Applicability of Sediment Transport Theories to Large Sandbed Rivers

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ABSTRACT: The inner delta of the Zaire river’s maritime reach has been surveyed intensively during the last 23 years in order to develop a method to predict its morphologic changes and to improve the dredging operations in the navigation channel giving access to the harbours of Boma and Matadi. Flow and sediment gauging form part of the tools of the prediction method; they produce interesting data for testing sediment transport theories. The paper presents results of the application of four theories or approaches: Schoklйтech, Shields, Meyer-Peter & Müller and Bagnold. Besides the calculation of the cross-sectional sediment transport rates for establishing the rating curves for the main-gauging sections, Bagnold’s approach has been adapted for application in local stations in view of predicting morphologic changes. The calculations are in fairly good agreement with the measurements.

1 INTRODUCTION
Numerous different theoretical and semi-empirical approaches have been developed for the calculation of sediment transport rates, (Einstein, 1950; Yalin, 1963; Bagnold, 1966; Ackers & White, 1973) /2/, /3/, /4/, /6/, mainly based on observations in laboratory experimental flumes /8/ and small streams. However, few attempts have been made to test their applicability in very large sandbed rivers, in which the river morphology may strongly control the transport rates. This paper presents results obtained with four formulas using data of the Zaire, second largest river in the world on the basis of annual flow discharge. The objective has been to find a sediment transport formula useful for the analysis of the sediment problems experienced in the inner delta of the Zaire river’s maritime reach. These problems are mainly related to sudden morphologic changes in the navigation channel /16/.

The Zaire river, located in the central part of Africa is the second largest river of the world on the basis of annual flow with a discharge of about 1.45 x 10^9 m^3 per year. Its hydrographic network covers about 3.7 million km^2 of the equatorial basin of central Africa /12/. The dominant characteristic of the river is the remarkable regularity of its regime due to the position of its basin (Fig. 1), one third of it north of the equator where the dry season occurs in January, two thirds of it south of the equator having a more composite regime. At Kinshasa, the capital of Zaire, at the place where the river begins to drop rapidly down over about 300 m to the maritime harbour of Matadi through the reach of the cataracts, the minimum and the maximum observed discharges are respectively 23,000 m^3/s and 80,000 m^3/s with an overall average of about 42,000 m^3/s.

The maritime reach of the river (Fig. 2), downstream of Matadi, can be divided into three parts. In the first part, extending from Matadi to Boma over about 60 Kms, the river flows through the “Crystal Mountains” with high velocities (locally up to 6.5 m/s during floods) and larger water depths.
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1 INTRODUCTION
Numerous different theoretical and semi-empirical approaches have been developed for the calculation of sediment transport rates, (Einstein, 1950; Yalin, 1963; Bagnold, 1966; Ackers & White, 1973) mainly based on observations in laboratory experimental flumes and small streams. However, few attempts have been made to test their applicability in very large sandbed rivers, in which the river morphology may strongly control the transport rates. This paper presents results obtained with four formulas using data of the Zaire, second largest river in the world on the basis of annual flow discharge. The objective has been to find a sediment transport formula useful for the analysis of the sediment problems experienced in the inner delta of the Zaire river’s maritime reach. These problems are mainly related to sudden morphologic changes in the navigation channel.

The field data were selected from an extensive data base established in collaboration between the Zairian Agency responsible for the maritime navigation, “Régie des Voies Maritimes” (R.V.M.), and the Belgian State Hydraulic Laboratory of the Ministry of Public Works. The results presented have been obtained in the Master’s thesis work of the first author.

2 BRIEF DESCRIPTION OF THE ZAIRE RIVER
The Zaire river, located in the central part of Africa is the second largest river of the world on the basis of annual flow with a discharge of about 1.45 x 10^10 m^3 per year. Its hydrographic network covers about 3.7 millions Km^2 of the equatorial basin of central Africa. One third of it north of the equator having a more composite regime. At Kinshasa, the capital of Zaire, at the place where the river begins to drop rapidly down over about 300 m to the maritime harbour of Matadi through the reach of the cataracts, the minimum and the maximum observed discharges are respectively 23,000 m^3/s and 80,000 m^3/s with an overall average of about 42,000 m^3/s.

The maritime reach of the river (Fig. 2), downstream of Matadi, can be divided into three parts.

In the first part, extending from Matadi to Boma over about 60 km, the river flows through the “Crystal Mountains”, with high velocities (locally up to 6.5 m/s during floods) and larger water depths.
The second part, between Boma and Malela, is the inner delta, 60 Km long and 19 Km wide, where shoals and islands divide numerous channels. Only a few of these channels can be made navigable for sea ships.

The water surface slope in this reach is very small, generally less than 10 cm per Km. Due to the high discharge, the changes in the bed morphology are significant, requiring frequent dredging and changes of navigation buoys.

The third part, between Malela and the river mouth near Banana, is the salt wedge estuary characterized by the presence of a deep canyon which begins near Malela, dropping abruptly to about 100 m depth and more.

3 DATA

Numerous hydraulic and sedimentologic measurements have been performed on the braided part of Zaire river with the aim to study and predict morphologic changes and to improve the dredging operations. The data used in this paper concern the main flow and sediment gauging section of "Ntua-Nkulu" and cover the period 1986 and 1987.

For each gauging, measurements have been made in several verticals across the river. Following parameters have been extracted from the field data base to investigate the selected models: date, slope, cross section area, width, coordinates of the section, water discharge, bedload, suspended load, total load discharge, velocity and water depth.

The sediment load discharges, expressed in m³/day, have been obtained from measurements with the Delft Bottle (DF), and occasionally with the Bedload Transport Meter Arnhem (B.T.M.A.) /15/. For measurements close to the bed (DF2), the inlet of the Delft Bottle has been positioned at 0.05 m, 0.15 m, 0.25 m, and 0.35 m above the bed. Between 0.40 m from the bottom and the water surface, an integrated sample is taken with a suspended Delft Bottle (DF1), at 4 to 8 levels depending on the total depth. The sample time is generally five minutes for each level. The velocity profiles have been measured with current meters at each vertical.

Figure 3 shows the variation with time of the water discharge gauged at Ntua-Nkulu, expressed per unit width (section width ranging from 1500 to 2000 m). The sediment size data were not available and taken from previous studies, as they remain quite stable with time, despite local spatial variations /13/. The mean grain size ranges from 0.20 to 0.35 mm. A mean size of 0.30 mm for bedload and 0.20 mm for suspended load transport has been adopted.

4 FORMULAS AND METHODOLOGY

Four approaches of the sediment transport problem were selected: Schoklitsch (1935), Shields (1936), Meyer-Peter & Muller (1948), Bagnold (1966), presented hereafter in short.

Bagnold's approach takes into account both suspended and bedload, the other only bedload. The bedload transport formulas are usually based on the shear stress (or tractive force) on the bed. Schoklitsch, Shields, and Meyer-Peter & Muller's relations are of this type.

4.1 Schoklitsch

Relying on experimental results of Gilbert (1914), the relation

\[ Q_b = \frac{7000}{\sqrt{\gamma}} S^{1.5} \left( \frac{Q - Q_c}{\gamma S} \right) \]  (1)

was derived by Schoklitsch for estimating the weight of bedload, the variables defined as:

- \( S \) slope of water surface, or bottom slope for quasi uniform flow
- \( Q_b \) cross-sectional bedload discharge (m³/d)
- \( Q \) water discharge (m³/s)
- \( Q_c \) critical water discharge at incipient motion (m³/s)
- \( d \) mean diameter of various particles of bedload (m)

In his considerations, Shields proceeds from the assumption that the rate of transport is necessarily a function of the dimensionless resistance coefficient

\[ \frac{1}{\tau_s \left( \gamma - \gamma_s \right) d_m} \]  (2)

where:

- \( \gamma_s \), \( \gamma \) specific weight sediment, specific weight water (t/m³)
- \( \tau_s \) shear stress (t/m²)
- \( \tau_c \) critical shear stress (t/m²)
- \( d_m \) effective diameter of sediment (mm)

In his considerations, Shields proceeds from the assumption that the rate of transport is necessarily a function of the dimensionless resistance coefficient \( \frac{1}{\left( \tau_s - \tau_c \right) d_m} \), the critical value controlling the incipient motion of bedload.

Computation of the shear stress uses the hydraulic radius \( R \) and the slope \( S \) of the river ( \( \tau_s = \gamma_s \cdot R \cdot S \) ) whereas the critical shear stress is obtained from Straub's graph (1935) for various sizes of sediment.
The bedload transport rate obtained from the Shields formula is the volume of the solid particles. The obtained value should be multiplied by a factor 1.60, in accordance with the porosity of sand, to obtain the volumetric value useful for morphologic studies.

4.3 Meyer-Peter & Müller

The formula published by Meyer-Peter and Müller in 1948, derived from results and interpretation of laboratory experiments carried out at the Laboratory for Hydraulic Research and Soil Mechanics at the federal Institute of Technology, Zurich, is essentially empirical, based on flume data.

With the aim to develop a more practical formula, the United States Bureau of Reclamation (U.S.B.R.) modified it. Conversion from metric units to English units has been made, and the following form was adopted:

\[
q_b = \left( \frac{y_s - y}{y_s} \right) \sqrt{g \left( \frac{M-N}{N} \right)^{3/2}}, \tag{3}
\]

where:

\[
M = 0.0667 \frac{Q_b}{Q} \left( \frac{d_{90}}{n_b} \right)^{1.5} y S \tag{3a}
\]

\[
N = 0.0076 (y_s - y) d_m \tag{3b}
\]

\( q_b \) bedload discharge (tons/ft.d)
\( g \) gravitational acceleration (m/s²)
\( Q_b \) the effective water discharge that determines the bedload transport (m³/s)
\( Q \) total water discharge (m³/s)
\( y \) average water depth (ft)
\( d_m \) diameter of particles 90% finer obtained from sieving test (mm)
\( n_b \) coefficient of roughness

Following simplifications have been made when applying to the Zaire data:

- \( Q/Q = 1 \). This assumption was made because it is difficult to estimate \( Q \) and, on the other hand, observations have shown that the entire flow produces bedload transport.
- Since sediment size distribution is usually very narrow, we assume that the particles are of a uniform size:
  \( d_m = d_m = 0.3 \text{ mm} \).
- \( n_b \) is assumed to be equal to Manning coefficient:
  \( n_b = n = (U)^{-1/2} R^{3/2} S^{1/2} \)
with \( R \) the hydraulic radius and \( U \) the mean velocity.

4.4 Bagnold’s approach

4.4.1 One-dimensional approach

One of the methods relating the work of energy expenditure of a stream and the quantity of sediment transported by the flow is that of Bagnold (1966) /2/. He noted that the sediment transport rate could be equated to a work rate.

Bagnold’s approach, for calculating total sediment transport, sums up the bedload and the suspended load. The bedload is defined as the part of the load which is supported wholly by solid to solid transmitted forces and the suspended load, as the part which is supported by fluid to solid transmitted forces.

The Bagnold’s transport rate of solids relationship is given as:

\[
i = i_b + i_s = \frac{e_b + e_s \frac{Q}{Q_b} (1-e_b)}{\tan \alpha} \tag{4}
\]

in which,

\( i \) total load transport rate (Kg/m.s)
\( i_b \) bedload transport rate (Kg/m.s)
\( i_s \) suspended load transport rate (Kg/m.s)
\( e_b \) transport rate of solids
\( e_s \) settling velocity
\( \alpha \) the coefficient of intergranular solid friction.

Bagnold states that \( e_b \) decreases with the increase of mean velocity as well as that of grain size; the maximum transport efficiency \( e_b \) is one third for fully turbulence and one half for laminar flow. The suspension efficiency \( e_s \) would have a constant experimental value of 0.016, and \( e_s (1-e_b) \) can be taken as 0.01.

It is generally assumed that \( U \) is almost equal to the mean flow velocity \( (u) \). This is not valid for sand bed rivers where saltation transport is important.

Once the average values of the four main parameters are known, namely \( e_b, e_s, \tan \alpha \) and \( U \), the total load may be obtained using the equation (4).
4.4.2 Pseudo-two-dimensional approach

In order to compute local sediment transport rates in single stations, Bagnold's method has been adapted /12/. The cross-section has been divided in several panels, each panel corresponding to one vertical of measurement, and the mean velocity at each vertical was calculated. The shear stress has been obtained from the measured vertical velocity profile, assuming that the shear velocity can be obtained from fitting the exponential law derived from Prandtl equation.

The power of the flow thus obtained differs from the local average power using Bagnold's one-dimensional approach. Even if the method could be criticized, it is, from an engineering viewpoint, interesting to assume that for this very large river, the product of the depth average local velocity with density and the square of a local “shear velocity” must be somehow related to the local power of the flow; this is the basic idea of the modification of Bagnold's approach for a two-dimensional application.

5 ANALYSIS OF RESULTS

5.1 Results of bedload models

5.1.1 General observations

From the results obtained with the three bedload models it is found that the predicted rates are considerably lower than the actual measured rates (Fig. 5). It is noticed that Shields model gives worse results. Regarding the ratio of measured bedload to calculated bedload discharges, the three models generate high values.

5.1.2 Analysis of the discrepancy ratio

Some graphical comparisons between measured and calculated bedload discharges are plotted (Fig. 4 & 5). The deviation noticed between measured and calculated values can be explained in different ways. Common to all models is the fact that the results were obtained mainly from experiments in large alluvial channels. Sediment transport may be strongly influenced by the secondary flow e.g. turbulence and gyres. The measured bedload discharge data are obtained in real turbulent flow conditions, whereas the calculated one is derived from a model based on laboratory flume data where the flow is guided in a single direction.

a) Schoklitsch formula

The formula derives from experimental results for sediments with mean size ranging between 0.3 mm and 5 mm. These sizes most common in the studied zone, /13/. Application of the formula bears good results for the Ntua-Nkulu gauging section.

Lebreton (1974), /9/, observed that the application of this formula gives good results only for coarse sediments and not for sand. This statement is questionable for large sandbed rivers.

b) Shields formula

There is significant difference between measured and calculated bedload discharges. This formula has also been established with laboratory flume data. Furthermore, it depends primarily on the critical shear stress, the grain size (d_m) and the range of the ratio $r = \frac{y_s}{y}$.

Shields used for his experiments two flume sizes, 40 cm and 80 cm, yielding following conditions:

1) $1.06 < r < 4.25$
2) $1.56 < y_s < 2.47$

In the present case the ratio $r = 2.65$ and $d_m = 0.3$ mm. Therefore one condition is not fulfilled.

c) Meyer-Peter & Müller

This formula is derived from experiments using laboratory flumes with a maximum width of 2 m, very different from the conditions in very large channels. The formula depends primarily on grain diameter and water discharge. Bogardi (1978), /3/, mentioned that several difficulties have been encountered including the determination of the effective particle size, which is included by the diameter at 90% by weight passing. In the area, it has been assumed that $d_m = d_m = 0.3$ mm, but the difference in result whether the uniform grain size or the non uniform one is assumed has been shown to be non-significative. The influence of water discharge appears when determining the roughness coefficient from the Manning equation.

5.2 Results of Bagnold's model

5.2.1 Presentation of results

Results of calculation using the pseudo-two-dimensional Bagnold's model are presented as follows. The distribution of computed monthly parameters along the cross-section has been plotted: total transport rate ($Q_t$), bedload ($Q_b$) and suspended load ($Q_s$) rates, power of the stream ($Pow$), shear velocity ($V_s$).... Cross-sectional average values of these parameters have been calculated for each month for comparison with results from other models.

The discrepancy ratios measured/calculated bedload transport rates have been computed. The one-dimensional and
the pseudo-two-dimensional Bagnold's approaches has also been compared (Fig. 4 & 5).

5.2.2 General observations

Analysis of the figures obtained from Bagnold's model results seems to be useful for the comprehension of the behaviour of hydraulic parameters involved in sediment transport phenomena of the selected section. In general it is noticed that the sediment transport rates, measured with samplers, and sediment transport capacities, computed with the modified Bagnold's approach, behave in a quite similar way, seemingly closely related to the varying bottom shear velocity or stress. The values of calculated sediment transport rates are generally overestimated, with the exception for some verticals. The calculated total load transport rates are, at high mean velocity, increasing with the depth of water, whereas at low mean velocity they decrease. In some cases an erratic behaviour of sediment transport rate is observed in deep waters with a high scattering of the discrepancy ratio (R) of calculated discharge rate to measured discharge rate. Generally the calculated and measured suspended load transport rates are higher than the bedload transport rates, influencing thus the total load (Fig. 4 & 5).

It is noticed that the calculated and measured bedload transport rates fit very well, whereas this is not the case for the suspension. An almost constant cross section shape at Ntua-Nkulu and a persistent peak of sediment transport rates near the left bank of the cross section were also observed, due to local geologic controls, as explained hereafter.

5.2.3 Discussion

a) Influence of geologic controls

The rather constant cross section shape observed at Ntua-Nkulu can be explained by the geological conditions of the site. The higher suspended transport rates observed near the left bank are due to the presence of hard rock outcrops, named Nzenze and Kinkela rocky bars, inducing secondary currents and thus causing resuspension of sediments. This section acts as a throat and can be regarded as the control section for the braided zone of the river.

b) Analysis of the discrepancy

The observed overestimation of calculated sediment transport rate could be due to errors in the measurement of different parameters /1/. If we consider the errors in the measurement of velocity, assuming the maximum error would be about X %, it would result in X % error in the estimation of power (Eq.(4c)) and for suspended load transport rate this error would be squared (Eq.(4b)). The error in the bedload also will be increased since e depends on the mean velocity, hence this error is very limited due to the present narrow range of grain size (0.2 - 0.3 mm). Error in the calculation of particle fall velocity, involved in Eq.(4b), affects the suspended load only. The fall velocity is assumed to be constant, 0.04 m/s, which is a constant according to the observations. It was decided not to use the effective measured fall velocity, as the uncertainty on this parameter is not so high. It is obvious that the error on this parameter affects more the calculated suspended load than the bedload transport rate.

Considering the computation of sediment transport rates, the average discharge of sediment during a period of steady flow is the discharge that should correlate with the average characteristics of flow, and the departure from the average can be considered as due to measurement uncertainties, difference between the measured discharge and the average discharge of sediment in the same flow at a steady state. But what is measured in the field are the instantaneous local sediment transport rates. Sediment load is obtained from samples taken in the bottom of the cross section near the left bank. The magnitude of turbidity near the bottom is an indicator of conditions for the exchange of bed particles between the water and the stream bottom. Also, the maximum height above the bed to which the bed load may interfere is variable and /15/ not well known. Therefore, it is not possible to determine how much of bedload may have been included in the sampling. From previous work /12/,/15/ it is found that bedload measurements, with B.T.M.A. on the bottom and D.F.2 at 0.05 m above the bottom, show a very erratic variation for the instantaneous as well as for the hourly average values, but less so for the sampling integrated over 0.40 m layer close to the bed D.F.2. The suspended load sampled with D.F.1 varies more regularly. The B.T.M.A. data (not always available) are highly variable. The rates determined with this instrument are close to those of the D.F.2 at 0.05 m, and these data were not included for sediment transport computations.

Other reasons of the observed overestimation and discrepancy can be due to the Bagnold's assumptions:
- the turbulent structure of the flow
- the procedure for determining the efficiency factors e, and e
- the sediment assumed to move in thick suspension.
Analysis of the variation of the average values of sediment transport load as a function of water discharge indicates very low discrepancy of the calculated bedload whereas higher discrepancies are noticed for the suspended load discharges (Fig. 4). Therefore the assumptions made by Bagnold about the suspension factor e, can be questioned. This remark is confirmed by Yang C.S. [18], when applying the Bagnold's approach to a tidal maritime environment. Yang found that "Bagnold suspended load transport and the total load equations Eq. (4b) & (4) with e_{1bean} = 0.01 are incorrect from the viewpoint of energy conservation. In these two equations the energy loss due to bedload transport rates has been counted twice ...".

c) Analysis of extreme values

The extreme values and the erratic behaviours of sediment transport load, velocity and shear observed in figure 4 may be attributed to the influence of the intensity of water intermingling (mixing or local effect). In fact, the magnitude of sediment transport may be influenced by the local intensity of water turbulence because water rising from the bottom holds the particles in suspension. The intensity of water intermingling augments together with an increase in velocity and water depth, as larger discharge volumes produce intense secondary currents which magnitude depends, among other, on the presence of bedforms. Usually, the sediment transport is affected by the flow velocity, the river bedform sizes and shapes, as well as by the size of bed material. The extreme values can be therefore useful indicators for the engineer, unless they appear both in the computed and measured values. This was observed in the reaches downstream Ntua-Nkulu section [16]. It must be kept in mind that the finest part of the suspended load and the wash load, beyond the scope of this study, may originate from the hydro-mechanical erosion in the catchment [7]; they do not interfere in the morphological processes in the inner delta. The major part of the materials moving in suspension in the maritime reach is composed from fine sand and mica.

Also sediment sampling procedures are affecting the uncertainty on the field data. The concentration of single samples taken in any point of a vertical may deviate considerably from the time - or space average concentration. Near banks, for instance, a shearing effect between two bodies of water with different mean velocities may originate eddies, producing resuspension of sediment where not expected on the basis of the flow characteristics.

5.3 Comparison of results

Figure 5 shows a comparison of bedload calculations with the selected models. Modified Bagnold's approach yields good results and seems to be reliable, with a discrepancy ratio of 0.46 to 1.69 at the main gauging station at Ntua-Nkulu. Schoklitsch formula gives also acceptable results with a discrepancy ratio varying from 1.6 to 2.83.

The other formulas give bad results.

Results of the application of the traditional Bagnold's approach (one-dimensional) to a set of data [1986] of Ntua-Nkulu are compared to those of the modified model (Fig. 5). It can be noticed that the results of bedload from the traditional model overestimate the values obtained with the modified approach.

Further research is necessary to explain these differences. The shear stress for the one-dimensional model was calculated with the formula relating to the slope and the grain size, whereas for the modified model it was produced by the shear velocity derived from the measured velocity profile. The bedload depends on the value of tan α, itself related to the shear stress. In the specific case of Zaire river we are in the edge of Bagnold’s graph for assessing the value of tan α, since the sediment size amounts to 0.3 mm.

Furthermore, the actual sediment transport rates may differ from the computed sediment transport capacity.

Comparison of the results with those provided by previous studies on the Zaire river ([1], [10], [11], [17]) is summarized in Table 6.

For different sets of data (i.e. for different periods) the modified Bagnold’s approach seems promising. The other models show erratic values.

6 CONCLUSIONS

From the analysis of results it can be concluded that the pseudo-two-dimensional approach provides better results than the one-dimensional approaches. This seems to prove that the shape of the channel and the heterogeneity of the flow in large rivers make the one-dimensional models inappropriate, at least for morphologic studies. The pseudo-two-dimensional Bagnold's approach provides reliable predictions of sediment transport load with a relatively constant discrepancy ratio while the other produce more or less erratic results. There is a clear trend for all formulas, the discrepancy ratio diminishing for increasing discharges. The approach developed by Bagnold seems physically sound and appears to be useful for the braided zone of the inner delta of the Zaire river's maritime reach.
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8 LIST OF ABBREVIATIONS IN FIGURES

- bedload
- suspended load
- total sediment load
- water discharge
- measured
- calculated
- power of stream
- shear stress
- velocity
- suspension
- discharge
- sediment
- one-dimensional
- two-dimensional

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/17/ SUPRIYA, T.

/18/ YANG, C.S.
**Table I: Comparison of discrepancy ratios**

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The discrepancy ratios represent the ratios between the measured sediment transport rates and the transport capacities computed with the models. **Bagnold's approach, modeled two-dimensional**
Fig. 1: Hydrographic network of the Zaire river

Legend Fig. 3:
- $Q$ (m$^3$/s) water discharge per unit width
- $P$ (kg/m s) Power of the flow per unit area
- Shearstr. ($kgf/m^2$) Cross-sectional average shear stress
- Mean veloc. ($m/s$) Cross-sectional average velocity

Fig. 3a: Water discharge per unit width in main gauging. Section Ntua-Nkulu (Jan 86 to Sept 87)

Fig. 3b: Evolution of hydraulic parameters; Main gauging section Ntua-Nkulu (Jan 86 to Sept 87)

Fig. 2: Maritime reach of the Zaire river
Fig. 4a: Gauging Dec 1986 - High water
Comparing field data with modified Bagnold's model

Fig. 4b: Gauging Jul 1987 - Low water
Comparing field data with modified Bagnold's model

Fig. 4c: Gauging Dec 1986 - High water
Variation of discharge, flow power and total sediment transport across gauging section

Fig. 4d: Gauging Jul 1987 - Low water
Variation of discharge, flow power and total sediment transport across gauging section

Fig. 4e: Gauging Dec 1986 - High water
Variation of water discharge, mean velocity and shear velocity across gauging section

Fig. 4f: Gauging July 1987 - Low water
Variation of water discharge, mean velocity and shear velocity across gauging section

Fig. 4: Results main gauging section Ntua-Nkulu
Fig. 5a: Comparison of measured data with computed (Bagnold)

Fig. 5b: Comparison of measured data with computed (models of M-P&M, Shields, Schoklitsch)

Fig. 5c: Evolution from Mar 86 to Dec 86 of discharge and computed sediment transport (Bagnold's 1-Dim & 2-Dim)

Fig. 5d: Evolution from Mar 86 to Dec 86 of measured bedload transport and computed (all models)

Fig. 5e: Ratio of measured transport to computed data (Bagnold's 1-Dim & 2-Dim)

Fig. 5f: Ratio of measured transport to computed data (Bagnold's 2-Dim, M-P&M, Shields and Schoklitsch)

Fig. 5: Results main gauging section Ntua-Nkulu