1950, bedload function with average discrepancy ratios of 0.5, 1.51, 0.47, 1.18 and 4.06 respectively. The predictions of van Rijn1984 and Scholllitsch1934 formulas are distributed on both sides of the perfect agreement line. Whilst both of Meyer-Peter1934 and Scholllitsch1943 formulas are mainly bounded between the perfect line and discrepancy ratio ½. Some results of Einstein1950 were in the area between the perfect agreement and r = 8.

Table (2) shows the accumulated percentages of the predicted bedload discharges according to each range of the discrepancy ratio. The higher percentage of predicted bedload within the closer range of discrepancy ratio 0.75 ~ 1.25 (Error% = -25 ~ +25) was equally between Scholllitsch1934 and van Rijn1984 formulas and the results approximately continued in this manner until the third zone of discrepancy ½ ~ 2 (Error% = -50 ~ +100). At this range, more than 76% of Scholllitsch1934 and 53% and of van Rijn1984 predictions were located within the range. The percentages of the other three formulas didn’t exceed 24% for the discrepancy range ½ ~ 2 (Error% = -50 ~ +100).

To clarify the behavior of the bedload formulas at different cross sections, having varied morphological characteristics, the formulas were applied for a range of discharges between 400 and 700 m³s⁻¹ at some sections along the reach. Figure (8) show that Einstein’s formula was over-predicting in all sections and it
showed multiple points of change in the slope at cross sections CS1, CS6-1, CS7 and CS9 depending on the water flow, whilst at sections CS6-4, CS11 and CS14, the formula curves were smoother. The Meyer-Peter$_{1934}$ and Schoklitsch$_{1934}$ formulas were always under-predicting. The Schoklitsch$_{1934}$ and van Rijn formulas fluctuated between the measurements being under and over depending on the characteristics of the cross section.

It is not clear that there is a unique prediction formula that can predict the bedload discharge with stable magnitude of error along the whole study reach. The important conclusion from the application results is, even for those formulas have agreement with field measurements in regular cross sections, there is no grante to have the same agreement in the irregular cross sections (meanders, sand bars, etc.).

Annual bed load quantities were computed using all the formulas for the period 2009-2013 along the study reach and are listed in table 3. The annual bedload quantities are ranging from 36 thousand ton (minimum) in 2009 to 50 thousand ton (maximum) in 2013 according to van Rijn$_{1984}$ formula.

6. Conclusions

The implemented regulation scheme on the Tigris River has limited the sources of sediment supply; it has also decreased the average water flow to 44% compared to previous periods.

The spatial distribution of the bedload was effected by the bed shear and the flow velocity at the sampling point. Whenever the velocity and the bed shear increase, the bedload increases also for certain limit depending on the particle size then it may transfer to suspension. Some exceptions are expected in the meanders due to the secondary current, where the bedload increases for a lower velocity and/or bed shear.

The complicated geometry of the river reach makes finding a unique representative bedload formula along the study reach rather difficult. Furthermore, even for those formulas having agreement with field measurements in regular cross sections, there is no grante to have the same agreement in the irregular cross sections (meanders, sand bars, etc.).

The closest bedload prediction formulas were van Rijn$_{1984}$ then Schoklitsch$_{1934}$ and the average discrepancy ratios were of the order 1.18 and 1.51 respectively.

Annual bedload quantities were estimated for the period 2009-2013 to be 36 thousand tons (minimum) in 2009 and ranged to 50 thousand tons (maximum) in 2013 according to the van Rijn$_{1984}$ formula. The average annual transport rate for the period 2009-13 was 42.6 thousand tons.

7. References


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Bridge, J. S., and Best, J. L., 1988, Flow, sediment transport and bedform dynamics over the transition from dunes to upper-stage plane beds: implication for the formation of planar laminae. *Sedimentology*, 35, 753-763.

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Wilcock, P.R., 2001. TOWARD A PRACTICAL METHOD FOR ESTIMATING SEDIMENT-TRANSPORT RATES IN GRAVEL-BED RIVERS. Earth Surface Processes and Landforms, 26, 1395–1408
World Meteorological Organization (WMO), 2003. MANUAL ON SEDIMENT MANAGEMENT AND MEASUREMENT. OPERATIONAL HYDROLOGY REPORT No. 47, WMO-No. 948.
Fig. 1 (A) Map of Iraqi Provences (B) Tigris River inside Baghdad, capital of Iraq (islands and sandbars bordered by red) (Ali et al. 2014, 2015) 
209x148mm (300 x 300 DPI)
Fig. 2 Schematic Diagram of Tigris River Hydrological Scheme (MWR 2005).
297x420mm (300 x 300 DPI)

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Fig. 3 Decadal hydrographs of Tigris River at Sarai Baghdad for the period 1930-2013 (data source: Al-Shahrabaly 2008), 1616x1055mm (96 x 96 DPI)

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Fig. 4 Contribution of runoff of Adhaim River sub-basin downstream Adhaim Dam to Tigris River discharges for the year 2004~2005.
1616x1057mm (96 x 96 DPI)
Fig. 5 Bedload sampling cross sections along the northern part of Tigris River (Ali et al. 2014, 2015)
297x420mm (300 x 300 DPI)

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Fig. 6. The measurements of the bedload exchanges with the measured velocity and the calculated bed shear stresses at the site 792.2054mm (30.6 x 96 gph).
Fig. 7 Comparison of bedload discharges predicted by different bedload formulas with measured bedload in Tigris River.
Fig. 8 Application of bedload formulas and rating curve at different cross sections for a range of discharges.  
297x420mm (300 x 300 DPI)
Fig. 8 Continued...
297x420mm (300 x 300 DPI)
Table 1 The measured bedload rates with the bed sediment properties and hydraulic-geometric parameters along the northern reach of Tigris River.

<table>
<thead>
<tr>
<th>C.S.</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{10}$ (mm)</th>
<th>Cross-sectional Area (m$^2$)</th>
<th>Top Width (m)</th>
<th>Hydraulic Radius (m)</th>
<th>Discharge (m$^3$ s$^{-1}$)</th>
<th>Water velocity (ms$^{-1}$)</th>
<th>Bedload rate (kg s$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>0.194</td>
<td>0.273</td>
<td>664.9</td>
<td>180.03</td>
<td>2.98</td>
<td>457.381</td>
<td>0.688</td>
<td>1.268</td>
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<tr>
<td>CS2</td>
<td>0.166</td>
<td>0.235</td>
<td>653.7</td>
<td>260.77</td>
<td>2.471</td>
<td>459.022</td>
<td>0.702</td>
<td>2.590</td>
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<td>CS3</td>
<td>0.1755</td>
<td>0.25</td>
<td>795.2</td>
<td>261.6</td>
<td>3.008</td>
<td>464.409</td>
<td>0.584</td>
<td>1.341</td>
</tr>
<tr>
<td>CS4</td>
<td>0.199</td>
<td>0.273</td>
<td>691.5</td>
<td>250.06</td>
<td>2.743</td>
<td>445.095</td>
<td>0.644</td>
<td>0.682</td>
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<tr>
<td>CS4-2</td>
<td>0.197</td>
<td>0.276</td>
<td>745</td>
<td>241.09</td>
<td>3.013</td>
<td>452.325</td>
<td>0.607</td>
<td>1.142</td>
</tr>
<tr>
<td>CS5</td>
<td>0.208</td>
<td>0.278</td>
<td>643.3</td>
<td>151.67</td>
<td>4.072</td>
<td>489.233</td>
<td>0.76</td>
<td>1.599</td>
</tr>
<tr>
<td>CS6-1</td>
<td>0.199</td>
<td>0.273</td>
<td>865.2</td>
<td>353.85</td>
<td>2.398</td>
<td>549.877</td>
<td>0.636</td>
<td>2.901</td>
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<td>CS6-2</td>
<td>0.21</td>
<td>0.275</td>
<td>421.284</td>
<td>185.2</td>
<td>2.271</td>
<td>286.409</td>
<td>0.68</td>
<td>0.924</td>
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<tr>
<td>CS6-3</td>
<td>0.145</td>
<td>0.255</td>
<td>369.83</td>
<td>83.37</td>
<td>4.113</td>
<td>251.023</td>
<td>0.679</td>
<td>0.190</td>
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<td>CS6-4</td>
<td>0.19</td>
<td>0.27</td>
<td>760.4</td>
<td>237.8</td>
<td>3.141</td>
<td>561.778</td>
<td>0.739</td>
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<td>CS7</td>
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<td>932.7</td>
<td>320.08</td>
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<td>651.709</td>
<td>0.699</td>
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<td>CS8</td>
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<td>236.86</td>
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<td>643.319</td>
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<td>CS9</td>
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<td>0.218</td>
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<td>530.443</td>
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<td>1128.40</td>
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<td>CS13</td>
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<td>CS14</td>
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<td>522.226</td>
<td>0.734</td>
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### Table 2: Accumulative percentages of predicted bedload according to the ranges of discrepancy ratio.

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<tr>
<th>Formulas</th>
<th>Ranges of discrepancy ratio and corresponding error percentages</th>
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<td>0.75 – 1.25</td>
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<td></td>
<td>-25 – +25</td>
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<td>Meyer-Peter 1954</td>
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<tr>
<td>Schoklitsch 1974</td>
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<tr>
<td>DuBois 1935</td>
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<tr>
<td>Casey 1935</td>
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</tr>
<tr>
<td>Shield 1935</td>
<td>0</td>
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<td>Rottner 1969</td>
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<td>van Rijn 1984</td>
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<td>Wilcock 2001</td>
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<tr>
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<td>Wong-Parker 2004</td>
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Table 3 Annual bedload predicted discharges for the period 2009–2013.

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<tr>
<th>Year</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
<th>Average</th>
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<td>Meyer-Peter 1964</td>
<td>17.88</td>
<td>21.07</td>
<td>19.59</td>
<td>21.96</td>
<td>24.61</td>
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<td>50.51</td>
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<td>81.87</td>
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<td>Duss 1993</td>
<td>1727.07</td>
<td>2090.06</td>
<td>1880.12</td>
<td>2087.37</td>
<td>2318.04</td>
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<td>Casey 1935</td>
<td>680.44</td>
<td>787.01</td>
<td>739.41</td>
<td>818.28</td>
<td>902.62</td>
<td>785.55</td>
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<td>Shield 1936</td>
<td>4085.10</td>
<td>4919.11</td>
<td>4512.47</td>
<td>5125.81</td>
<td>5885.69</td>
<td>4905.63</td>
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<td>18.93</td>
<td>20.75</td>
<td>22.66</td>
<td>19.97</td>
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<td>Kalinske 1947</td>
<td>1116.36</td>
<td>1187.90</td>
<td>1170.03</td>
<td>1215.75</td>
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<td>Einstein 1950</td>
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<td>178.58</td>
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<td>Einstein-Brown 1950</td>
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<td>649.50</td>
<td>616.26</td>
<td>672.13</td>
<td>728.98</td>
<td>647.79</td>
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<td>Rottner 1959</td>
<td>548.02</td>
<td>635.31</td>
<td>595.40</td>
<td>657.92</td>
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<td>634.46</td>
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<td>2636.03</td>
<td>2507.32</td>
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<td>Chang et al. 1963</td>
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<td>Bagnold 1966</td>
<td>897.41</td>
<td>1019.74</td>
<td>967.51</td>
<td>1056.16</td>
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<td>1017.59</td>
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<td>van Rijn 1988</td>
<td>36.17</td>
<td>42.58</td>
<td>39.56</td>
<td>44.23</td>
<td>50.50</td>
<td>42.61</td>
</tr>
<tr>
<td>Wilcock 2001</td>
<td>461.07</td>
<td>527.38</td>
<td>498.35</td>
<td>546.79</td>
<td>598.19</td>
<td>526.36</td>
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<tr>
<td>Julien 2002</td>
<td>1441.44</td>
<td>1644.19</td>
<td>1555.99</td>
<td>1703.37</td>
<td>1860.09</td>
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<td>Wilcock-Crowe 2003</td>
<td>576.33</td>
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<td>622.94</td>
<td>683.49</td>
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<td>657.95</td>
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<td>Cammen-Larson 2003</td>
<td>1024.46</td>
<td>1158.42</td>
<td>1102.02</td>
<td>1199.45</td>
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<td>Wong-Parker 2006</td>
<td>398.76</td>
<td>448.85</td>
<td>428.72</td>
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<td>499.27</td>
<td>448.16</td>
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Total Sediment Load in Tigris River at Baghdad City

Abstract: The construction of hydraulic structures upstream Baghdad has limited the sediment supply to the river & reduced the water discharge of the river by 44%. The total load discharge rate in a reach that extend 18 km upstream the center of Baghdad was studied where 247 suspended load samples were collected from 16 cross sections. The results indicated that the suspended load is the dominant mode in the total load with minimum percentage 93.5%. After adding the bed load, the total load ranged from 29.1 to 190.3 kg/s. These measurements have been used to establish total load rating curve of power function that spatially reliable. The spatial distribution of sampled cross sections took into consideration the variance of river topography where 7 meanders, 2 islands & several bank depositions are characterized the geometry of the river. The associated errors from using the proposed rating curve are within reassuring levels & less than any errors produced from other twenty two total load predictors that were applied on the same reach. Scattering of the other predictors results can be attributed to the spatial variance in topography while it has less effect on the proposed rating curve. According to the final results obtained, it's recommended to use the proposed procedure for establishing spatial total load rating curve for morphologically complicated rivers. The annual transported quantities of total load were estimated 2.47 & 4.23 million tons for 2009 & 2013 respectively.

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Jillian Labadz
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Opposed Reviewers:
Dear Editor

Please find attached our paper entitled: Total Sediment Load in Tigris River at Baghdad City

The importance of this work is establishing a total load rating curve. It shows the spatial distribution of the load. Furthermore, it can be applied on reaches of the river that are not straight (i.e. existence of meanders, islands, and bank depositions). The implementation of this research was carried out on a stretch of the River Tigris north Baghdad. The catchment area of this reach is of limited sediment supply. This due to the existence of hydraulic structure further upstream, so the reach transferred to supply-limited mode. The results of direct field measurements were compared with 22 mathematical procedures. In addition, this work represents the first attempt to combine the bed load to sediment transport rate in the Tigris River.

We think that this approach can be implemented on other rivers having the same characteristics of the River Tigris.

I hope that you can accept publishing your journal.

Thanking you in advance.

Best regards.

Nadhir Al-Ansari
Professor, LTU
Total Sediment Load in Tigris River at Baghdad City

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Abstract

The construction of hydraulic structures upstream Baghdad has limited the sediment supply to the river & reduced the water discharge of the river by 44%. The total load discharge rate in a reach that extend 18 km upstream the center of Baghdad was studied where 247 suspended load samples were collected from 16 cross sections. The results indicated that the suspended load is the dominant mode in the total load with minimum percentage 93.5%. After adding the bed load, the total load ranged from 29.1 to 190.3 kg/s. These measurements have been used to establish total load rating curve of power function that spatially reliable. The spatial distribution of sampled cross sections took into consideration the variance of river topography where 7 meanders, 2 islands & several bank depositions are characterized the geometry of the river. The associated errors from using the proposed rating curve are within reassuring levels & less than any errors produced from other twenty two total load predictors that were applied on the same reach. Scattering of the other predictors results can be attributed to the spatial variance in topography while it has less effect on the proposed rating curve. According to the final results obtained, it’s recommended to use the proposed procedure for establishing spatial total load rating curve for morphologically complicated rivers. The annual transported quantities of total load were estimated 2.47 & 4.23 million tons for 2009 & 2013 respectively.

Keywords: Sediment sampling, Total load, Sediment rating curve, Spatial variation, Supply-limited, Tigris River.

1. Introduction

Total sediment load transported in natural rivers is usually of two modes which are defined in three different criteria. Either by the type of movement, in contact with the bed as bedload or in suspension as suspended load, or by the method of measurement, measured load along
the water column & on the bed or unmeasured load at the lower zone above the bed where neither bedload sampler nor point sampler can measure, or by the source of sediment, wash load which are the finer sediment those suspended always & bed material load which can be suspended or move in contact with the bed (Julien, 2010). Suspended load is relatively finer sediment that are transported in suspension by the effect of flow turbulence while bedload, the coarser one, moves in contact with the river’s bed by sliding, rolling or saltation due to the boundary shear stress. An exchange may occur between suspended load & bedload, & between bedload & bed material depending on the size of the sediment particles, flow transport capacity, flow velocity, & boundary shear stress (Hickin, 1995; WMO, 2003). In addition, it is difficult to separate washload, which depends on upstream supply of sediment rather than transport capacity, & the suspended particles from bed material. The d_{10} of the bed material is commonly used as a cutoff between washload (d_i < d_{10}) & bed material load (d_i > d_{10}) (Julien, 2010).

The main sources of fluvial sediments are watershed erosion, stream erosion & human activities (Vanoni, 2006). Since sediment sources are unlimited & streams have sufficient sediment transport capacity, sediment transport will continue (Friend, 1993). Limited condition of the supply source of the fine sediment is the case in most natural rivers (Hickin, 1995), so in such rivers, bed-material is the main source of sediment load. The components of load transported in suspension, should be distinct from wash load, or even as bedload (Hickin, 1995; WMO, 2003).

Prediction of sediment load is of primary importance for river engineering & geomorphology (Recking, 2009) & it effects on driving fluvial processing (Cook et. al., 2013). Sediment rating curve is the common predictor that is used for estimating sediment transport as well as other prediction formulas. Sediment rating curve can be used for reconstructing long-term sediment transport records or compensating the missing in existing
sediment transport records (Walling, 1977b; Asselman, 2000) as well as using boundary condition for sediment load in morphological models. Whether using sediment rating curve or any other prediction formulas, they required field sediment measurements that can be used to determine rating coefficients or derive model formula.

Measuring total load requires measuring the two components of sediment, bedload & suspended load. While the former one can be measured by using different samplers, such as manually operated portable samplers, vortex tube, pit & trough (Diplas et. al., 2008). Different techniques can be used for measuring the suspended sediment along the water column, depth-integrating sampling or point-integrating sampling (Edwards & Glysson, 1998).

Tigris River is one of the most important rivers in the Middle East. It rises from Taurus Mountains range in the southeastern part of Turkey & flows toward the southeast for 1580 km passing through Turkish-Syrian border & entering Iraq until it combines with Euphrates River at Qurnah in the southern part of Iraq forming the River Shatt Al-Arab. This river discharges its water in the Arabian Gulf (Fig. 1.A).

Tigris River bisects Baghdad City, the capital of Iraq, into two parts for a distance of about 50 km within urban zone & 10 km within rural zones. The northern part of Tigris River reach is considered as study reach in this work. It’s about 18 km long & extends from Al-Muthana Bridge at the north to Sarai Baghdad gauging station at the center of Baghdad (Fig. 1.B). This segment of river is characterized by its single thread; compound meanders, & alluvial plain characteristics. The river banks are protected against erosion by aligned stones & cement mortar for the levels between 29 & 37 m.a.s.l. The dominant recent water levels within the reach are below all the protection levels (Ali, 2013).

The study reach has many new islands, side & point bars. They appeared in the river course mainly during the last two decades (Fig. 1.B). These obstacles in the river course have
negative impacts on the hydraulic performance of the river, such as reducing the flood capacity, limiting navigation & logging water intakes of water treatment stations as well as the environment & aesthetic impacts (Ali et al., 2014).

In this work, an attempt has been made to calculate the total load transport rate of Tigris River within the northern part of Baghdad directly by using field measurements & indirectly using mathematical formulas as well as improving spatial sediment rating curve.

2. Roles of Tigris River flow regulation on limitations of sediment sources

The main source of sediment for any natural river is the eroded soil from the watershed of the river & its tributaries as intermittent fluxes, as well as, the bed & the banks’ erosion along the river course. The quantity & properties of the fine sediment that are available to be transported depends on several factors & can be divided into two groups. The factors governing the capacity of the river to transport sediment are, channel geometry, alignment, slope, roughness, velocity distribution, turbulence, etc., which can affect all the transportation modes, while the factors governing the erodibility of sediment are topography of watershed, geology, climate, land use, soil type, particle size, etc.. These factors highly affect the transport of fine sediment (Julien, 2010).

The case of flow of Tigris River inside Baghdad is fully controlled by system of dams & regulators constructed on the main river & its tributaries upstream Baghdad (Fig. 2). This regulation disturbed the delivery of fine sediment from the head water. High trap efficiency for fine particles by dams can reach up to 95.33% at Mosul Dam reservoir on the main river (Issa, 2015).

The average monthly discharge of the river at Sarai Baghdad gauging station has decreased from 1207 m³/s for the period 1931-1959 to 927 m³/s for the period 1960-1999 when some dams were constructed (Fig. 3). Since 1980 onward, the fluctuation in river flow was damped
for the period 2000-2013, the average monthly discharge was 522 m$^3$/s (43% & 56% of the average flow of the two previous periods respectively).

Tigris River hydrograph (Fig. 4) shows the peak flow at Sarai Baghdad gauging station takes place during April & May. While now, the hydrograph is becoming flatter & flatter since 1990. This is due to flow regulation by the dams constructed at the head water as well as the draught period which is affecting the region due to climate changes (Al-Ansari et al., 2015).

In view of the above, limited sediment quantities reaches Baghdad city. The only source of sediment in the Tigris that reaches Baghdad is the area restricted from the lower sub-basin of Adhaim tributary & the catchment between Sammara Barrage (Fig. 2) as well as the bed & banks erosions. The delivery of fine sediment from Adhaim Tributary has not been measured, but a glance to its possible extra flow contribution (rather than flow released from Adhaim Dam) to Tigris River, can give an indication for estimated sediment delivery. The extra water flow contribution from Adhaim Tributary sub-basin & Tharthar Lake back feed to Tigris River was determined so that it will not exceed 260 m$^3$/s. In average, it was 8% of the average monthly discharge at Sarai Baghdad during 2004-2005, which was moderate year compared with recent more dry years (Ali et al., 2016). So with all these evidences, the study reach shall consider as supply-limited river at Baghdad.

3. River geometry & bed composite

The geometry of the study reach consists of a series of 7 meanders (see figure (1.B)) with radii of curvature ranged from 475m to 1245m (University of Technology, 1992). Along the 2$^{nd}$ & 3$^{rd}$ meanders there are two islands are grown & well as several banks depositions.

Morphology & bed sediment of the river inside Baghdad were investigated several times by Geohydraulique (1977), Al-Ansari & Toma (1984), Khalaf (1988), University of Technology (1992) & finally by Al-Ansari et al. (2015) where 46 bed samples were collected.
along the northern reach in Baghdad using van Veen grab. The grain size of the sediment was analysed using sieves & hydrometer test. It was noticed that fine sand was dominating the bed with an average median size of 0.178 mm. The size of the bed sediment relatively decreased compared to earlier investigations (see Al-Ansari & Toma, 1984). In addition, the sediments were moderately sorted, fine skewed & leptokurtic.

Geohydraulique (1977) concluded that Tigris River sediment is “bed-load”, since the suspended load concentrations never reach 3 g/l in high water & never exceed 0.2 g/l in low water periods.

4. Sediment sampling

As mentioned before, measuring the total load sediment required measuring both of the sediment components, those moving in contact with the bed & those moving in suspension along the water column. The two components of the total load were measured at 16 cross sections along the study reach (Fig. 5). Helley-Smith bedload sampler was used to measure the bedload. More details about the bedload are mentioned by Ali et al. (2016).

For the suspended load, pumping instrument was used to collect point-integrating samples at certain depths along the water column. Suction velocity was higher than the settling velocity of all particle sizes of bed material. The intake nozzle of pumping system installed on the top of HS sampler in a way similar to that used in Delft-Nile sampler (Van Rijn & Gaweesh, 1992) but with single intake nozzle.

Number of samples were selected at each water column depending on its height; the longer the height, the higher the number of samples as listed in Table (1).

The selection of sampling cross sections was chosen according to the morphological conditions such as existing of meander, island, sandbar & side bar within the river reach. Sampling was performed at the quartile (three verticals) for each cross section apart from
sections CS1 & CS4-2, & CS6-2 & CS6-3 where four & two verticals were measured respectively. Total number of collected samples was 212 of 500ml volume.

Number of verticals that were considered for each cross section was limited due to the shortage of available field work period. The security permission that the authors got to navigate the river & to conduct bathymetric survey & collect sediment samples was not enough to allow following the sampling procedure that proposed by Edwards & Glysson (1998) with high number of cross sections, as well as, focusing was on the spatial variation of sediment load more than the temporal variation. The difficulties faced in the field work were mainly related to safety & security issues.

Velocity profiles measurements were conducted in conjugation with sampling in all cross sections. Velocity profiles & river discharges were measured using FP111 Global Water Flow Probe & SonTek RiverSurveyor Acoustic Doppler Current Profiler (ADCP) (Figure 6).

5. Calculations of Sediment Load

Water samples were filtered in the lab using filter paper of 4 μm retention. The weights of filter papers were measured before using. Then the filter papers were dried at 70ºC after filtration of the samples. Precise balance of 4 decimal digits of a gram was used to weigh the dried filters. The extracted weights then were converted to sediment concentrations in units (g/l) or (kg/m³).

Using ADCP for measuring velocity profile gave the opportunity to measure the near bed velocity & even bed movement velocity (Atsuhiro et al., 2009). The velocity that was unmeasured from the water column is the most upper part because ADCP transducers cannot give reading in the final 11cm. On the other hand, the unmeasured zone of suspended sediment was the zone near the bed where the sampler could not take a sample. To compensate the unmeasured zone, an extrapolation technique proposed by van Rijn (1993) was applied to complete the velocity profile & the sediment concentration profile. According
to this extrapolation technique, the velocity at the water surface was considered equal to the closest measured velocity on the water column (see Fig. 7). The lower part of sediment concentration profile was extrapolated by three methods & the average was considered. These three extrapolations are described as follow:

1. The sediment concentrations between the bed & the first sampling point were assumed to be equal to that in the first sampling point.

2. The sediment concentrations between the bed & the first sampling point were computed by the power form $c = A \times Y^B$

   where:

   $Y : (h-z)/z = \text{dimensionless vertical coordinate},$

   $z : \text{vertical coordinate above bed},$

   $h : \text{water depth},$

   $A, B$ are coefficients determined by applying a regression method on the measured concentrations of the first three sampling points above the bed.

   a. Selecting $B = 0.1$ to 5 varied by step 0.1

   b. Computing $A = \frac{1}{3} \sum Y^B \times c_i / \sum (Y^B Y^B)$ .........................................................(1)

   c. Computing $T = \sum (AY^B - c_1)$ .................................................................(2)

   d. Repeating the procedure for all the range of $B$. The $A$ & $B$ coefficients corresponding to a minimum $T$ -value are selected as the "best" coefficients.

3. The sediment concentrations between the bed & the first sampling point were computed by the exponential form $c = e^{Ez + F}$

   where:

   $z : \text{height above bed}$
\( E, F \) are coefficients determined by applying a linear regression method on the measured concentrations of the first three sampling points above the bed.

\[
E = \frac{3 \sum_{i=1}^{3} (z_i \ln c_i) - 3 \sum_{i=1}^{3} (z_i) \sum_{i=1}^{3} (\ln c_i)}{3 \sum_{i=1}^{3} (z_i z_i) - \left( \sum_{i=1}^{3} z_i \right)^2} \quad \text{............................................... (3)}
\]

\[
F = \frac{3 \sum_{i=1}^{3} (z_i z_i) \sum_{i=1}^{3} (\ln c_i) - 3 \sum_{i=1}^{3} (z_i) \sum_{i=1}^{3} (z_i \ln c_i)}{3 \sum_{i=1}^{3} (z_i z_i) - \left( \sum_{i=1}^{3} z_i \right)^2} \quad \text{............................................... (4)}
\]

The depth-integrated suspended sediment load \((g_s)\) per unit width was computed as:

\[
g_s = \sum_{i=1}^{N} (v_i c_i + v_{i_{-}} c_{i_{-}})(z_i - z_{i_{-}})/2 \quad \text{............................................... (5)}
\]

where:

\( v_i \): fluid velocity at height \( z_i \) above the bed,
\( c_i \): sediment concentration at height \( z_i \) above the bed,
\( N \): total number of points along the water column including extrapolated & interpolated values.

Figure (8) shows the velocity & sediment concentration profiles at some cross sections.

The measured suspended sediment discharges \((G_s)\) along the river reach were tabulated in Table (2). By combining the measured bedload discharges \((G_b)\) (Ali et al., 2016) to the measured suspended loads, the total sediment loads \((G_t)\) were determined at the cross sections along the river study reach.

The average suspended load was 102.25 kg/s while the maximum load was 190.3 kg/s at CS2 associated with water discharge of 459 m\(^3\)/s & the minimum load was 29.1 kg/s at CS3 associated with water discharge of 464.4 m\(^3\)/s.
The ratios of bedload to suspended load were also mentioned in table (2). The maximum, average & minimum ratio values were 6.5, 2.4 & 0.63 respectively. These ratios indicate that suspended load is the vast dominant sediment load along the study reach & possible exchange between the bedload & the suspended load occurs depending on the hydro-morphological conditions at each cross section. For instance, the cross sections having relatively high bedload ratio are either an expanded section coming downstream to a contraction; such as between CS5 & CS6-1, or due to dredging operations for deepening the sections; such as CS3, or a section of pool shape where high depositions occurred on the inner bank due to the spiral flow; such as CS13. River geometry is an influencing factor affecting the spatial distribution of the total load, as well as, the spatial distribution of flow velocities.

6. Sediment Rating Curve

The usual procedure that to establish a sediment rating curve is by collecting sediment samples at wide range of different discharges at certain location on the river course, & then using one of the regression techniques to determine the best coefficients of the rating equation which is usually of power function form (Walling, 1977a; Fenn et.al., 1985). Such rating curve may be good representative for sediment transport at the sampling location, but there is no guarantee that it will be as good as it is if used in other locations of different morphology.

Looking for sediment rating curve that is spatially reliable along a river reach of complicated morphology, such as meanders & islands, is more required from view of river engineering & modelling specially when there are evidences on sedimentation processing going on along that river reach. Such spatial sediment rating curve can be used as sediment inflow or outflow boundary condition in hydro-morphological models at any cross section along the reach. Instead of repeating sediment measurements at one location for a period of time, sediment samples are to be collected at several locations along the river segment those can represent the spatial variance in the topography, such as meandering, riffle-pool, or
island, for that river segment on condition that there is no tributary, distributary or regulator along that river segment. On the other hand, sampling time period has considered covering an acceptable range of water discharges from hydrological perspective.

Regression sediment rating curve was established previously for Tigris River inside Baghdad by many authorities using the historical records of suspended load at three gauging stations (Sarai Baghdad, north of Baghdad & south of Baghdad) when none of the dams was constructed yet on the main river in Iraq (Geohydraulique, 1977, Al-Ansari et.al., 1979). After starting to impound operation of Mosul Dam reservoir on Tigris River in 1986, a modification was applied on the sediment rating curve by Khalaf (1988) depending on more measurements in north of Baghdad gauging station. All these rating curves took the form of power function.

By adding the current measurements of total load to the previous measurements available by Khalaf (1988), new rating curve coefficients of power function were determined (Figure 9). The new rating curve function is represented in Eq. (6) with determination coefficient ($R^2$) equal to 0.7963

$$Q_t = 0.0002Q_w^{2.1312}$$ ................................................................. (6)

Where

$Q_t$: total sediment discharge (kg/s)

$Q_w$: water discharge (m$^3$/s)

Errors produced from using sediment rating curve were calculated as percentages of the values from measured data as expressed in Eq. (7) (Walling, 1977a).

$$\text{Error}(%)=\left(\frac{\text{RatingCurveEstimation}}{\text{MeasuredBedload}}-1\right) \times 100$$ ......................................................... (7)

The errors produced from applying (Eq. (6)) at the same sampling locations along Tigris River reach are shown in Fig. (10). 26.66% of the estimated bedload have an error ranged between +10% & -10% & accumulated 46.66% of the estimations have errors ranged between
-25% & +25%, & accumulated 66.66% of the estimations have errors ranged between -50% & +50%, & accumulated 80% of the estimations have errors ranged between -100% & +100%, then 20% have error higher than +100%.

7. Calculation of Total Load Using Formulas

Wide spectrums of total load or bed sediment load predicting formulas were proposed & developed by many researchers depending on different approaches. For each approach, a specified concept was considered as motivation for deriving the approach’s formula & a certain number of parameters were controlled in the lab measurements to estimate the constants for the formula. Brief descriptions for the following approaches are given below:

7.1 Unit stream power concept

The rate of work being done by a unit weight of water in transporting sediment must be directly related to the rate of work available to a unit weight of water (Yang, 1996). Yang emphasized the power available per unit weight of fluid to transport sediments.

1. Yang formula (1973) (Yang, 1996)

7.2 Shear stress approach

The movement of bed material particles will start when the criteria of incipient motion is exceeded. So, shear stress near the bed will entrain the sediment particles to motion as long as the shear stress is greater than the critical shear stress of the particles. The following formulas which belong to this approach were used in this work:

1. Laursen formula (1958) (Vanoni, 2006)
2. Chang-Simons-Richardson formula (1965) (Yang, 1996)
7.3 Probabilistic approach

Probability concepts were introduced in bedload prediction by the pioneer work of Einstein in 1942. The turbulent flow fluctuations are the driver for sediment entrainment rather than the flow forces exerted on the particle. Both of the entrainment & the deposition were expressed in probability terms (Yang, 1996).

1. Einstein bedload function (1950) (Graf, 1971)
2. Garde-Dattatri formula (1963) (Garde, 2006)
3. Colby formula (1964) (Yang, 1996)

7.4 Regression approach

Data driven models (regression, ANN) were used to explain the bedload transport process due to the limitations of defining this complex process into precise formula (Taludar et al., 2012). The following formulas were used within this approach:

1. Shen-Hung formula (1972) (Yang, 1996)

7.5 Power Concept

This approach has developed from the concept that there is a relation between the available energy to the river with the rate of work done by the river to transport sediment (Yang, 1996). The following formula was used within this approach:

3. Ackers-White formula (1973) (Yang, 1996)


7.6 Regime approach

This approach was developed based on the regime theory where the data used for establishing its relations were taken from large stable irrigation canals (Vanoni, 2006).

1. Inglis-Lacey formula (1968)

These twenty two total load formulas were applied on the study reach to predict the total sediment load discharge to find the best suitable formulas.

8. Application of Total Load Formulas

Two kinds of data sets are required to apply total load formulas, physical properties of river bed sediment & hydraulic-geometric parameters of the study reach. The bed sediment properties includes; density, specific weight, specific gravity, median diameter & $d_{90}$ (Vanoni, 1975, Graf, 1971). All these variables were measured by sampling bed materials using van Veen grab along the study reach & analysed by sieves & hydrometer, most of these results were published by Al-Ansari et al. (2015). The hydraulic-geometric parameters included; water depth, cross sectional area, top width, wetted perimeter, hydraulic radius, water surface slope & water discharge (Vanoni, 1975, Graf, 1971). These parameters were extracted from field measurements along the study reach as well as water density & salinity. All the values of bed sediment properties & the hydraulic-geometric parameters are tabulated in table (3).

The results of applying the twenty two total load formulas along sixteen cross sections within the study reach are displayed in Figure (11). This figure shows the computed total load discharges from the formulas & the total load discharges in the cross sections.

To evaluate which formula can give the best prediction of total load discharge compared with actual field measurements; discrepancy ratio & error percentage (Eq. (7)) were used to scale the precision of the predicted total load discharges. The discrepancy ratio ($r$) is the ratio
of the predicted to the measured transport rate (van Rijn, 1993). The perfect agreement between predicted & measured sediment rates is when $r = 1$ (0% error). Three zones of different discrepancy ratios were added in Figure (11) to explain the distribution of the compared results around the prefect agreement line.

Many of the formulas predicted considerably higher or lower sediment discharges, while three formulas predicted the total load discharges closer to the actual measured. These formulas are Colby formula (1964), Brownlie formula (1981) & Guo-Julien formula (2004) with average discrepancy ratios of 1.17, 0.74 & 1.42 (errors 17%, -26% & 42%) respectively.

The predictions of Colby formula were distributed on both sides of the perfect agreement line where 64.7% of the estimations are within the first discrepancy zone $0.5 < r < 2$ (-50% < error < +100%) & accumulative 94.1% of Colby formula estimations are within the second discrepancies zone $0.25 < r < 4$ (-75%<error<+300%). In addition, same distribution for the predictions of Brownlie & Guo-Julien formulas with lower $r$ values were recognized. Table (4) shows the percentage of the predicted sediment discharges according to each range of the discrepancy ratio (error percentage). The higher percentage of data within the closer range of discrepancy ratio $0.75 \sim 1.25$ (error -25% ~ +25%) was equally between Colby & Brownlie & the results continued on this manner approximately. Some of other predictors have limited results within the discrepancy zone $0.5 \sim 2$ (error -50% ~ +100%), such as Garde-Dattatri, Graf-Acaroglu, Einstein, Toffaleti & Chang et al., but their results percentages within the zone are still less than 50%.

To explain the scattering behavior of the total load formulas at different cross sections along the study reach, they were applied at those cross sections for a range of discharges between 400 & 700 m$^3$/s. Figure (12) shows the results of the predictors at eight cross sections (CS1, CS3, CS6-1, CS6-4, CS7, CS9, CS11 & CS14) & it seems from the figure that the behavior of most of the total load formulas is consistent except Guo-Julien formula where
it has two slopes or more in some cross section such as CS1, CS6-1 & CS6-4, steep slope for low flow until 500 m$^3$/s then mild slope for higher flow. The variation of equivalent water depth or hydraulic radius versus water discharge is not consistent at all cross sections as well as flow velocity which leads to such results for the more sensitive formulas. According to the results (Fig. 12), it is hard to say that there is a unique prediction formula that can predict total sediment discharge along the whole study reach with stable magnitude of error. The important conclusion from these results is, even those formulas have results compatible with field measurements in regular cross sections, there is no guarantee to get the same compatibility at irregular cross sections (meanders, sand bars, etc.). So, the total sediment rating curve is still the closer predictor to the measurements.

Annual total load quantities were computed using all predictors for the period 2009-2013 along the study reach were listed in Table (5) with the associated discrepancy ratios to the sediment rating curve in Table (6). The results indicate that Colby (1964) formula has the closest discrepancy ratio to unity in average followed by Guo-Julien (2004) & Brownlie (1981) formulas respectively. According to the total load rating curve, the annual total load quantities varied between 2 to 4.23 million ton in 2009 & 2013 respectively while they are of magnitude 2.85 & 3.87 million ton respectively according to Colby (1964) formula.

9. Conclusions

The applied hydrological controlling schemes on Tigris River decreased the average monthly discharge at Sarai Baghdad gauging station to 522 m$^3$/s during the period 2000-2013. This reduction represents 43% & 56% of what it was during 1930-1959 & 1960-1999 respectively.

The river reach within Baghdad is considered as supply-limited from sediment supplying view point since most of fine sediment is trapped now by dams & regulators upstream of
Baghdad. Limited local sources & the lower sub-basin of Adhaim tributary, can supply fine sediment to the river reach during rainy seasons or high flow. Other sources are bed & banks erosion along the reach downstream Sammara Barrage.

The geometry of the study reach consists of a series of 7 meanders & two islands with many depositions on the banks. This complicated geometry made finding a unique total load formula that can represent the sediment transport along the study reach rather difficult. The river bed mainly consists of fine sand according to the analysis of the bed material samples.

The suspended load that was measured using 212 samples of 500ml volume was the dominant component in the total load where the average total load discharge was 102.25 kg/s, while the maximum & minimum were 190.3 & 29.1 kg/s respectively. The current total load is consistent with previous measurements during low water flow.

The measured total loads at 16 cross sections were used to fit sediment rating curve of power function form. The new rating curve is slightly different from the previously fitted in 1988. The advantage of this rating curve is spatially reliable along spatially topographically variant reach when there are evidences on sedimentation processing are going on along that reach.

The errors produced from applying the new rating curve ranged between +10% & -10% for 26.66% of the cross sections & accumulatively ranged between +50% & -50% for 66.66% of the cross sections. The scattering in some measurements may be attributed to the spatial variation in river morphology.

Between the twenty two total load formulas that were applied along the study reach to find the best predictor, three formulas gave results close to the measured. The closer predictors were Colby (1964), Brownlie (1981) & Guo-Julien (2004) with average discrepancy ratios of values 1.17, 0.74 & 1.42 respectively.
The annual total load quantities estimated by Colby (1964) ranged between 2.85 to 3.87 million ton in 2009 & 2013 respectively compared with 2.47 million ton in 2009 to 4.23 million ton in 2013 according to the total load rating curve.

In general, the results indicate that the spatial total load rating curve is better than other predictors for morphologically complicated rivers.

10. References


Atsuhiro, Y., Shintaku, S., Ejima, K., Fukami, K., & Kanazawa, H. (2009). DEVELOPMENT OF A SEDIMENT DISCHARGE MEASUREMENT SYSTEM WITH ADCP, 10th International Conference on Fluid Control, Measurements, & Visualization, August 17-21, Moscow, Russia.


Figure 1. (A) Map of Iraqi Provences (B) Tigris River inside Baghdad, capital of Iraq (islands and sandbars bordered by red) (Ali et al., 2014, 2015)

Figure 2. Schematic Diagram of Tigris River Hydrological Scheme (MWR, 2005).

Figure 2. Schematic Diagram of Tigris River Hydrological Scheme (MWR, 2005).
Figure 3. Mean monthly flow of the River Tigris at Sarai Baghdad for the period 1931-2013 (Al-Shahrabaly, 2008).

Figure 4. Decadal hydrographs of River Tigris at Sarai Baghdad for the period 1930-2013 (Al-Shahrabaly, 2008).
Figure 5. Total load sampling cross sections along the northern reach of Tigris River inside Baghdad City.

Figure 6. ADCP velocity distribution at (A) CS1, (B) CS14.
Figure 7. Velocity profile and sediment concentration profile complements (van Rijn, 1993).

Figure 8. Velocity and sediment concentration profiles at (A) CS1-60, (B) CS6-3-L, (C) CS7-C, (D) CS9-L.
Figure 9. Total load rating curve along the northern reach of Tigris River inside Baghdad City.

Figure 10. Comparison of estimated total load using sediment rating curve with measured total along Tigris River reach inside Baghdad with percentage of associated error.
Figure 11. Comparison of total load discharges predicted by different twenty two sediment transport formulas with measured total load in Tigris River.
Figure 12. Application of total load formulas at different cross sections for a range of discharges.
### Table 1. Criteria for selecting number of suspended load samples and their position in water column [Al-Ansari, 2005]

<table>
<thead>
<tr>
<th>Water depth (m)</th>
<th>Number of samples</th>
<th>Depths of the samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3 &lt; h &lt; 0.6</td>
<td>Single sample</td>
<td>0.6h</td>
</tr>
<tr>
<td>0.6 &lt; h &lt; 3.0</td>
<td>2 samples</td>
<td>0.2h, 0.8h</td>
</tr>
<tr>
<td>3.0 &lt; h &lt; 6.0</td>
<td>3 samples</td>
<td>0.2h, 0.6h, 0.8h</td>
</tr>
<tr>
<td>6.0 &lt; h</td>
<td>6 samples</td>
<td>0h, 0.2h, 0.4h, 0.6h, 0.8h, 1.0h</td>
</tr>
</tbody>
</table>

### Table 2. Calculations of the measured total sediment load along the study reach

<table>
<thead>
<tr>
<th>Cross-section</th>
<th>$G_s$ (kg/s)</th>
<th>$C_{rms}$</th>
<th>$G_b$* (kg/s)</th>
<th>$G_t$ (kg/s)</th>
<th>$G_b$ / $G_s$ (%)</th>
<th>$Q_w$ (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>112.430</td>
<td>246.68</td>
<td>1.268</td>
<td>113.698</td>
<td>1.13</td>
<td>457.38</td>
</tr>
<tr>
<td>CS2</td>
<td>190.310</td>
<td>415.99</td>
<td>2.590</td>
<td>192.900</td>
<td>1.36</td>
<td>459.02</td>
</tr>
<tr>
<td>CS3</td>
<td>29.100</td>
<td>62.89</td>
<td>1.341</td>
<td>30.441</td>
<td>4.61</td>
<td>464.41</td>
</tr>
<tr>
<td>CS4</td>
<td>92.160</td>
<td>207.79</td>
<td>0.682</td>
<td>92.842</td>
<td>0.74</td>
<td>445.10</td>
</tr>
<tr>
<td>CS4-2</td>
<td>136.330</td>
<td>302.44</td>
<td>1.142</td>
<td>137.472</td>
<td>0.84</td>
<td>452.33</td>
</tr>
<tr>
<td>CS5</td>
<td>37.580</td>
<td>77.10</td>
<td>1.599</td>
<td>39.179</td>
<td>4.25</td>
<td>489.23</td>
</tr>
<tr>
<td>CS6-1</td>
<td>44.620</td>
<td>81.44</td>
<td>2.901</td>
<td>47.521</td>
<td>6.50</td>
<td>549.88</td>
</tr>
<tr>
<td>CS6-2</td>
<td>31.130</td>
<td>109.09</td>
<td>0.924</td>
<td>32.054</td>
<td>2.97</td>
<td>286.41</td>
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<tr>
<td>CS6-3</td>
<td>30.330</td>
<td>121.27</td>
<td>0.190</td>
<td>30.520</td>
<td>0.63</td>
<td>251.02</td>
</tr>
<tr>
<td>CS6-4</td>
<td>96.620</td>
<td>172.61</td>
<td>3.420</td>
<td>100.040</td>
<td>3.54</td>
<td>561.78</td>
</tr>
<tr>
<td>CS7</td>
<td>180.610</td>
<td>278.10</td>
<td>1.564</td>
<td>182.174</td>
<td>0.87</td>
<td>651.71</td>
</tr>
<tr>
<td>CS8</td>
<td>143.980</td>
<td>224.60</td>
<td>3.938</td>
<td>147.918</td>
<td>2.74</td>
<td>643.32</td>
</tr>
<tr>
<td>CS9</td>
<td>128.670</td>
<td>243.42</td>
<td>2.171</td>
<td>130.841</td>
<td>1.69</td>
<td>530.44</td>
</tr>
<tr>
<td>CS10</td>
<td>145.220</td>
<td>253.90</td>
<td>2.716</td>
<td>147.936</td>
<td>1.87</td>
<td>573.97</td>
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<tr>
<td>CS11</td>
<td>138.600</td>
<td>240.48</td>
<td>2.630</td>
<td>141.230</td>
<td>1.90</td>
<td>578.38</td>
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<td>CS13</td>
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<td>137.68</td>
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<td>75.279</td>
<td>3.55</td>
<td>529.97</td>
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<tr>
<td>CS14</td>
<td>127.890</td>
<td>245.76</td>
<td>1.930</td>
<td>129.820</td>
<td>1.51</td>
<td>522.23</td>
</tr>
</tbody>
</table>

* Bedload discharges have been taken from [Ali et al., 2016]
Table 3. Bed sediment properties and hydraulic-geometric parameters along study reach during measurement’s period.

<table>
<thead>
<tr>
<th>C.S.</th>
<th>$d_{50}$ (mm)</th>
<th>$d_{90}$ (mm)</th>
<th>Cross-sectional Area ($m^2$)</th>
<th>Top Width (m)</th>
<th>Hydraulic Radius (m)</th>
<th>Discharge ($m^3/s$)</th>
<th>Water velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS1</td>
<td>0.194</td>
<td>0.273</td>
<td>664.9</td>
<td>180.03</td>
<td>2.98</td>
<td>457.381</td>
<td>0.688</td>
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<tr>
<td>CS2</td>
<td>0.166</td>
<td>0.235</td>
<td>653.7</td>
<td>260.77</td>
<td>2.471</td>
<td>459.022</td>
<td>0.702</td>
</tr>
<tr>
<td>CS3</td>
<td>0.1755</td>
<td>0.25</td>
<td>795.2</td>
<td>261.6</td>
<td>3.008</td>
<td>464.409</td>
<td>0.584</td>
</tr>
<tr>
<td>CS4</td>
<td>0.199</td>
<td>0.273</td>
<td>691.5</td>
<td>250.06</td>
<td>2.743</td>
<td>445.095</td>
<td>0.644</td>
</tr>
<tr>
<td>CS4-2</td>
<td>0.197</td>
<td>0.276</td>
<td>745</td>
<td>241.09</td>
<td>3.013</td>
<td>452.325</td>
<td>0.607</td>
</tr>
<tr>
<td>CS5</td>
<td>0.208</td>
<td>0.278</td>
<td>643.3</td>
<td>151.67</td>
<td>4.072</td>
<td>489.233</td>
<td>0.76</td>
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<tr>
<td>CS6-1</td>
<td>0.199</td>
<td>0.273</td>
<td>865.2</td>
<td>353.85</td>
<td>2.398</td>
<td>549.877</td>
<td>0.636</td>
</tr>
<tr>
<td>CS6-2</td>
<td>0.21</td>
<td>0.275</td>
<td>421.284</td>
<td>185.2</td>
<td>2.271</td>
<td>286.409</td>
<td>0.68</td>
</tr>
<tr>
<td>CS6-3</td>
<td>0.145</td>
<td>0.255</td>
<td>369.83</td>
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<td>4.113</td>
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<tr>
<td>CS6-4</td>
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<td>0.27</td>
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<td>237.8</td>
<td>3.141</td>
<td>561.778</td>
<td>0.739</td>
</tr>
<tr>
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<td>0.277</td>
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<td>320.08</td>
<td>2.881</td>
<td>651.709</td>
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<tr>
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<td>0.275</td>
<td>979</td>
<td>236.86</td>
<td>4.053</td>
<td>643.319</td>
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<tr>
<td>CS9</td>
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<td>0.218</td>
<td>772.9</td>
<td>255.5</td>
<td>2.911</td>
<td>530.443</td>
<td>0.686</td>
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<tr>
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<td>0.243</td>
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<td>578.375</td>
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<td>114.9</td>
<td>5.889</td>
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</tr>
<tr>
<td>CS14</td>
<td>0.135</td>
<td>0.213</td>
<td>711.4</td>
<td>137.84</td>
<td>4.976</td>
<td>522.226</td>
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Table 4. Percentage of predicted total load discharges for each range of the discrepancy ratio

<table>
<thead>
<tr>
<th>Formulas</th>
<th>Accumulative of data percentage in each range of the discrepancy ratio or corresponding error percentage</th>
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<td></td>
<td>0.75 ~ 1.25</td>
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<td>5.88</td>
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<tr>
<td>Garde-Dattatri 1963</td>
<td>23.53</td>
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<tr>
<td>Graf-Acaroglu 1968</td>
<td>17.65</td>
</tr>
<tr>
<td>Yang 1973</td>
<td>0</td>
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<tr>
<td>Yang 1979</td>
<td>0</td>
</tr>
<tr>
<td>Ackers-White 1973</td>
<td>0</td>
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<td>Ackers-White 1990</td>
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<td>Toffaleti 1969</td>
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<tr>
<td>Shen-Hung 1972</td>
<td>0</td>
</tr>
<tr>
<td>Maddock 1973</td>
<td>0</td>
</tr>
<tr>
<td>Maddock 1976</td>
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</tr>
<tr>
<td>Bagnold 1966</td>
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<tr>
<td>Colby 1964</td>
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<td>Chang et al. 1965</td>
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</tr>
<tr>
<td>van Rijn 1984</td>
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</tr>
<tr>
<td>Guo-Julien 2004</td>
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</tr>
<tr>
<td>Einstein 1956</td>
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</tr>
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<td>Simons et al. 1981</td>
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<tr>
<td>Brownlie 1981</td>
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<td>Inglis-Lacey 1968</td>
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Table 5. Annual predicted total load quantities for the period 2009-2013.

<table>
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<tr>
<th>Year</th>
<th>2009</th>
<th>2010</th>
<th>2011</th>
<th>2012</th>
<th>2013</th>
<th>Average discrepancy ratio</th>
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<tbody>
<tr>
<td>Rating Curve (ton/yr)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laursen 1958</td>
<td>902713</td>
<td>1109429</td>
<td>1003976</td>
<td>1150377</td>
<td>1368471</td>
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<td>909120</td>
<td>824146</td>
<td>948184</td>
<td>1111181</td>
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</tr>
<tr>
<td>Garde-Duttatri 1963</td>
<td>21554748</td>
<td>26841666</td>
<td>24112350</td>
<td>27893106</td>
<td>33173747</td>
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<td>Graf-Acaroglu 1968</td>
<td>11220618</td>
<td>13320806</td>
<td>12314328</td>
<td>1339079</td>
<td>15677470</td>
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<td>Yang 1972</td>
<td>379054</td>
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<td>427143</td>
<td>499774</td>
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<td>568508</td>
<td>666840</td>
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Three-Dimensional Morphodynamic Modelling of Tigris River in Baghdad

River Research and Applications

Three-Dimensional Morphodynamic Modeling of Tigris River in Baghdad

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7 Corresponding author

ABSTRACT

Bathymetric and land surveys were conducted for the northern Tigris River reach (18 km length) in Baghdad, producing 180 cross sections. A river bed topography map was constructed from these cross sections. The velocity profiles and the water discharges were measured using ADCP at 16 cross sections, where intensive number of sediment samples was collected to determine riverbed characteristics and sediment transport rate. The three-dimensional morphodynamic model (SSIIM) was used to simulate the velocity field and the water surface profile along the river reach. The model was calibrated for the water levels, the velocity profiles and the sediment concentration profiles using different combinations of parameters and algorithms. The calibration and the validation results showed good agreement with field measurements, and the model was used to predict the future changes in river hydro-morphology for a period of 14 months. The results of the future predictions showed the Tigris River behaved like an underfit river, increases in depositions on the shallow part of the cross section having lower velocity and, and the river deepens the incised route to fit its current hydrologic condition leaving the former wide section as a floodplain for the newer river. The net deposition/erosion rate was 67.44 kg/s in average and the total deposition quantity was 2.12 million ton annually. An expansion in the size of current islands was predicted. An indication of the potential threats of the river banks’ collapse and the bridge piers’ instability was given by high erosion along the thalweg line.

Keywords: 3D modeling, SSIIM, bed changes, sediment transport, ADCP velocity measurements, sand bed, under-fit river, Tigris River.

INTRODUCTION

Estimating erosion and sedimentation rates and their locations along river’s reach can be as important as estimating flow and velocity for preparing future plans. These estimations require a good knowledge of the effective variables that control the processes, as well as understanding the relationships which control these variables. Discharge, flow velocity, bed material, sediment supply and channel slope are the major effective variables that control the erosion and sedimentation processes.

In the past, estimation of velocity was achieved by using physical models jointly with field measurements. However, after the great development in computational methods and computer capabilities, simulation models became popularly used for this purpose. Usually simulation models use some of the well-known equations such as Navier-Stokes equations established based on conservation concepts such as conservation of mass and momentum to govern the

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relationships between the control variables in the treated phenomena. Moreover, simulation models started coupling many models together (such as hydrodynamic, sediment transport and morphology models) to reach for the integrity in the subject of erosion and sedimentation.

Simulation models are powerful and relatively inexpensive tools for predicting future morphological changes, evaluating alternatives of river control works, and environmental management of the river. The reliability and the accuracy of a simulation model’s performance in reproducing field conditions of real-life problems in the model are questionable unless they are subjected to calibration and validation processes to adjust to the parameters and the algorithms of the model and re-examining the results for different site conditions.

Field investigations and measurements are necessary to provide simulation models with different sets of values for the important variables in the model, such as flow velocity, water level and sediment concentrations. These variables are to be used for comparisons with model results during calibration and validation steps.

The SSIIM (Simulation of Sediment movements In water Intakes with Multiblock option) model is a three dimensional morphodynamic model that is used to simulate velocity field, water surface profile, sediment transport, morphological changes as well as pollution dispersion in rivers and reservoirs with high flexibility for dealing with complex geometries. It couples several models (convection-diffusion, transport-dispersion and sediment continuity models) with geometry editor and input file for pre-processing step and graphical display package for post-processing step (Olsen, 2014).

In this research, 18 km reach of the River Tigris was studied and the data were used in SSIIM 3-D model to predict future changes expected in the reach.

*Tigris River*

Tigris River bisects Baghdad, the capital of Iraq, into two parts for a distance of about 50 km within the urban zone starting from Al-Muthana Bridge to the north and ending at the confluence with the River Diyala to the south. This river reach has single thread, compound meanders, and alluvial plain characteristics. The river banks are protected against erosion for 66% of the length in the urban zone by aligned stones and cement mortar between levels 29 and 37 m.a.s.l. at the start of the reach and drop gradually to the south. Recently, the dominant water levels in the reach are below the protection level.

The flow of the river is fully controlled in Baghdad by a system of dams and regulators constructed on the main river and the tributaries upstream of Baghdad. The river discharge was reduced 44% of previous period. The discharge dropped to 522 m³/s as a monthly average flow of the period (2000-2013). The flow peaks, when most of the sediment was transported during them (Al-Ansari et al., 1979), had been vanished due to the effect of the regulating schemes in Iraq and Turkey as well as climate change where a drought period is affecting the region (Al-Ansari, 2013 and 2016; Al-Ansari et al., 2014 and 2015a).

During the last two decades many new islands, side depositions and point bars appeared in the Tigris River’s reach in Baghdad (Fig. 1). Eighteen obstacles between islands and sandbars were recognized inside Baghdad in 2008 (Ali, 2013; Ali et al., 2012 and 2014). Dredging operations have been started at specified sites along the river inside Baghdad to overcome the
sedimentation problems. However, deposition processes are still active along the river and some of completed dredging sites need to dredge again. Accordingly, the estimation for sedimentation rate and predicting the future changes in the riverbed topography of Tigris River are required.

The northern reach of Tigris River in Baghdad of length 18 km was considered as a study reach in this work. It extends from Al-Muthana Bridge at north of the city until Sarai Baghdad gauging station at the Center of the city (see Figure 1). The aims from this work are:

1. Investigating the zones of erosion and sedimentation along the study reach.
2. Determining the rate of sedimentation in the study reach.
3. Predicting the future changes in the riverbed topography along the study reach.

To achieve these aims, the SSIIM was used for simulating the flow field and sediment transport during calibration and validation steps by the assistant of field measurements of water levels, velocity distribution and sediment load, which are preparing the model for predicting the future changes of bed topography.

RIVER BATHYMETRY AND FIELD MEASUREMENTS

The study reach consists of a series of 7 meanders (Fig. 2) of radii of curvature that are ranging from 475m to 1245m (University of Technology, 1992). Two islands existed in the river reach. The smaller one is located in the upstream of CS4 directly and the second (is the larger) is within CS6-1 and CS6-4.

Bathymetric survey was conducted to the riverbed and land survey to the river banks at 180 cross sections along the reach to produce high quality topographical map for the river course as shown in Figure (3). The riverbed elevations ranged between 13.5 and 28.76 m.a.s.l. The deepest locations are on the outer banks of the meanders in the part directly downstream from the peaks. The map showed depositions near water surface at level between 27 and 28.76 m.a.s.l.

Using the van Veen grab, 46 bed sediment samples were extracted along the reach to determine the characteristics of the material on the bed. Fine sand is the dominant on the riverbed and 9 particle sizes were recognized in the whole reach ranging from clay to medium sand. The results of the analysis of the bed sediment samples were published by Al-Ansari et al. (2015b).

Water discharge and velocity distribution were measured in 16 cross sections (see Fig. 2) using SonTek RiverSurveyor M9 ADCP with repetitions for more accuracy. Additional measurements for flow velocities were measured at certain depths of shallow water using FP111 Global Water Flow Probe.

Total load transport rates were determined in 16 cross sections by collecting 288 bedload samples using the Helley-Smith sampler and 212 suspended load samples suction pump, and then combining the analysis results to get the total load. A total load rating curve (Eq. 1) was established using the total load transport rates. The details of the analysis of the sediment samples were published by Ali et al. (2016a, 2016b).

\[
G_t = 0.0002Q^{2.1312} \quad ................................ (1)
\]

Where
\[
G_t : \text{total sediment discharge (kg/s)}
\]
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\( Q_w \) : water discharge (m\(^3\)/s)

THE SSIIM MODEL

The using of the SSIIM model required preparing several issues before starting to run the model. These issues can be categorized as follow:

1. Geometry
   a. Geometry grid
   b. Bathymetry of the river

2. Hydrodynamic
   a. Initial water surface profile
   b. Boundary conditions
   c. Bed composition and roughness

3. Morphology
   a. Sediment fractions
   b. Sediment boundary and transport

4. Calibration and validation
   a. Water levels
   b. Flow velocities
   c. Sediment concentrations

**Model Geometry**

A geometry grid was prepared for the river bed and lower banks. The size of the grid was chosen in a way to make a balance between the accuracy of the grid to represent the details of the complicated topography of the river as well as satisfying the stability requirements of the model solution and between computational time’s consumption. The horizontal size of the grid was 90000 cells and the length to width ratio of the grid cells was not larger than 3. The bathymetric and land surveys’ data was converted to geometry points and interpolated over the constructed grid.

**Hydrodynamic and Morphology**

An initial water surface profile was considered from previous simulation for 1-D flow in the river for a discharge close to the average monthly flow (Ali et.al., 2012), then the model adjusted the water profile for the current hydraulic conditions using free surface algorithms. The upstream boundary condition was dependant on water inflow while the downstream boundary condition was dependant on specifying the water level corresponding to water inflow.

The results of the analysis for the bed material samples were used to prepare a distribution map for bed sediment fractions for all bed grid cells. The bed roughness was either calculated from Manning’s roughness coefficient \((n)\) or from \(d_{50}\) of particle size distribution and bedform height based on the sediment fractions map for bed grid cells.

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Settling velocities for the sediment fractions in the riverbed were determined using Rubey settling velocity for the sizes up to 50\(\mu\)m, for smaller sizes, Stokes settling velocity was used as shown in Table (1).

The total load rating curve (Eq. 1) was used to specify the sediment inflow.

**Calibration and Validation**

To evaluate the accuracy and the validity of the simulation model, holding a comparison between the results of the model with the field measurements of water level, flow velocity and sediment concentration is essentially and extremely important. By evaluating the size of the differences (errors) between results and measurements, one can accept or refuse the model’s results, the minimum differences is indicative of a higher quality of simulation. Another set of measurements needs to be used for the validation rather than the used for the calibration.

In morphodynamic models, calibration can be achieved by adjusting several parameters, those assumed initially according to the experiences in the application field. The calibration parameters considered for this work are listed below with their selected values:

1. Bed roughness \((n=0.02, n=0.025, n=0.0286, n=0.04, 3d_{50}/\text{bedform})\)
2. Water surface profile algorithm (back water, pressure field)
3. Turbulence model (standard \(k-\epsilon\), local \(k-\epsilon\))
4. Simulation time step (6hr, 1hr, \(1/4\)hr)
5. Discretization scheme (pow, sou)
6. No. of internal iterations (100, 150, 1000)
7. Sediment formula (suspended load, total load)
8. Active layer thickness (0.1m, 0.2m)

Field measurements were conducted during a range of water flows between 450 and 645 m\(^3\)/s. The flow 530 m\(^3\)/s is the closest discharge to the monthly average and was selected to be used for model calibration and a higher discharge (645 m\(^3\)/s) was selected for the validation of the model.

Calibration processes were repeated 39 times for different combinations of calibration parameters. Of these, 28 trials were designed for hydrodynamic part or parameters; such as bed roughness, a water surface algorithm, a turbulence model, the computational time step, a discretization scheme and number of internal iterations, while the remaining 11 trials were used for the sediment concentration part; such as the entrainment algorithm, the sediment formula and the active layer thickness.

Root mean square error (RMSE) was used as indication of the differences between the simulation results and measurements. For the first group of parameters, RMSE were calculated for water levels and velocity profiles, while for the second group, RMSE was calculated for sediment concentration profiles. The field measurements for CS9 and CS13 were used in the calibration processes because they were measured at the same flow rate, 530 m\(^3\)/s. The RMSE for the whole calibration trials was listed briefly in tables (2) and (3).

An overview of the calibration results in table (2) shows that most of the results are approximately identical. However, some of these parameters and algorithms were given smaller RMSE and they have to be considered for the validation process. Explanation for the importance of these parameters and their effects can be discussed in following points:

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1. Water surface profile algorithm, pressure field, gave better results than the backwater algorithm for both water levels and velocity profiles.

2. Both turbulence models, local k-ε and standard k-ε, gave approximately similar results. The local k-ε model is based on the water velocity and its algorithm was not explained clearly in the documents of the SSIIM model, so the standard k-ε is preferred.

3. Although the second order upwind (SOU) scheme has an advantage over the first order power law (POW) scheme because it takes into consideration the first order derivative of the partial differential equation, while the POW scheme ignores it, the POW scheme gave less RMSE for the velocity than the SOU scheme. However, the SOU will be considered in the following steps because of its order of accuracy and to reduce the possible false diffusion in the convection term, as well as the difference between RMSE for both of them is negligible.

4. There is no apparent effect of the time step size on the calibration results, which may return to the steady state flow that was considered in the calibration. However, as soon as the sediment transport process starts and erosion and sedimentation processes develop, the importance of the time step size will arise due to developing the changes in bed and the associated changes in water depth and velocity vector. The smaller the time step is the more stable the solution and the longer the time of computations are. A one-hour time step will be considered in the next steps unless stability requirements impose smaller one.

5. Roughness is one of the most important parameters in the set. Many of results’ magnitudes and accuracies depend on the bed roughness value; such as the water surface profile and velocity profile. Bed roughness is either computed from Manning’s roughness coefficient or from bed sediment grain size and bed form height. The later one gave a smaller RMSE, so it will be used in the next steps.

6. At each time step, there are number of internal iterations for resolving the system of equations to reduce the residuals of the variables. Increasing number of internal iterations is required the convergence in the solution; on the other hand, it will increase the computational time effectively if the grid has huge number of cells. Internal iterations between 200 and 500 were considered.

The other set of parameters are those that affect the sediment concentration profile (Table 3) these were given a wider variance of RMSE values. Discussion of their values and the corresponding RMSE are outlined in the following points:

1. An algorithm that can invoke the SSIIM model to convert the computed sediment concentrations to sediment entrainment rates for the bed cells is by using the sediment formula (Olsen, 2014). Using this algorithm gave RMSE significantly lower values than those computed without the entrainment algorithm.

2. Applying sediment load according to the sediment rating curve improved the concentration profile and reduced the RMSE. Otherwise, the computation of the concentration profile will be based only on the erosion rate from the bed.

3. The Van Rijn bedload formula (van Rijn, 1993) and suspended load formula (van Rijn, 1984) can be used separately or combined in the SSIIM model. Combining them together will compute the total sediment load. The RMSE relating to total load computation was less than the one for suspended load alone.

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4. The river bed can be simulated by two layers, active and inactive. The active layer is the top one and it has a constant thickness during simulation time. The active layer interacts with the flow and any erosion or sedimentation will compensate to/from the inactive layer. Zhang et al. (2015) found that the thickness of the active layer has to be considered as a calibration parameter. It often equated to half of the average bedform height (Mosselman, 2012) or even more up to 1.5 times the sand dune height (Tuijnder, 2010). The calibration processes showed that it was the active layer thickness 0.1m that gave smaller RMSE.

As a result of the calibration processes, the easting and northing components of the calibrated velocity profiles at CS9 and CS13 are shown together with the measured profiles in Figures (4) and (5) respectively. The calibrated sediment concentration profiles are shown in Figure (6) together with the measured concentration profiles.

The simulated depth-averaged velocity distribution of the Tigris River is shown in figure (7), where the velocity ranged between 0.07 m/s at stagnant locations close to the banks and 1.35 m/s at some locations along the deep route. The velocity field is affected by the precision of the geometry shape, where dispersion can be noted at certain locations which can be attributed to the discretization in the bathymetric survey and to the flow disturbance due to dredging operations.

The measurements of ADCP for the depth-averaged velocity profiles were compared with the simulated velocity field and it was found that the flow directions were compatible in the most of the measurements as shown in Figure (8). However, a few incompatibilities were found at some locations of circulation and they might be attributed to the false diffusion related to the discretization scheme (Dorflmann and Knoblauch, 2009).

The validation process was applied to a water flow of 645 m$^3$/s using the chosen set of parameters and algorithms from the calibration step. A new water surface profile was computed for the new water flow. The inflow of sediment discharge was set according to the water flow. To examine the model validation, the new simulation results were compared with another set of measurements which included water levels, velocity profiles and sediment concentration profiles. The RMSEs were determined for the validation step as shown in Table (4).

The results in Table (4) show that the values of RMSE for the validation process were of the same magnitudes for calibration processes. Accordingly, the model can be considered valid for simulating water surface, velocity field and sediment concentrations for Tigris River reach.

Future Prediction

To predict the future changes in any river system accurately, knowledge about the future changes in the controlling variables (such as water flow, extra sources of sediment supply, human activities, and so on) must be satisfied. This will guide the model user to adjust the parameters at the appropriate simulation time to keep the model results on track. Otherwise, the future prediction will be limited to the present knowledge of the river conditions. Significant changes can occur to the controlling variables which should be taken into consideration; otherwise unrealistic results might be produced by the model.

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After satisfying calibration and validation processes, the SSIM model can be used for predicting the future changes of river bed topography and the velocity field for the northern reach of the Tigris River in Baghdad. The aims of predicting future changes of river topography and morphology are to set guidelines for:

1. Predicting locations of future sedimentations and the expected rate of deposition, which can help in: drawing future dredging plans and deciding whether to maintain the functionality of the water intakes or to improve the flooding capacity of the river?
2. Finding out locations of future erosion which can help in warning of the possibilities of failure in river banks.
3. Changes in navigation routes can be found, which can help to warn of expected shallow routes.
4. River ecosystem and biodiversity can be predicted.

According to the available data and information about the study reach, some assumptions have to be set for simulating the future predictions as follows:

1. River flow was assumed steady and equal to the discharge that was used in the calibration (530 m$^3$/s).
2. The corresponding inflow of sediment discharge was assumed constant with the same size fractions.
3. The thickness of active and inactive bed layers were assumed to be 0.1m and 10m thick respectively along the study reach and of the same sediment fractions. The thickness of the inactive layer was considered according to the thickness of depositions found in a previous study (Geohydraulique, 1977).
4. Bridge piers will not be considered in the river geometry to avoid the disturbance in the flow field, as well as computations of local souring around bridge piers, requiring a finer grid which is beyond the scope of current work.

Using a constant water discharge at 530 m$^3$/s, which is close to the average monthly discharge for the last 13 years, simulations of future predictions can help in establishing a comparison between the locations of deposition and their patterns from the simulation results of bed changes and deposition locations those are recognized from the available aerial images in Google Earth and from other sources for the study reach during the period of 2002 to 2012. This comparison has a degree of importance because of the continuity of the dredging operations along the river in Baghdad that makes continuous monitoring for the river bed changes hard to conduct by researchers.

Long term prediction of three dimensional morphodynamic models for a large scale study reach would consume long computational time and monitoring efforts due to high potential computations and instability problems. So, the prediction period will be limited to 14 months.

RESULTS AND DISCUSSION

The disequilibrium behaviour is the usual behaviour in a river that is responding to the local or global hydrologic changes, such as climate changes or human activities (regulation and damming). Such a river tends to adjust the channel dimensions and slope continuously by
reducing channel width and increasing flow depth since both discharge and sediment load are decreased (Hickin, 1995).

The sediment capacity of the Tigris River inside Baghdad is reduced, where the water surface slope is within the range 6.5 to 6.8 cm/km, corresponding to the discharges of the period (2009-2013). Upstream Baghdad it is steeper between Samarra and Baghdad (14 cm/km) as shown in the document of MoWR (2012). Also, the average cross sectional velocity was around 0.7m/s.

**Bed Changes**

Since the flow discharges and sediment loads were reduced by the headwater regulation system, so the Tigris River in Baghdad tends to deposit part of the eroded sediment upfront on the shallow part of the cross sections having lower velocity and, on the other hand, it deepens the incised route to fit its current hydrologic condition leaving the former wide section as a floodplain for the newer river as shown in Figure (9). Higher depositions were distributed between circulation zones and meanders’ inner banks, where the flow velocity is low.

The results of future predictions for the changes in the river bed showed that the Tigris River behaved as an under fit river (Hickin, 1995). The exception in the Tigris River inside Baghdad is that the river is confined by protected banks, where the bases of the banks are filled by stones, so the margin of sinuosity is limited.

The depth of the incision at some locations seemed to be exaggerated, where it reached the whole bed sediment layer as shown in figure (10) around station 2000m and 10000m. This is attributed to the high depth of the bed sediment layer which is composed of easily erodible material. So, investigations about the real depth of the loose sediment layer and the characteristics of the strata underneath are required. Figure (10) shows the changes in thalweg line elevations. It give an indication of the potential threats of the river banks’ collapse since erosion is taking place below the protection base level especially in peaks of the meanders as shown in figure (11). Small parts of the protection had already collapsed on the outer bank meander at CS13 (Fig. 11.C), which prompted the MoWR to drive in some sheet piles to stabilize the river bank.

Some depositions took the shape of longitudinal barriers parallel to the flow direction while the deposition behind the barriers towards the banks continues to develop as shown in figure (12). The front sides of the barriers with the relatively faster flow were built from the coarser sediments while in the back sides, the finer sediment was depositing.

The results of model showed that the net deposition/erosion rate was 67.44 kg/s as an average along the study reach and the total deposition quantity was 2.12 million ton annually.

**Velocity Field**

In general, the range of the predicted depth-averaged velocity increased, where it reached up to 1.49 m/s as shown in figure (13). Specifically, the velocity increased in the incised route zone and decreased in the shallow part of the section. The flow pattern became oriented and smoother compared with the pre-prediction case.
Depositions in Reality

Figure (14) showed part of the depositions along the Tigris River in Baghdad. Google Earth photos captured on different dates, showed the size of the depositions relative to river width. The associated water discharges for the dates of the photos were added to the figure to give an indication of the water levels when the depositions are uncovered. Considering the predicted velocity distribution in the figure, it can be concluded that re-deposition at same sites is possible. So the prediction from the model can be close to the reality.

Development of Islands

Two islands are located within the study reach. The smaller one is located upstream of an acute meander (point D in Fig. 9). This island is undergoing dredging. As shown in figure (12D), depositions were built up touching the island and extended downstream forming a pond in the shade of the island. This pond is a result of depositing more fine sediment, which means developing the island again. The second island (Kura’at Island), which is relatively the bigger, is at a bisecting meander. The origin of this island can be chute cut-off then it developed later to an island. Future predictions show that the front of the island is eroding while the width and the tail length of the island are increasing as shown in figure (15). The growth rate is faster towards the outer side of the meander and it gives an indication of possible contact with the outer bank in the future, since the right branch of the meander is getting deeper and might develop to accommodate most of the flow over the whole cross section.

Navigation Routes

Figure (16) show the predicted water depths along the Tigris River. Water depth was within the range of 2 to 4 m except for the incised route zone where the depth reached up to 15m. So navigation for small boats is still possible even in the zones close to the banks where the draught of small boats is less than about 1.2m. Caution should be taken seriously by passenger boats or ferries when navigating close to the banks. Port areas are either to be kept clean from depositions by dredging or should be extend into deeper water.

CONCLUSIONS

The results of implementation of the 3-D flow model on the Tigris River reach inside Baghdad for predicting the future bed changes, led to the following conclusions:

1. The calibration of the SSIM model succeeded in reproducing the water levels, velocity profiles and sediment concentration profiles at the cross sections used for this purpose. The validation process for other cross sections produced results of the same order of error. The SSIM model has been found valid to be used for predicting future changes in the riverbed.

2. The future predictions showed the Tigris River behaved like an underfit river tending to adjust the channel dimensions and the slope by deepening an incised route that is more fit with its current discharges (water and sediment), and leaving the former wide section as a floodplain of the newer river.

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3. Higher depositions were distributed between circulation zones and meanders’ inner banks, where the flow velocity is low.
4. The erosion along the thalweg line gave an indication of the potential threats of the river banks’ collapse and the bridge piers’ instability.
5. Some deposition took shape of longitudinal barriers, forming settling ponds for finer sediment to deposit behind the barriers towards the banks.
6. The net deposition/erosion rate was 67.44 kg/s as an average along the study reach and the total deposition quantity was 2.12 million tons annually.
7. The locations of depositions are compatible with those of the river in the real world.
8. The re-deposition in the model at the same real sites along the river indicated that sedimentation processes will continue in the river for the current hydrologic conditions and deepening of the incised route will also continue until the cross section of the river is adapted or the erosion reaches a stiffer bed layer that cannot be easily eroded.
9. The pond formed by deposition in the shade of the small island, is working on re-build the dredged part again.
10. The width and the tail length of the larger island (Kura’at Island) increased. A faster rate growth on the left side of the island may lead in the future to the connecting of the island with the near bank, and the right branch of the meander might develop to accommodate most of the flow over the whole cross section.
11. Small boats of small draught have the possibility to port close to the river banks. Areas around ports have to be kept clean from sediment to let passenger boats or ferries porting easily. Otherwise ports have to be extended to deeper water.

REFERENCES


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Table 1: Sediment fractions of bed material with settling velocity

<table>
<thead>
<tr>
<th>Particle size (mm)</th>
<th>Settling velocity (m/s)</th>
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Table 2: RMSE of water level (m) and velocity profile (m/s) for calibrating of hydrodynamic parameters

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<td>local k–ε</td>
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<td>Discretization scheme</td>
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Table 3: RMSE of sediment concentration profile (mg/l) for calibrating of sediment parameters

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<th>Concentration</th>
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<td>yes</td>
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<tr>
<td>no</td>
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<tr>
<td>Sediment formula</td>
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<td>Suspended load</td>
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<td>Total load</td>
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<td>Active layer thickness</td>
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<td>1.0</td>
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Table 4: RMSE of water surface profile (m), velocity profile (m/s) and sediment concentration profile (mg/l) for validation process

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<thead>
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<th>Validation</th>
<th>RMSE</th>
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<td>Water level</td>
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<tr>
<td>Velocity</td>
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<td>Concentration</td>
<td>0.085</td>
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Figure 1. The Tigris River inside Baghdad (the islands and sandbars were bordered by red).
297x420mm (300 x 300 DPI)
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Figure 2. Cross sections of sediment sampling and ADCP measurements along the northern reach of the Tigris River,
297x420mm (300 x 300 DPI)

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Figure 3. The riverbed elevations (m.a.s.l.) of the northern reach of Tigris River. 99x141mm (220 x 220 DPI)
Figure 4: The calibrated and the measured velocity profiles of CS9.
297x420mm (300 x 300 DPI)
Figure 5: The calibrated and the measured velocity profiles of CS13.
297x420mm (300 x 300 DPI)
Figure 6: The calibrated and the measured sediment concentration profiles of CS9 and CS13.

297x420mm (300 x 300 DPI)
Figure 8: The simulated velocity vectors on the water surface (black arrows) and on the riverbed (green arrows) with the measurements of the averaged depth velocity by ADCP (blue lines).
Figure 9: The predicted bed elevations (m.a.s.l.) of the Tigris River.
297x420mm (300 x 300 DPI)
Figure 10: The initial and the predicted elevations of the thalweg line of the Tigris River.
Figure 11: The predicted locations of the potential threats of river banks collapse (points A, B and C in Fig. 9).
209x148mm (300 x 300 DPI)
Figure 12: Barrier depositions parallel to the flow direction (points D and E in Fig. 9).
Figure 14: The predicted sites of depositions in the SSIIM model and the depositions in reality including the discharges of the river.

297x420mm (300 x 300 DPI)

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Figure 16: The predicted water depths (m) in the Tigris River of water discharge 530 m3/s.
101x142mm (220 x 220 DPI)