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DYNAMIC WAVE ROUTING OF KRISHNA REACH BY USING HEC-RAS

Priyanka R Dhumal[1], Pranjali A. Chiwhane

Abstract: The present paper focuses on the concepts of hydraulic flood routing model, with time-varying roughness updating to simulate flows through natural channels, based on the quasi-steady dynamic wave and full dynamic wave theory, emphasizing the solving of the intricate Saint Venant's equation (continuity & momentum equation). A real case study of Dynamic wave routing through HEC-RAS has been dealt with reaches (XS 68 to Karad (chainage 120 km) & Xs 223 to Karad – Kurundwad) in Krishna River. The technique provides a reliable initialization of stage/discharge profile for the flood mapping. The examinations including the initialization of stage profile, conservation of mass, iteration convergence, Manning's N, effectiveness evaluation, and convergence with optimum theta (implicit weighing factor) values are conducted to verify the flood mapping and in validating the model. The result of the hydraulic model shows that the value of simulated discharge is nearly equal to the actual calculated value of discharge by area velocity method at Kurundwad (X-section_10) (0100 hrs on 28th July, 2010, i.e., end of simulation period).
Keywords: HEC-RAS, Dynamic Wave Routing, Hydraulic model, Saint Venant equation.

1. Introduction

Hydraulic routing employs the full dynamic wave (St. Venant) equations [11]. These are the continuity equation and the momentum equation, which take the place of the storage-discharge relationship used in hydrologic routing. The equations describe flood wave propagation with respect to distance and time. Henderson rewrites the momentum equation as follows: [5]

$$S_f = S_0 - \left(\frac{\partial y}{\partial x}\right) - \left(\frac{V\partial V}{g\partial x}\right) - \frac{1}{g}\frac{\partial V}{\partial t}$$

Where,

S_f = friction slope (frictional forces), in m/m; S_o = channel bed slope (gravity forces), in m/m;
2nd term = pressure differential; 3rd term = convective acceleration, in m/sec^2; Last term = local acceleration, in m/sec^2

$$q = A\left(\frac{\partial V}{\partial x}\right) - \left(VB\frac{\partial y}{\partial x}\right) - B\frac{\partial y}{\partial t}$$

The description of each term:

A (.V/.x) = prism storage, VB (.y/.x) = wedge storage, B (.y/.x) = rate of rise,
Q = lateral inflow

The full dynamic wave equations are considered to be the most accurate solution to unsteady, one dimensional flow, but are based on the following assumptions used to derive the equations Henderson,:[5]

1. Velocity is constant and the water surface is horizontal across any channel section.
2. Flows are gradually varied with hydrostatic pressure prevailing such that vertical acceleration can be neglected.
3. No lateral circulation occurs.
4. Channel boundaries are considered fixed and therefore not susceptible to erosion or deposition.
5. Water density is uniform and flow resistance can be described by empirical formulae (Manning, Chezy) [2] [3] Solution to the dynamic wave equations can be divided into two categories: approximations of the full dynamic wave equations, and the complete solution.

The three most common approximations or simplifications to the full dynamic equations are referred to as Kinematic, Diffusion, and Quasi-steady models. They assume certain terms of the momentum equation can be neglected due to their relative orders of magnitude. The full momentum equation is

$$S_f = S_0 - (\frac{\partial y}{\partial x}) - (\frac{V \partial V}{g \partial x}) - \frac{1}{g}\frac{\partial V}{\partial t}$$

Kinematic and diffusion models have found wide application and acceptance in the engineering community [9]. This acceptance can be attributed to their application to mild and steep slopes with slow rising flood waves [1]. Henderson [5] supported this by computing values for each term in the momentum equation. It was found that the last three terms of the momentum equation are two orders of magnitude less than the channel bed slope value and therefore are negligible for steep slopes.

1.1 Fully Dynamic Wave Routing

1.1.1 Description

Complete hydraulic models solve the full Saint Venant equations simultaneously for unsteady flow along the length of a channel. They provide the most accurate solutions available for calculating an outflow hydrograph while considering the effects of channel storage and wave shape [9]. The models are categorized by their numerical solution schemes which include characteristic, finite difference, and finite element methods.

Characteristic methods were used for early numerical flood routing solutions based on the characteristic form of the governing equations. The two partial differential equations are replaced with four ordinary differential equations and solved along the characteristic curves [5]. The four equations are commonly solved using explicit or implicit finite difference techniques [9] [10] [11]. State that characteristic methods incorporate cumbersome interpolations with no added accuracy compared to the finite difference techniques.

The finite difference method describes each point on a finite grid by the two partial differential equations and solves them using either an explicit or implicit numerical solution technique.

Explicit methods solve the equations point by point in space and time along one time line until all the unknowns are evaluated then advance to the next time line [9]. Much research has been performed on this topic [9] [10]. Implicit methods simultaneously solve the set of equations for all points along a time line and then proceed to the next time line [11]. [9][10][11][12], among others. The implicit method has fewer stability problems and can use larger time steps than the explicit method. Finite element methods can be used to solve the Saint Venant equations [12]. The method is commonly applied to two-dimensional models.

1.2 Theoretical Calculations For One-Dimensional Flow

The following paragraphs describe the methodologies used in performing the 1-D flow calculations within HEC-RAS. The basic equations are presented along with discussions of the various terms. Solution schemes for the various equations are described. Discussions are provided as to how the equations should be applied, as well as applicable limitations.

- Steady Flow Water Surface Profiles
- Unsteady Flow Routing

1.2.1 Steady Flow Water Surface Profiles

HEC-RAS is currently capable of performing one-dimensional water surface profile calculations for steady gradually varied flow in natural or constructed channels. Subcritical, supercritical, and mixed flow regime water surface profiles can be calculated. Topics discussed in this section include: equations for basic profile calculations; cross section subdivision for conveyance calculations; composite Manning's n for the main channel; velocity weighting coefficient alpha; friction loss evaluation; contraction and expansion losses; computational procedure; critical depth determination; applications of the momentum equation; and limitations of the steady flow model. (**Fig.1**) depicts the terms of the energy equation representation.

***1.2.1.1* Equations for Basic Profile Calculations**

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

$$Z_2 + Y_2 + \frac{a_2 V^2}{2g} = Z_2 + Y_2 + \frac{a_2 V^2}{2g} + he \qquad (1)$$

$Z_{1,}$ Z_2= elevation of the main channel inverts
Y_1, Y_2= depth of water at cross sections
V_1, V_2= average velocities (total discharge/ total flow area)
a_1, a_2= velocity weighting coefficients
g = gravitational acceleration, h_e=energy head loss

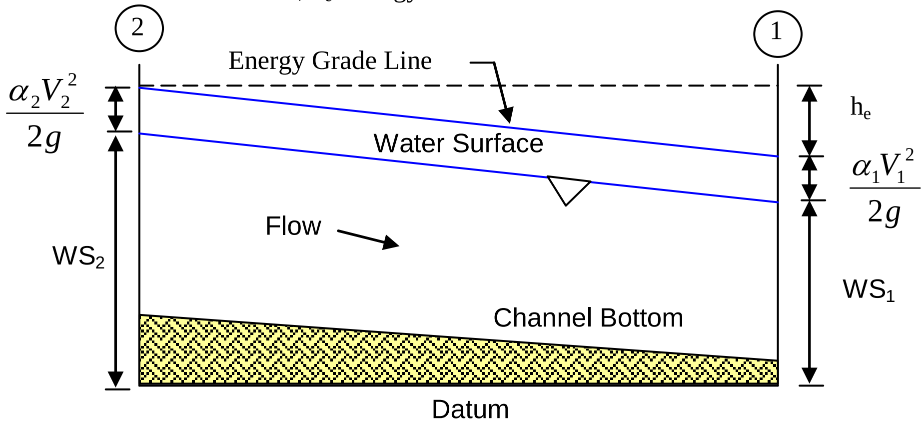


Fig.1 REPRESENTATION OF TERMS IN ENERGY EQUATION

Friction losses

The energy loss term h_e in equation 1 is composed of friction loss h_f and form loss h_o. Only contraction and expansion losses are considered in the geometric form loss term.

$$h_e = h_f + h_o \qquad (2)$$

To approximate the transverse distribution of flow of the river is divided into strips having similar hydraulic properties in the direction of flow. Each cross section is sub divided into portions that are referred to as subsections. Friction loss is calculated as shown below:

$$h_f = \left(\frac{Q}{K^1}\right)^2 \qquad (3)$$

$$\text{Where, } K^1 = \sum_{j=1}^{J} \left[\frac{1.49}{n_j}\right] \frac{\frac{(A_2 + A_1)}{2}\left[\frac{R_2 + R_1}{2}\right]_j^{1/2}}{L_j^{1/2}} \qquad (4)$$

A_1, A_2 = downstream and upstream area, respectively of the cross sectional flow normal to the flow direction

J = total number of subsections
L_j = length of the j^{th} strip between subsections
n = Manning's roughness coefficient
Q = water discharge
R_1, R_2 = downstream and upstream hydraulic radius

Other losses

Energy losses due to contractions and expansions are computed by the following equation:

$$h_0 = C_L \left| \frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right| \quad (5)$$

Where, C_L = loss coefficient for contraction and expansion. If the quantity within the absolute value notation is negative, flow is contracting, C_L is the coefficient for contraction; if is positive, flow is expanding and C_L is the coefficient of expansion. In the standard step method for water surface profile computations, calculations proceed from the d/s to u/s based upon the reach's downstream boundary conditions and starting water surface elevation.

2. Study Area and Data Sets

Considering the availability of hydrological, meteorological, soil, and other collateral data, the reaches from XS 68 to Karad, XS 223 to Karad and Karad to Kurundwad in Krishna River were selected as the study area for the present Dynamic wave modelling study. The following texts narrate the study area with its brief characteristics. The cross section details as obtained from Central Water Commission (Upper Krishna Division) are marked by the numbers described by Govt of India.

Fig.2 INDEX MAP OF KRISHNA BASIN

In India the Krishna River rises from Western Ghats near Mahabeleshwar in Maharashtra state and after traversing a length of 304 Km. in Maharashtra, it enters into Karnataka state. Total length of the river is 1392 Km. and passes through Maharashtra, Karnataka and Andhra states joins the Bay of Bengal (**Fig.2**). The Krishna basin in Maharashtra is broadly classified in two sub basins viz. Krishna sub-basin and Bhima sub-basin. In the upper reaches of Krishna River in Maharashtra, rainfall is found to the tune of 4000 to 7000 mm. There are 11 Major, 12 Medium and 263 minor dams located in Upper Krishna basin. Total live storage capacity of these dams is 7136 Mm^3. Looking to all these aspects, dams have limited scope in limited flood control. Therefore, heavy floods are observed and are always possible in future when there is intensive precipitation in all over area of Krishna sub-basin.

2.1 Data sets

The accuracy of the model depends on the detail and accuracy of the river geometry that is input to the model (as well as the choice of appropriate time and distance steps). Input data for each cross section must describe channel slope and geometry; over bank storage; natural and man-made constrictions (such as bridges); channel and over bank roughness coefficients, and lateral inflows or outflows. In addition each model needs upstream and downstream "boundary conditions" – usually a flow hydrograph at the upstream end and some form of stage-discharge relationship at the downstream end.

2.1.1 Geometry Data

The study area consist of geometric data is in the form of a station and elevation. It consist of total 51 cross section which is a surveyed by central water commission Upper Krishna Division Pune. That describes the main channel and bank station. The distance between surveyed cross section varies from 1km to 5km.

2.1.2 Boundary condition

The simulation period from 27th July 2010, 28thjuly 2010 is constrained by the availability of data to prescribe the models boundary condition. The upstream boundary condition at XS 68 and XS 223is given in the form of stage/ flow Hydrograph. The downstream boundary condition is represented by a rating curve constructed from observed water levels and discharge measurements at kurundwad provided by the Upper Krishna Division, Pune.

3. Results and Discussion

By running an Hydraulic model it gives the all hydraulic parameter for each cross section and water surface profile plot of the reach **(Fig .3)** The result of the hydraulic model shows that the simulated Discharge is **3803.82 m^3/s** against the actual calculated value of **3847** m^3/s by area velocity method at Kurundwad (**Fig.4**) (X-section_10) (0100 hrs on 28th July, 2010, i.e., end of simulation period). The simulated water level is ***534.15m*** against observed value of ***534.21*** m.

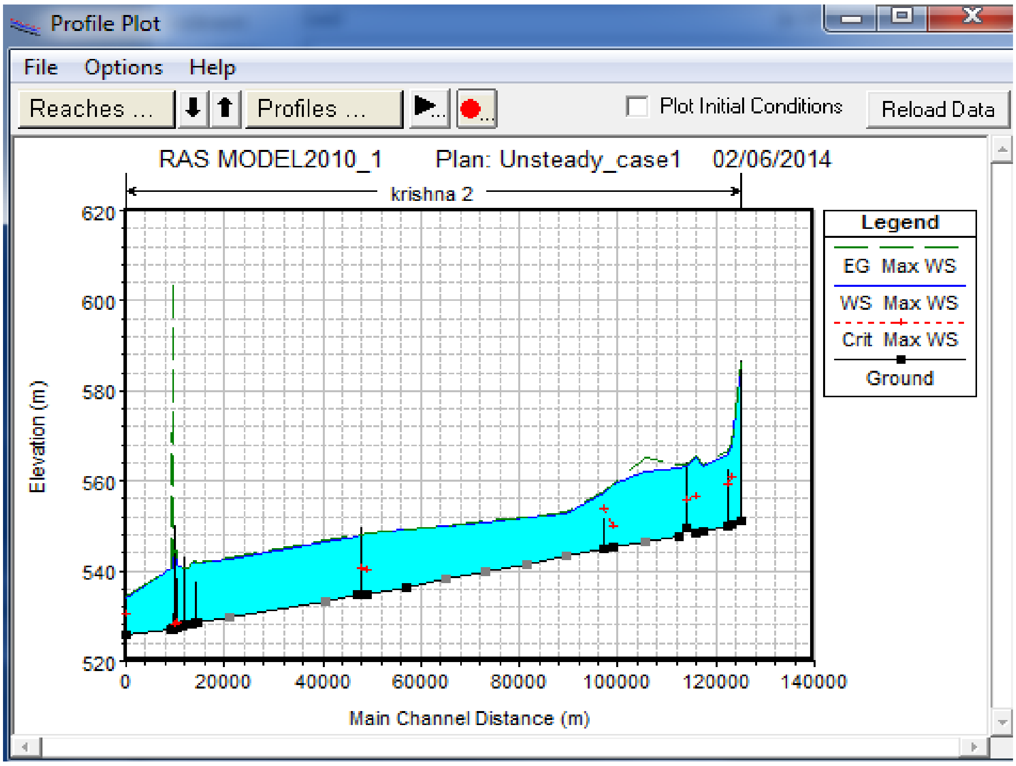
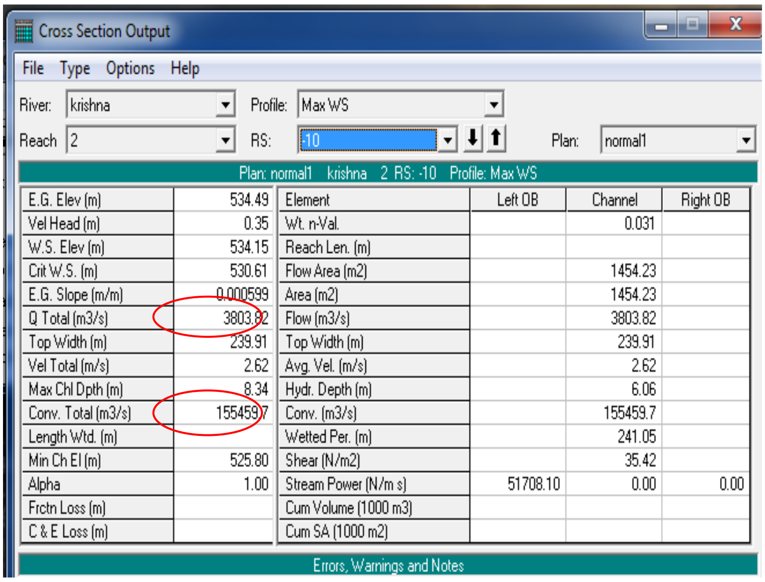


Fig.3 PROFILE PLOT



Cross Section Output

File Type Options Help

River: krishna Profile: Max WS

Reach 2 RS: -10 Plan: normal1

Plan: normal1 krishna 2 RS: -10 Profile: Max WS

E.G. Elev (m)	534.49	Element	Left OB	Channel	Right OB
Vel Head (m)	0.35	Wt. n-Val.		0.031	
W.S. Elev (m)	534.15	Reach Len. (m)			
Crit W.S. (m)	530.61	Flow Area (m2)		1454.23	
E.G. Slope (m/m)	0.000599	Area (m2)		1454.23	
Q Total (m3/s)	3803.82	Flow (m3/s)		3803.82	
Top Width (m)	239.91	Top Width (m)		239.91	
Vel Total (m/s)	2.62	Avg. Vel. (m/s)		2.62	
Max Chl Dpth (m)	8.34	Hydr. Depth (m)		6.06	
Conv. Total (m3/s)	155459.7	Conv. (m3/s)		155459.7	
Length Wtd. (m)		Wetted Per. (m)		241.05	
Min Ch El (m)	525.80	Shear (N/m2)		35.42	
Alpha	1.00	Stream Power (N/m s)	51708.10	0.00	0.00
Frctn Loss (m)		Cum Volume (1000 m3)			
C & E Loss (m)		Cum SA (1000 m2)			

Errors, Warnings and Notes

Fig.4 DETAILED CROSS-SECTION OUTPUT TABLE

3.1 Validation of Discharge Using Area Velocity Method

Area of Kurundwad X-Section at WL 534.395 m on 28th July, 2010 = 2840.654 m^2 (Measured)
Avg. Velocity on 28th July, 2010= 1.354 m/sec
Discharge (Q) = Area * Velocity= 2840.654 m^2 X 1.354 m/s= 3847.240 (m^3/s)
The simulated discharge for 28th July is 3804 (m^3/s), hence it proved to be in agreement.

4. Conclusion

This paper presents a methodology and case study of a Dynamic wave routing conducted along Krishna reach. The HEC-RAS model gives a accurate result as compare to the conventional method .from the simulated discharge and water level we can find out the flood plain for the further study.

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