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Fluvial Stage-discharge Rating of Wadi Hanifa Main Channel

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Abstract. Recent cross-sectional survey of Wadi Hanifa reach, within Arriyadh city, has been analyzed. The analysis includes the estimation of carrying capacity, possible maximum scour depth and sediment transport capacity of the selected reaches. A well-known resistance equation has been adopted and modified to a simple form in order to be used in the present analysis. The modified resistance equation has been used to calculate the mean velocity through the channel sections. In addition, a sediment transport equation has been applied for the prediction of transport capacity of the various sections. Results show that the existing drainage sections of Wadi Hanifa channel reach under study have adequate carrying capacity under existing bank-full conditions, but these reaches are subject to bed erosion even in low flow situations. Regarding sediment transport rate, it can be estimated that the channel flow has a relatively low range of bed material concentration. Finally, stage-discharge curves for various sections have been developed. Based on stage-discharge rating data of various sections, water surface profile and sediment-rating curve of Wadi Hanifa have been developed.

Introduction

Runoff from a catchment area can be best estimated and calibrated by measuring discharge of the streams draining in it. Available methods for measurement of discharge in a stream include: (i) control device method, (ii) area-velocity method, (iii) area-slope method and (iv) tracer technique. Flow measurement devices are constructed in channels in an effort to ensure a constant stage-discharge relationship, which results in an accurate discharge measurement without velocity measurements. The area velocity method requires the velocity measurement and area of flow to obtain flow rate. This method is generally used for flow measurements where flow in a state of quasi-steady conditions and the velocity measurement is not difficult. The third and fourth methods are indirect way of discharge measurement, that do not require current meters. In tracer technique, a known quantity of dye is injected into the flow upstream and then measured some distance downstream when the dye has become completely mixed. By comparing the concentration at the downstream site with the mass of dye injected, the flow rate can be deduced. However, this method is particularly suitable for stony mountainous stream where dye is mixed quickly and measurement by other methods are difficult. In spite of many difficulties in discharge measurement, it is also not feasible to measure the discharge everyday due to economic consideration. Thus stream flow data is not recorded regularly, instead water levels are recorded and the stream flow is deduced by means of rating curves [1]. A rating curve is used to convert records of water level into flow rate.

The use of rating curve of a river channel has long been of interest to hydraulic engineers and modelers. A numerical model requires a logical scheme, where stage and velocity can be predicted for a given channel characteristics including bed material, bed slope and discharge. Engineers are often faced with problem of designing a channel to accommodate a given discharge with a given bed slope and an unknown sediment discharge.

In the light of the above, this paper attempts to explore the possible application of flow resistance equation in order to develop a stage-discharge rating curve. Prior to the application of a flow resistance equation, a well-known predictor developed by Garde and Ranga Raju [2] has been adopted and modified to a relatively simple form for easy application in alluvial channels.

Stage-discharge Predictor

Many techniques for stage discharge prediction have been proposed in literatures, [2-8]. Brownlie [8] suggested a flow depth equation for sand bed channels in which discharge was considered as the independent variable. He also made rigorous statistical comparison of techniques of various stage-discharge predictors based on the collected data of Rio-Grande conveyance channels and Mississippi river. He found that the flow depth predicted by the Garde and Ranga Raju [2] procedure yields consistent results with those of field data compared to other predictors. This can be considered as an independent comparison of the procedure of Garde and Ranga Raju [2] in addition to the comparison with data on which the equation is based.

Based on the analysis of large number of data from canals, rivers and laboratory flume, Garde and Ranga Raju [2] obtained a relationship and later Ranga Raju [9] graphically presented a function of the form

$$K_1 \frac{U}{\sqrt{g(\Delta\rho_s/\rho)R}} = f\left[K_2\left(\frac{R}{d}\right)^{1/3}\left(\frac{S}{\Delta\rho_s/\rho}\right)\right] \qquad (1)$$

where U is the mean velocity, R is the hydraulic radius, S is the channel longitudinal slope, d is the sediment size and $\Delta\rho_s/\rho$ is the relative sediment density. The empirical coefficients K_1 and K_2 are functions of sediment size as shown in Fig. 1. The data used for development of Eq. (1) cover a wide range of sediment sizes and flow conditions. Comparing their procedure with observed data, Garde and Ranga Raju [2] stated that the mean velocity can be predicted within ± 30 percent accuracy. However, the functional form was only presented graphically, but if the relationship is to be adapted to other computer applications, it is worth to fit a function to this relationship.

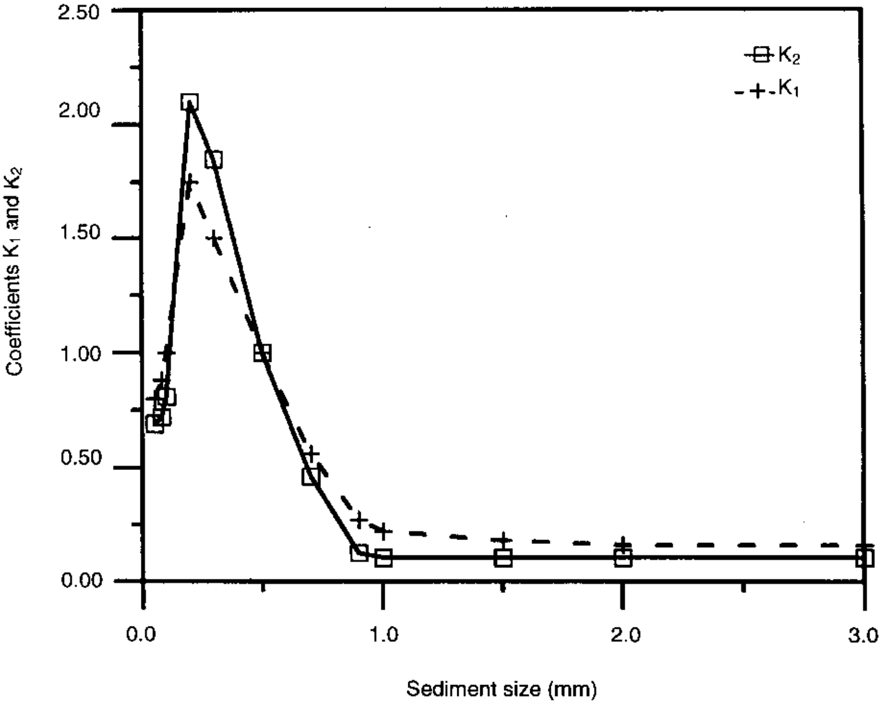

Fig. 1. Coefficients K_1 and K_2 versus sediment size [10].

The graphical relation derived from the functional form, Eq. (1) can be very closely approximated, after rearranging, by the following expression:

$$\frac{U}{U_*} = \lambda(K_2^x / K_1)(\frac{\Delta\rho_s}{\rho})^{(1/2-x)}(\frac{R}{d})^{x/3} S^{(x-1/2)} \quad (2)$$

where U_* is the shear velocity ($=\sqrt{gRS}$), λ is the empirical coefficient and x is an exponent and need to be evaluated using the observed data. In this study, the values of λ and x for three different regimes have been evaluated further by fitting observed data from Ranga Raju [10] for median condition. The values of λ and x are shown in Table 1.

Table 1. Values of coefficient λ and exponent x of Eq. (2).

λ	x	$R^* = K_2(R/d)^{1/3}(S\rho_s/\rho)$	
4.0	0.55	≤ 0.01	(lower regime)
98.12	1.192	0.01 to 0.02	(transition)
5.63	0.43	≥ 0.02	(upper regime)

Expressing the entire coefficient term $\lambda(K_2^x/K_1)$ of Eq. (2) by an inverse of a single resistance factor F_f, Eq. (2) can be rewritten as;

$$\frac{U}{U_*} = \frac{1}{F_f}\left(\frac{S}{\Delta\rho_s/\rho}\right)^{(x-1/2)}\left(\frac{R}{d}\right)^{x/3} \quad (3)$$

in which the values of F_f are function of sediment size and can be obtained from Fig.2. Values of F_f have been calculated using coefficients K_1, K_2 (Fig.1) and λ and x values from Table 1. Venoni [11] reported a similar form of Eq. (3), which was an another version of Garde and Ranga Raju equation. Also, Eq. (3) has a similar form of the dimensionless Manning-Stricler type equation for exponent x = 1/2. However, for movable bed alluvial channels the value of x varies for three different regimes due to variation of resistance occur from various bed formations.

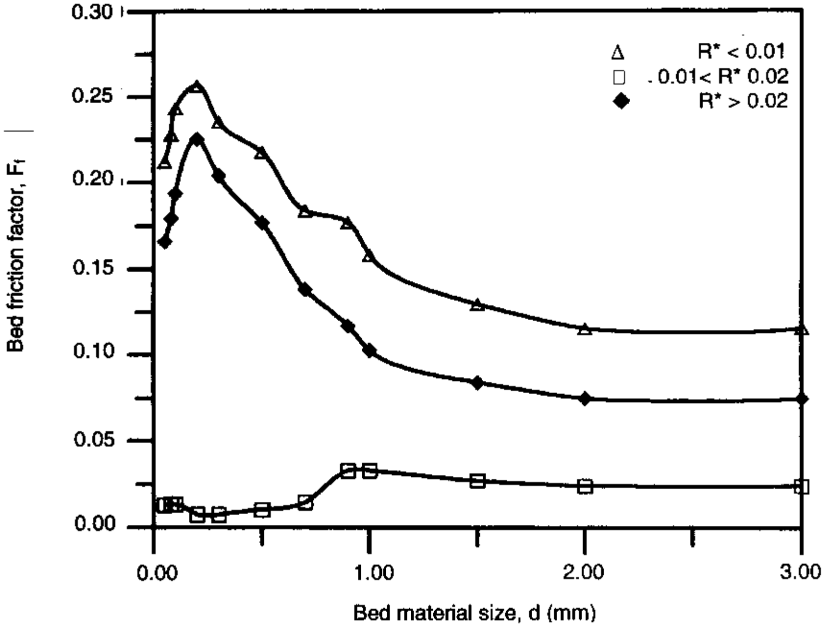

Fig.2 Alluvial resistance factor as a function of sediment size.

Wadi Hanifa Main Channel

Wadi Hanifa is one of the principal Wadis draining the eastern slope of the Tuwaig mountains. It extends from north of Al Uyaynah to the south of Al-Hair city. The watershed area of Wadi Hanifa is estimated to be about 4400 km^2. Wadi Hanifa flows southward from its source near the town of Al Uyaynah until it ends into Wadi Shahba, a distance of about 150 km (Fig.3). The main Wadi flood-channels is slightly east of the center of the catchment area and flow northwest to the southeast. Most of Arriyadh city is located in the Wadi Hanifa catchment area. The average longitudinal slope of the study reach ranges between 0.00153 and 0.0027. The bed and banks of the channels are mainly composed of alluvial deposits of large to medium size sands and silts.

Prior to extensive urban activities, the source of water in the main channel was the seasonal rainfalls. During the last decade, a considerable urban and agricultural developments occurred in the catchment area which led to considerable disposal of sewage effluent and agricultural drainage [12]. Due to the increase in consumption and disposal of industrial water, the groundwater table within Arriyadh city has risen considerably. Therefore, water table is usually lowered through pumping of groundwater to the Wadi resulting in a continuous source of water through the Wadi.

The amount of rainfall on Wadi Hanifa catchment area shows extreme variation from year to year. In some years the amount of rainfall is very little while in others torrential rain take place creating flash floods. In 1990 and 1995 the annual rainfall recorded in Arriyadh city is about 40 mm and 234 mm, respectively. Sendil *et al.* [13] reported hydrological characteristics of Wadi Hanifa based on observed rainfall, evaporation and other climatic data. Their study was mainly concentrated towards development of intensity-duration-frequency relationships of Wadi Hanifa catchment based on 18 years (1964-1981) rainfall records. They found that the maximum rainfall intensity is 94.2 mm/hr for 100 year return period.

Ishaq and Khararjian [14] presented low flow and high flow rating curves of a storm sewer out-fall to Wadi Hanifa. The estimated discharge was 5.3 m^3/s at 0.25 m depth for low flow case. However for high flow condition, the discharge could be as much as 460 m^3/s at 2.5 m depth. Their analysis was based on observed water level data at the sewer out-fall. Based on recent rainfall data and catchment characteristics, Alhamid [15] derived flood hydrographs for some sections on Wadi Hanifa main channel. The estimated 50-year peak discharge through these sections were found to range from 42 to 150 m^3/s depending on routing and size of the conveyance channels.

Methodology

(i) Prediction of discharge capacity

Recent survey data of eight sections in Wadi Hanifa channel have been collected from Arriyadh Development Authority (ADA) [16] and used in the present analysis. Location of the present study and cross-sections details is shown in Fig.3 and Fig.4, respectively. The actual cross sections are analyzed for different flow depths, increasing from the bed level to the bank-full level. The procedure includes the computation of hydraulic mean depth, D, at different water levels. For each cross-section, the hydraulic mean depth at each water level is computed conventionally using the following relation:

$$D_i = \frac{1}{T_i}\left(\Sigma A_{i-1} + \frac{1}{2}(T_{i+1} + T_i)(h_{i+1} - h_i)\right) \tag{4}$$

where A is the flow area, T is the top width and h is the water level at level i ,i= 1,2,3...up to number of division, from lowest bed level to bank full level. However, sections associated with floodplain are analyzed by using flow division procedure. In this procedure, the main channel and side channel velocities are calculated separately with the assumption that small amount of exchange of shear stress at the interface of water between main and side channel is accounted [17]. Therefore, in calculating the hydraulic mean depths of two channel portions, wetted perimeter of entire main channel including this interface is considered, while for the side channels only the wetted perimeter for bed and banks are considered.

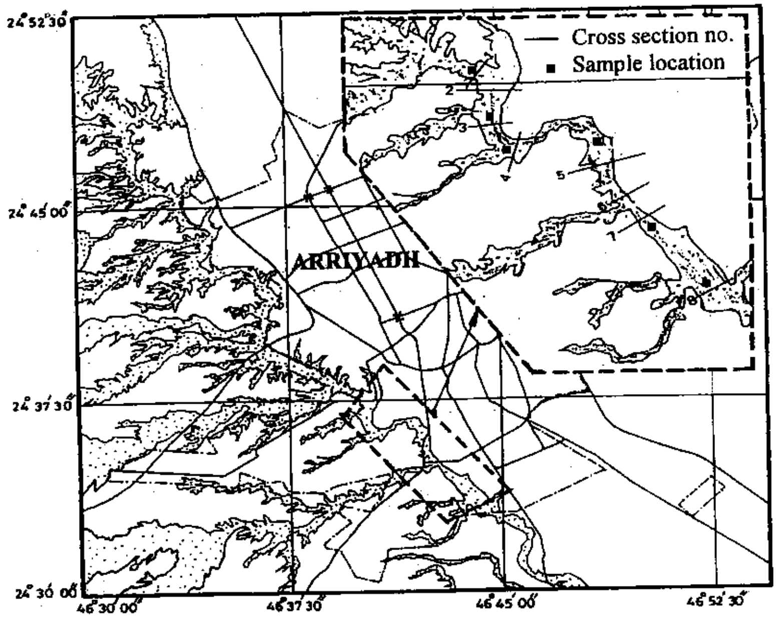

Fig. 3. Location of Wadi Hanifa and the study area showing cross-sections.

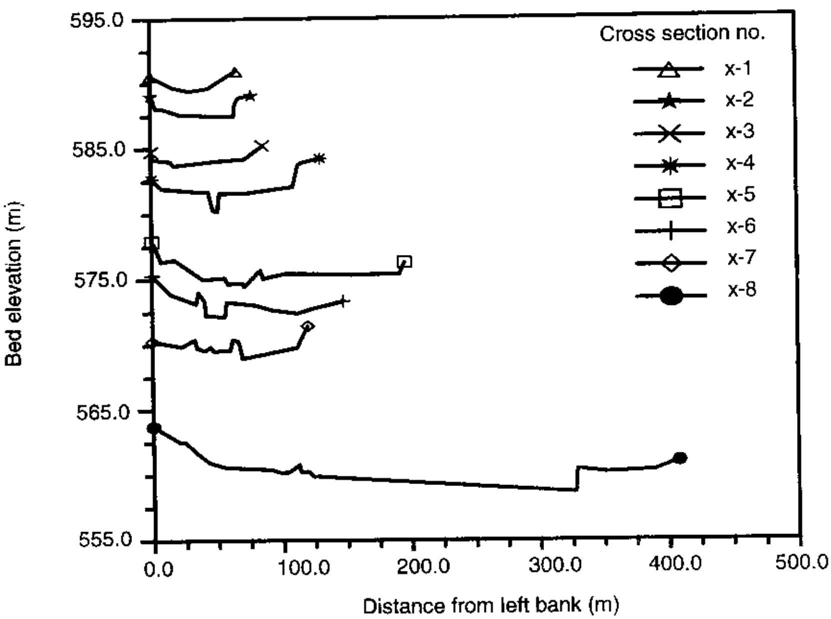

Fig. 4. Selected cross-sections of Wadi Hanifa main channel.

For Wadi Hanifa channel reach, sediment characteristics do not varying significantly. Typical grain size distribution curves of bed material from four different locations are shown in Fig. 5. It can be seen that the shape of size distribution curves for all the cases seems to be similar and the median sediment size is close to 2.0 mm. This sediment size can be considered representative size of the Wadi Hanifa channel.

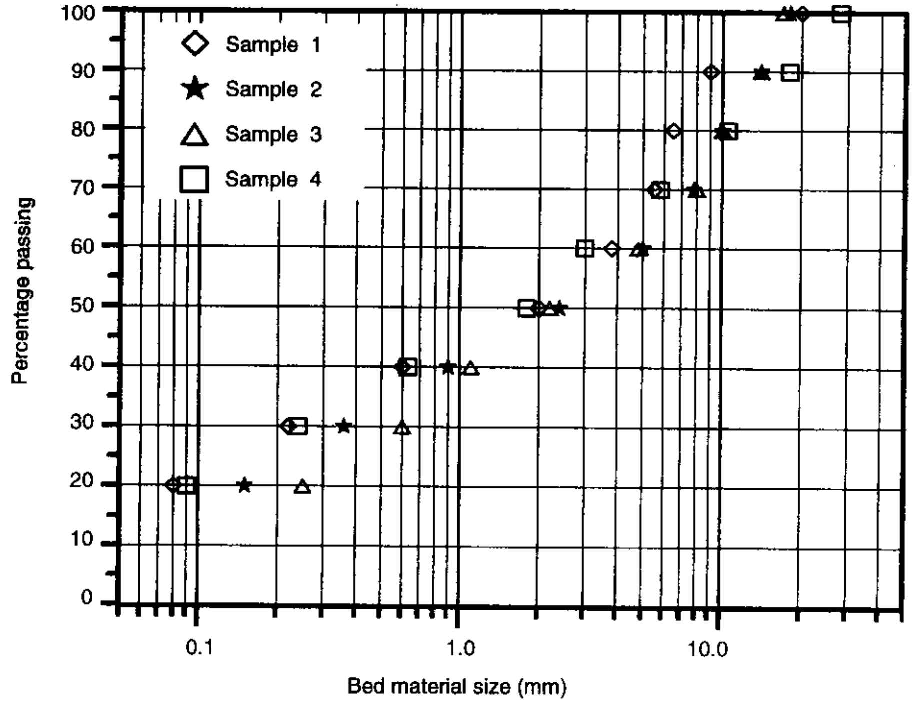

Fig. 5. Typical grain size distributions of bed material for selected locations.

Thus for bed material size $d_{50} \geq 2$ mm, relative density, $\Delta\rho_s/\rho = 1.6$, the parameter R* found to be less than 0.01, which falls in lower regime. Therefore, using the F_f value as obtained from lower regime curve of Fig.2, and with R = D, Eq. (3) becomes;

$$\frac{U}{U_*} = 7.292(S)^{0.05}\left(\frac{D}{d_{50}}\right)^{0.186} \tag{5}$$

For given bed slope, S, and sediment median size, d_{50}, Eq. (4) and Eq. (5) have been used to calculate the mean velocity for any given stage. Accordingly, discharge may be computed by multiplying the mean velocity with the corresponding flow area.

ii) Shear stress difference

The average shear stress on the bed of an open channel at which the sediment particles just begin to move is called the critical shear stress. Shields was the first to give a semi-theoretical analysis for determination of critical shear stress and his analysis is commonly used. He expressed a graphical relation between dimensionless critical shear stress $\tau_* = \tau_c/(\Delta\rho_s gd)$ and boundary Reynolds number, $R_e^*=U_*d/\nu$, for incipient motion of bed particles. Here τ_c is the critical shear stress, ν is the kinematic viscosity of water and g is the acceleration due to gravity. In the turbulent region, it was shown in Shields diagram (after Chang [18]) that τ_* range between 0.047 and 0.06 for Reynolds number $100< R_e^* <500$. Thus, for known values of flow depth, slope and sediment size, the shear stress difference can be computed.

(iii) Estimation of erosion depth

During floods, the channel bed may undergo scour due to flowing of water. This general scour of the river bed may cause several problems such as bank instability. Although the bed levels may be restored by deposition as the floods recedes, it is necessary for the hydraulic engineers to estimate maximum bed scour depth adjacent to the banks. This is particularly important when a hydraulic structure like culvert or bridge abutments need to be constructed in the channel bed.

Predicting a general scour can be dealt with as a problem of determining equilibrium boundary shear stress responsible for resistance of grains to motion. The grain at the surface of a stationary bed must be subject to $\tau \leq \tau_c$. Hence a depth can be estimated to satisfy this threshold requirement. No suitable theoretical method has yet been available in literature for proper estimation of general scour depth in natural channels. However many empirical equations are proposed, for example, Blench [19] suggested an empirical equation to estimate the extent of erosion which is

$$h_s = z\left(\frac{q^2}{F_{b0}}\right)^{1/3} \tag{6}$$

where, h_s is the maximum scoured depth measured from water level in meter, z is factor accounting for the local flow pattern, ranging from 1.5 to 2.0, when flow is parallel to the bank, q is the local discharge intensity in $m^3/s/m$ and F_{b0} is the zero bed factor which is a function of median size of bed material. Neill [20] provided values of F_{b0} for different sediment size (after, Hemphill and Bramley [21]). For given sediment size and estimated discharge, Eq.(6) is used to compute the anticipated maximum erosion depth.

(iv) Total sediment transport capacity

The total sediment transport rate is an integral part of any problem involving alluvial channels, because these channels do not just carry water, but water and sediment. The sediment flow in a channel is partly responsible for its instability, such as erosion and deposition, silting of reservoirs etc. Therefore, the sediment transport rate must be known to the hydraulic engineers for designing a new channel on the same plain.

The total sediment load in nature is the sum of bed load, suspended load and wash load. However, excluding wash load, sum of bed and suspended load is often called total load. Bed material load also some times referred as total load. Several total load prediction formulas are available in related literatures. For instance, the Colby equation [22], Engelund-Hansen [23] formula, Yang [24] equation and Ackers and White [25] equation which are well known and widely used as sediment discharge predictors [18] White *et al.* [26] compared eight different formulae using 1000 flume and 260 field measurements. It was shown that Ackers-White equation predicts 68% of data within discrepancy ratio range 0.5 to 2, compared to 63% by Engelund-Hansen equation [27]. The laboratory data include 0.04 to 4.494 mm and field data from 0.095 to 68 mm. Lau and Krishnappan [28] tested eight different formulae using their laboratory sediment discharge data. They found that the prediction by Ackers-White equation agrees reasonably well with observed data. Using sediment data of Sacramento river, Nakato [29] also tested eleven sediment discharge formula. He showed that the overall prediction by Ackers-White equation looks excellent. Based on the evidences from these literatures, Ackers-White equation has been selected for prediction of sediment transport rate through the channel sections. Moreover, this equation can be applicable to wide range of sediment sizes.

For application in Wadi Hanifa channel, the Ackers-White equation can be expressed as;

$$C_s = 0.074\left(\frac{d_{50}}{D}\right)\left(\frac{U}{U_*}\right)^{0.061}\left(\frac{F_{gr}}{0.173}-1\right)^{1.54} \tag{7}$$

in which, the particle mobility number F_{gr} is given by

$$F_{gr} = 99.77(U_*)^{0.061}\left(\frac{U}{5.657\log_{10}\left(\frac{10\cdot D}{d_{50}}\right)}\right)^{0..939} \tag{8}$$

In Eq. (7), C_s is the total load concentration by weight. For known sediment size, channel slope, flow depth and mean velocity, Eq. (8) is used for the computation of F_{gr}. The values of F_{gr} are shown in Fig.6 for known values of flow depths.

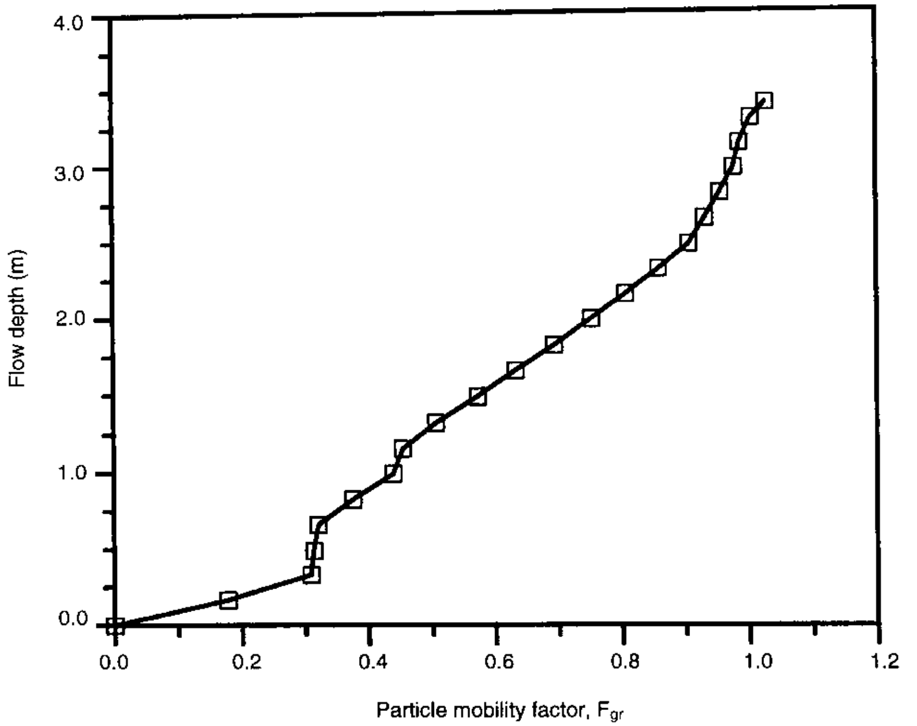

Fig. 6. Flow depth versus Ackers-White particle mobility factor, F_{gr}, for Wadi Hanifa channel.

Results and Discussion

Eight cross-sections over 14 km length of Wadi Hanifa reach have been used to determine flow and sediment transporting capacity. The geometric dimensions of these sections are shown in Table 2. It can be seen that the Wadi Hanifa channels are becoming flared towards the downstream side. Consequently, the flow cross-sectional area increases from 42.35 m^2 to 308 m^2, except two cross-sections.

Discharge capacity, shear stresses, maximum erosion depth and sediment concentration have been estimated for various existing cross-sections under bank full conditions. Results are shown in Table 3. It is seen that the actual shear stresses exceeded considerably the corresponding critical value. Results indicated that all the existing sections under study are subject to erosion for given longitudinal slope and sediment size. Here, dimensionless critical shear stress of 0.056 derived from Shields diagram [17] has been used as a threshold condition. As computed and shown in Table 3, the discharge capacity of cross-section X-1 is found to be 74.80 m^3/s when the flow depth is 1.175 m. Maximum discharge carrying capacity of sections X-3 and X-7 (Table 3) are found less compared to other cross-sections in the reach. Reason is that, these two sections are relatively shallower and narrower under bank-full condition and have less flow area as listed in Table 2.

Table 2. Geometric dimensions of selected cross-sections along Wadi Hanifa reach at bank full condition

Cross-section No.	Distance (down-stream) (m)	Max. depth (m)	Bank-full width (m)	Flow Area (m^2)	Hydraulic Mean depth (m)	Average bed slope $\times 10^{-3}$
X-1	0	1.17	58.92	42.35	0.718	1.76
X-2	1093	1.50	72.00	83.27	1.156	2.31
X-3	2493	1.15	82.00	65.83	0.802	2.69
X-4	3787.5	2.56	111.93	118.58	1.050	2.01
X-5	7048.5	1.77	176.30	160.20	0.908	1.53
X-6	8875	1.99	134.86	159.60	1.183	1.86
X-7	9980	1.49	103.52	72.73	0.702	2.52
X-8	14300	1.815	361.00	308.00	0.854	2.37

Table 3. Discharge capacity, maximum scour depth and sediment transport rate through the cross-sections of Wadi Hanifa reach for bank full condition

Cross-section No.	Depth at bank full level (m)	Shear stress difference (τ-τ_c)* (N/m^2)	Discharge capacity (m^3/s)	Max. scour depth from water level (m)	Sediment concentration (ppm)
X-1	1.175	+ 10.65	74.80	2.424	671.1
X-2	1.50	+ 24.45	236.38	4.489	1026.8
X-3	1.151	+ 19.42	158.37	3.198	1150.7
X-4	2.56	+ 19.11	293.90	3.912	866.7
X-5	1.77	+ 11.88	307.47	2.989	603.7
X-6	1.985	+ 19.85	408.53	4.304	809.5
X-7	1.485	+ 15.61	154.08	2.693	1039.5
X-8	1.815	+ 14.77	659.72	3.026	775.4

* $(\tau - \tau_c) = \rho gDS - \tau_* (\Delta\rho_s gd)$

+ ve sign means bed and banks are subject to erosion

Consequently, the estimated maximum discharge carrying capacity becomes less for these two sections. Higher carrying capacity is envisaged for downstream section X-8 which is 639.72 m^3/s for a flow depth of 1.815 m. Obviously, many other branch channels are linked to the main Wadi channel along the sides of downstream reach, providing higher flow carrying capacity of the main channel itself. However, for constant flow depth of 1.00 m, the corresponding discharge through the sections X-1 and X-8 are estimated as 57.74 m^3/s and 82.17 m^3/s respectively as shown in Table 4. For sections X-4 and X-7, the estimated discharges are found to be 7.68 m^3/s and 42.21 m^3/s respectively, which are obviously very low compared to those discharges at bank-full levels. This is due to the fact that, these sections have a narrow deep cut at a depth close to 1.00 m (Fig.4), which provides less flow area in discharge computation. It is worth mentioning here that, the discharge close to bank-full stages increases at a higher magnitude when compared with that of low stages. This higher rate of discharge increment is due to wider water conveying area associated with flood plain. Foregoing results are obtained based on constant slope and sediment size. However, it can be estimated that for a ±10% change in longitudinal slope will give a ±5.38 % change in discharge. On the other hand, for a specific bed slope, 10 % decrease of sediment size will give only 1.75 % increase in predicted discharge.

Maximum possible erosion depths for various sections have also been computed. It is observed that for discharge intensity $q=1.286\ m^3/s/m$ and $d_{50}=2.0$ mm, erosion depth can reaches up to 1.249 m from the bed level for the case of cross-section X-1. Similarly, other computed erosion depths for other cross-sections are also tabulated in Table 4.

Table 4. Discharge capacity, maximum scour depth and sediment transport rate through the cross-sections of Wadi Hanifa reach for flow depth = 1.00 m

Cross-section No.	Shear stress difference (τ-τ_c) (N/m^2)	Discharge capacity (m^3/s)	Max. scour depth from water level (m)	Sediment concentration (ppm)
X-1	+ 11.31	57.74	2.174	847
X-2	+ 15.60	106.17	2.896	906
X-3	+ 12.39	98.42	2.375	866
X-4	+ 10.55	7.68	2.032	831
X-5	+ 5.95	36.45	1.208	682
X-6	+ 7.94	54.02	1.561	762
X-7	+ 7.41	42.21	1.467	744
X-8	+ 5.55	82.17	1.140	663

+ ve sign means bed and banks are subject to erosion

Equation (5) has been applied to generate stage discharge data for each cross-section. A typical stage-discharge rating curve generated for section X-3 is shown in Fig.7. This can be used as discharge predictor by knowing water level of the flow. This water level can be measured in the field by installing water level recorder at suitable location of the channel reach. Based on stage-discharge data, water surface profiles of the

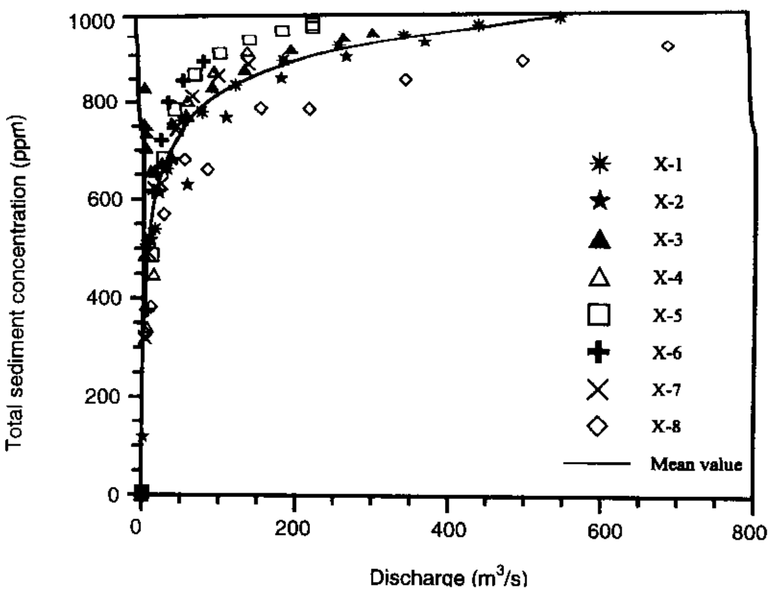

Fig. 9. Sediment rating curves in term of total load concentration versus discharge.

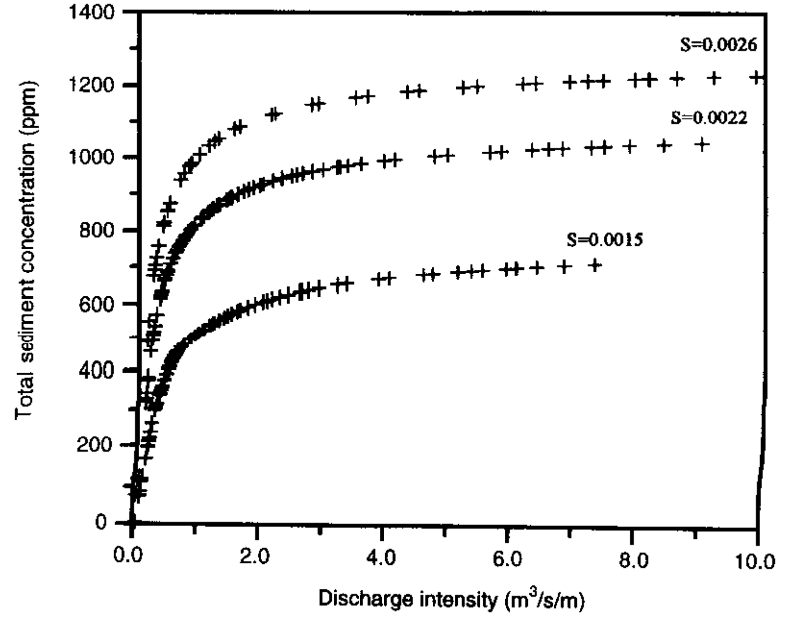

Fig. 10. Sediment rating curves in term of total load concentration versus discharge for different bed slopes.

Concluding Remarks

A simple resistance equation applicable for the alluvial channels has been proposed. Based on the collected cross-sectional and sediment data, Eq. (5) can be used to generate stage-discharge rating curves for various sections of the Wadi Hanifa. Computed water surface profile shows that the channel reach can carry up to125 m^3/s discharge, without causing over bank spills for a considerable portion of the reach. Combined with Eq. (5), sediment transport rate is also predicted using Eq.(7) as adapted for Wadi Hanifa case. It reveals that the channel sections have adequate discharge carrying capacity under bank-full condition, except for one or two sections. The channel sections have low range of bed material concentration (within the range 603 to 1150 ppm). It is worth mentioning that the results of the present study need to be verified based observed flow and sediment data.

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الخصائص النهرية لمجرى وادي حنيفة

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ملخص البحث. هذه الورقة هي محاولة لتحديد الخصائص النهرية لمجرى وادي حنيفة المار عبر مدينة الرياض وذلك من خلال تحليل البيانات المساحية لعدد من المقاطع العرضية للمجرى، و تشتمل هذه الخصائص على تحديد الطاقة الاستيعابية لكل مقطع وعمق الجرف المحتمل بالإضافة إلى كمية الرواسب المنقولة، وتم تحديد السرعة المتوسطة لكل قطاع باستخدام واحدة من المعادلات المشهورة لمقاومة التدفق والتي تم تبسيطها لتناسب الدراسة، كما تم حساب كمية الرواسب المنقولة باستخدام معادلة انتقال الرواسب.

وأظهرت النتائج أن المقاطع التي تم دراستها تستطيع استيعاب كميات كبيرة من التدفق دون غمر المناطق المجاورة ولكن هذه المقاطع مهددة بالجرف حتى أثناء التدفقات القليلة، أما بالنسبة لكميات الرواسب المنقولة من هذه المقاطع فيمكن تصنيفها ضمن الفئة قليلة التركيز، وتم إيجاد علاقات تربط بين عمق التدفق وكميات المياه وكذلك عمق التدفق وكمية الرواسب المنقولة لكل مقطع.