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Technical Note:

One-Dimensional Hydrodynamic Modelling for River Flood Forecasting

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Note from the Editor: The ability to forecast maximum water depth during maximum discharge of a design flood is very important in designing flood protection scheme along the river reach. This paper explains the use of ISIS Flow, a one-dimensional hydrodinamic computer modelling for river flood forecasting. The computer simulations produced detailed information from each node including the maximum water depth during maximum discharge, thus it can be expected that an economical flood protection structure can be produced.

Introduction

Floods have accompanied mankind throughout its entire history. Although the causes for this have varied, e.g. extreme changes in the river catchments and material deposition along the riverbed, the impact is the same. Floods are dangerous to people's lives and vital interests. Floods are now the main hydrological topics worldwide. The sensitivity of environment and national economy to the impact of floods is becoming ever more pronounced. In certain developing countries such as Indonesia, the land use in the catchment area changes continuously and it is very difficult to be restrained. No wonder that flood damage in the past that was caused by discharges having a return period of 100 years, can today be the results of a 20-year maximum discharge [1, 2]. Therefore the prediction of stage, discharge, time of occurrence and duration of the flood, especially of peak discharge at a specified point on a stream is absolutely necessary. These activity is known as flood forecasting, whereas flood warning is defined as the provision of advance notice that a flood may occur in the near future at a certain station or in a certain river basin [3].

When a region is affected, the system of flood control service is activated and operates according to previously drafted flood plans. An effective flood warning system needs to be based on accurate timely flow forecasts.

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It has widely been recognised that the collection of river flow data involved the use of autographic recorders operated by floats in which the analysis of the data was extremely time consuming. Although many autographic are still used particularly in developing countries, the explosion in the electronic industry has led to sophisticated microprocessor based loggers. These instruments are versatile, quite reliable and can operate at remote sites without mains electricity and can also be used to log data from a number of different censors simultaneously [4]. In recent years, telemetry systems have also developed rapidly. This system is designed to control the data transmission from an instrument site to a control centre, negating the need to collect it manually [5].

Literature Review

Modelling River Flow

Water flow in natural channels is almost always unsteady. It is a complex phenomenon and cannot be understood in all details [6]. That is why in certain cases unsteady flow is sometimes approximated by steady flow, particularly when the change of discharge with time is very gradual. In hydraulic engineering problems it is important to recognise when an unsteady flow may properly be treated as a steady flow. The mathematical treatment of unsteady open channel flow is an important but relatively difficult problem. The difficulty exist, basically because many variables enter into the functional relationship and because the differential equations cannot be integrated in closed forms except under very simplified conditions [7]. For engineering purposes most of the solutions of unsteady flow equations are numerical with a great number and variety of techniques.

Mathematical modelling in rivers is the simulation of flow conditions based on the formulation and solution of mathematical relationships expressing hydraulic principles. Advanced mathematical treatment of unsteady flow in open channels was started with the development of two partial differential equations. Although there were many attempts to modify and to improve them, the equations remain substantially unchanged. The equations resulted from these various attempts are more complete and sophisticated but reduce to the basic de Saint-Venant equations whenever simplified for practical use [7].

It is not possible to solve de Saint-Venant equation analytically, hence the need for numerical solution. The finite difference schemes are the practical method used to deal with natural channels, where the watercourse is simulated by a series of computational points [8]. This method was developed because of the limitations imposed on time step, Δt, when using explicit schemes [9]. Each computational points represents an elementary reach corresponding to the space step, Δx, in which each point corresponds to the cross section. This section should be selected to represent all important topographical and hydraulical features of the reach.

The ISIS Flow

ISIS Flow is a program for modelling steady and unsteady flows in open channel networks. It is used to model open channel and overbank flows in any network of channels. ISIS Flow computes flow depths and discharges using de Saint Venant equations, a method based on the equation for shallow water waves in open channels [10]. For unsteady flow solutions ISIS Flow uses the governing hydraulic equations for each unit. These equations are inevitably a combination of empirical and theoretical equations many of which are non-linear. The non-linear equations are first linearised and the solution to the linear version of the problem is then found via matrix inversion. An iterative procedure is used to account for the non-linearities. The Preissmann four-point implicit finite difference scheme is employed for the channel equations and the matrix is inverted using a powerful sparse matrix solver [10].

Model Simulations

Limitations

The river reach in which the ISIS Flow model to be developed was treated as a single river without controls where the flow is specified at the upstream boundary and stage at the downstream end. In this case the Direct Method was applied. Two types of run, steady and unsteady were selected before entering the appropriate time parameters for simulations. It should be noted that entries for all parameters are not required for some runs. For example, steady runs require a value for run start time only. Because of the steady run can occur at any time for which data is available in the boundary conditions it was no need to choose a particular start time. A start time of 1.00 hour can simply be used to simulate a steady run [9]. For steady flow runs the values of *Initial Condition* must be specified for both stage and discharge. For unsteady flow runs these values can either be entered directly or obtained from steady flow runs. The *Boundary Condition* is an additional variable to the initial condition required to solve the scheme [10].

In steady Direct Method a single convergence value for both stage and flow is displayed after each network's iteration. If no iterations are necessary then the value is not displayed. For unsteady runs the convergence ratios of flow (Q_{RATIO}) and stage (H_{RATIO}) are only displayed if, after six iterations Q_{RATIO} and H_{RATIO} are greater than the convergence values given in the *General System Parameters* section of the data file [10]. In this case computations move on to the next time step and it should ensure that the non-convergence has not invalidated an important part of the simulation. If non-convergence continues to occur then the simulation should be aborted.

Scaling of Cross Sections and Discretisation

The choice of spacing of river cross sections is fundamental to the success of the application of a computational hydrodynamic model such as ISIS Flow. A sufficient number of cross sections must be provided so that the model of each stream reach preserves the geometric and hydraulic properties of the channel. Guidelines to select cross section have been developed over a number of years from a mixture of theoretical analysis and experience. Open channel systems modelled by taking the channel parameters and computing stages and discharges at a set of discrete cross sections. Each cross section is separated from the next by a distance, Δx, and the solution is carried forward in time by a series of discrete time step, Δt. The factors influencing the choice of distance and time step are the bed slope, Manning's value, minimum discharge, channel top width and water surface curvature. For non-tidal reaches the ISIS Flow user's manual [10] presents that the distance between cross sections should generally not be more than 20 times of top width of the channel and the time step should be sufficiently small to resolve the shape of the boundary condition.

Study Area

The river reach used for the simulation was River South Tyne, one of tributaries of River Tyne located in the North East of England. The stretch of about 15 km long between Haltwhistle and Haydon Bridge was selected. The catchment area at Haydon Bridge, the end of the stretch, is 751.1 km^2 [11]. The sections should generally be spaced approximately uniformly where possible, but in this case, due to the layout of the reach, different spacing was adopted. A cross section was selected such that the model can be executed as simply as possible. Closer spacing was adopted particularly in river bend and internal boundary condition, such as bridge, island and junction whereas in straight reach the longer spacing was allowed. Reach of the river was divided into 27 cross sections. Schematic diagram of river network on the reach is presented in Figure 1 whereas Table 1 shows the description of cross sections together with the distance and their locations.

Roughness Coefficient and Bed Slope

The Manning's equation was developed to describe flow in an infinite channel with constant cross section, energy gradient and roughness, conditions rarely encountered in natural channel. River South Tyne can physically be categorised as gravel and boulder-bed river [12]. The average Manning roughness coefficient adopted in this simulation was 0.035. This value is provided by The Environment Agency [13].

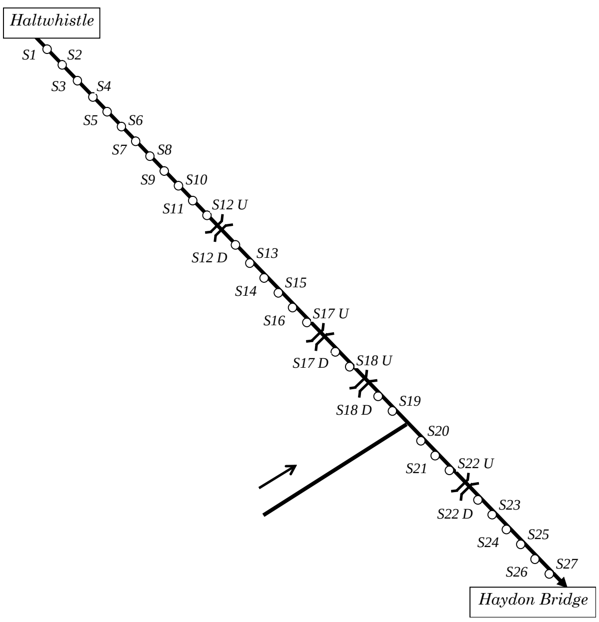

Fig. 1. Schematic diagram of river network between Haltwhistle and Haydon Bridge

According to the Natural Environment River Council [14] the average bed slope of river Tyne is 0.00183. As different reaches of the same river have different bed slope, it was necessary to determine the bed slope between observed cross section. In this case bed slopes between Haltwhistle and Haydon Bridge were determined based on 1:10000 scale of the map of extent flooding on river South Tyne [11]. The flood level was measured by the assumption that the water surface slope is representing the bed slope. No measurement was taken in certain sections where the bed slope data were available. These values were preferred to be used and can be directly adopted in simulations. Bed slopes between cross section can be seen in Table 1 with the bed elevation above ordinary datum (AOD) measured at centre line of the river.

Results and Discussion

Steady Run

During the hydraulic calculation for steady run, no errors were found in node labels. At the beginning a set of data was used as boundary condition both for upstream and downstream end of the network. These data were adopted from rating curve based on recorded measurement at Haydon Bridge gauging station (Figure 2). Different values of flow with different time were supplied for flow time boundary (QTBDY) at upstream and different values of stage and time for head time boundary (HTBDY) at downstream. Since steady flow does not change with time, only the values in the first row were adopted by steady Direct Method.

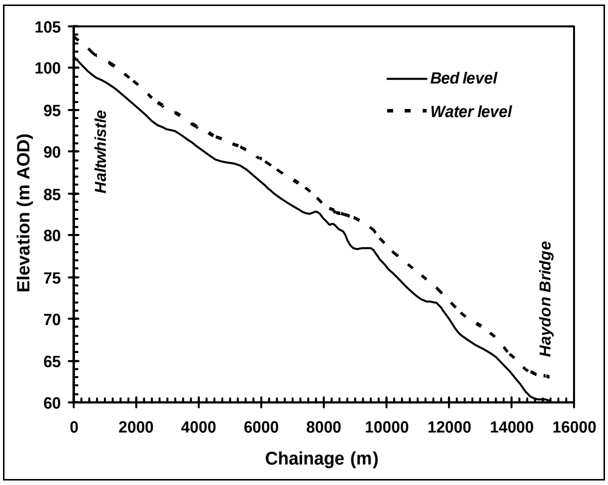

Fig. 2. Rating curve of River South Tyne at Haydon Bridge gauging station

Various values selected as boundary condition initially caused error in the simulations. Careful examination indicated variable was outside interpolation range in one or more sections. In this case the distance was then set within the range and the boundary data did not extend over full model run time. Another problem encountered in the simulation was insufficient number of cross sections. In this case, for the purpose of running the steady Direct Method additional sections were added to improve simulation. Two more cross sections than those presented in Figure 1 were added (i.e. sections 9010 and 26027) so that the total number of cross section increased to 29 points.

Table 1. The scaling and location of cross sections within selected river reach

Section	Distance from preceding section (m)	Bed slope (10^3)	Bed level at centreline (m AOD)	Characteristic of section
S1	0		101.69	straight
S2	600	3.48	99.60	straight
S3	560	2.41	98.25	bend
S4	750	3.23	95.83	straight
S5	780	2.80	93.64	straight
S6	560	2.01	92.51	bend
S7	540	2.25	91.30	bend
S8	750	2.96	89.08	straight
S9	780	0.86	88.41	straight
9010	790	3.42	86.62	straight
S10	400	3.42	85.71	straight
S11	900	2.48	83.48	bend
S12 (U)	360	1.66	82.88	straight (bridge, upstream)
S12 (D)				straight (bridge, downstream)
S13	400	2.78	81.77	bend
S14	150	2.78	81.35	island
S15	150	2.78	80.93	island
S16	150	2.78	80.51	straight
S17 (U)	330	2.03	79.84	straight (bridge, upstream)
S17 (D)				straight (bridge, downstream)
S18 (U)	560	1.99	78.73	straight (bridge, upstream)
S18 (D)				straight (bridge, downstream)
S19	260	6.32	77.09	straight (confluence, upstream)
S20	460	2.33	76.02	straight (confluence, downstream)
S21	900	3.42	72.94	bend
S22 (U)	520	2.19	71.80	straight (bridge, upstream)
S22 (D)				straight (bridge, downstream)
S23	720	4.83	68.32	bend
S24	940	2.05	66.39	bend
S25	620	3.78	64.05	bend
S26	660	2.98	62.08	bend
26027	430	2.98	61.23	straight (interpolate, downstream)
S27	220	2.98	60.80	straight (bridge, upstream)
	15240 m (total length of the reach)			

The results of simulations can be seen in Table 2 at the start time of 1.00 hour. The extreme flow discharge of 251.153 m^3/s at water depth of 2.219 m were adopted as flow time boundary (QTBDY) at the upstream and head time boundary (HTBDY) at the downstream end of the reach. The value of the bed level is the lowest level of the bed as described in cross section data. Subtracting stage by the bed level gives water depth for each section. Water profile along the reach is shown in Figure 3 based on the full results of steady run.

The highest water depth calculated by steady Direct Method was at Sect17. This is because of the existence of a bridge structure in which the width of the river at the observed cross section decreased. This contraction contributed to the increased of water depth to 3.686 m or almost 50 % increased from the average water depth of the reach. The lowest water depth was found at Sect14. Although two islands exist at this section the width of cross section was relatively wide so that the area of cross section was still able to accommodate the discharge. At this section the water depth was 1.613 m or 35 % less than the average water depth.

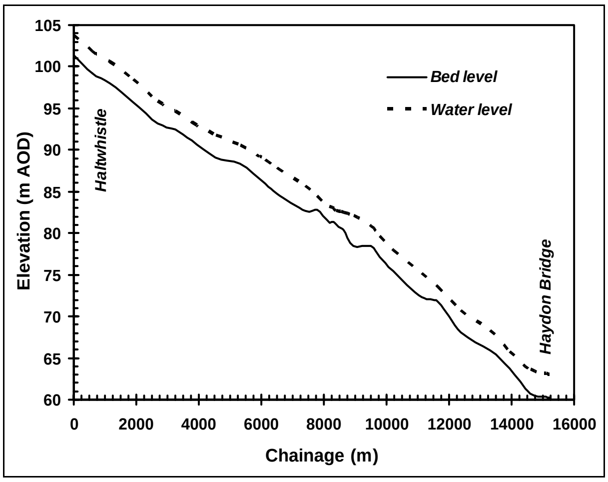

Fig. 3. Water levels for steady state run at discharge of 251.153 m^3/s

Table 2. The results of steady run simulation using Direct Method at start time 1.00 hour

Label	**Chainage (m)**	**Discharge (m^3/s)**	**Stage (m AOD)**	**Froude Number**	**Velocity (m/s)**	**Bed level (m AOD)**
Sect1	0	251.153	103.797	0.596	2.368	101.430
Sect2	600	251.153	101.841	0.521	2.446	99.260
Sect3	1160	251.153	100.492	0.470	2.198	98.040
Sect4	1910	251.153	98.542	0.583	3.017	95.680
Sect5	2690	251.153	95.823	0.628	2.865	93.160
Sect6	3250	251.153	94.571	0.431	1.789	92.470
Sect7	3790	251.153	93.334	0.528	2.358	91.070
Sect8	4540	251.153	91.760	0.370	1.774	89.050
Sect9	5320	251.153	90.508	0.483	2.016	88.300
9010	6110	251.153	88.783	0.458	2.122	85.910
Sect10	6510	251.153	87.880	0.501	2.375	84.700
Sect11	7410	251.153	85.689	0.511	2.337	82.710
Sect12	7770	251.153	84.543	0.704	2.717	82.780
Sect13	8170	251.153	83.221	0.402	1.483	81.320
Sect14	8321	251.153	82.953	0.449	1.453	81.340
Sect15	8470	251.153	82.584	0.475	1.552	80.700
Sect16	8620	251.153	82.393	0.298	1.245	80.420
Sect17	8950	251.153	82.086	0.323	1.850	78.400
Sect18	9510	251.153	80.907	0.626	2.880	78.400
Sect19	9770	251.153	79.679	0.704	3.375	77.080
Sect20	10230	251.153	77.970	0.565	2.499	75.400
Sect21	11130	251.153	75.255	0.538	2.421	72.290
Sect22	11650	251.153	73.566	0.680	2.804	71.700
Sect23	12370	251.153	70.718	0.427	2.074	68.090
Sect24	13310	251.153	68.342	0.708	3.103	65.930
Sect25	13930	251.153	65.825	0.541	2.282	63.720
Sect26	14590	251.153	63.635	0.758	3.109	60.750
26027	15020	251.153	63.086	0.280	1.306	60.413
Sect27	15240	251.153	63.019	0.192	0.945	60.240

Unsteady Run

The results from the steady run were used as the initial conditions for unsteady run and then an initial six steady iterations were calculated before the proper simulation began. The purpose of these initial iterations was to smooth the transition from the steady methods, particularly from the steady to the unsteady method. There were many problems encountered during the simulation. At the beginning simulation was carried out to simulate flow along the entire cross sections. It showed that a poor model convergence was occurred in six iterations (see Table 3). In other word model convergence criteria was not met for one or more time steps during the run. The possible causes include the lack of cross section data. The ISIS Flow was unable to compute the values because of the water level rose more than 3 m above the maximum level of cross section data. To solve this problem, a solution was made assuming an extra 3 metres wall at the maximum breadth.

The sections in which the poor model convergence occurred were expanded to continue the simulations. Expansion was conducted by considering the real condition of the locations. In this case, ISIS Flow used a simplified method to compute the solution at those cross sections. Expansion of cross sections was to minimise the loss of accuracy. Another important factor was the data given in the boundary condition. This must encompass the simulation start and finish times. The optimum time step was determined by carrying out trial runs with a range of time steps. In this simulation the largest time step at which the results did not change between runs was adopted. Run time step of 20 seconds was then selected and simulation was carried out from the start time 1.00 hour to finish time 36.00 hours.

It can be seen from the simulation that the maximum difference of discharge is 69.852 m^3/s. This occurred at Sect27, the downstream end of the reach. The maximum difference of stage has been found at Sect25, changing from 66.061 m AOD at the maximum discharge of 251.161 m^3/s to the minimum stage of 65.651 m AOD where the flow discharge decreased to 199.278 m^3/s. The difference means the fluctuation of water level at this section was 0.410 m.

Similar to that found in steady run, Sect17 also produced the highest water depths at both maximum and minimum discharges. Table 4 shows that the maximum and the minimum discharges at this point are 251.164 m^3/s and 227.753 m^3/s respectively. The maximum discharge produced a considerable value of 4.028 m in water depth whilst the minimum discharge had water depth of 3.686 m. These two values are 50 % higher than the average values in maximum and minimum category.

Table 3. An initial six steady iterations for direct unsteady run where the poor model convergence occurred

Itera-tion	Time (hours)	Q_{RATIO}	H_{RATIO}	δQ_{max}	δH_{max}
1	1.00	-1.04×10^{-1} (at Sect26)	4.40×10^{-4} (at Sect11)	8.09×10^{-2} (at Sect26)	4.35×10^{-4} (at Sect19)
2	1.00	-6.48×10^{-2} (at Sect17)	-1.12×10^{-3} (at Sect22)	5.70×10^{-2} (at Sect17)	2.67×10^{-4} (at Sect18)
3	1.00	-7.33×10^{-2} (at Sect27)	-1.12×10^{-3} (at Sect22)	1.29×10^{-1} (at Sect22)	5.42×10^{-4} (at Sect14)
4	1.00	-9.00×10^{-2} (at Sect27)	-4.25×10^{-4} (at Sect27)	7.08×10^{-2} (at Sect27)	1.68×10^{-4} (at Sect14)
5	1.00	-1.39×10^{-1} (at Sect27)	-1.02×10^{-3} (at Sect19)	1.03×10^{-1} (at Sect16)	4.65×10^{-4} (at Sect18)
6	1.00	-1.58×10^{-1} (at Sect27)	-7.17×10^{-4} (at Sect19)	1.08×10^{-1} (at Sect22)	4.04×10^{-4} (at Sect21)

The lowest water depths for both maximum and minimum discharges were found at Sect14. The water depth at maximum discharge of 251.166 m^3/s was 1.723 m, or about 66 % of the average water depth of the maximum category. The water depth slightly decreased to 1.613 m at the minimum discharge of 237.966 m^3/s. Unlike at Sect17 the difference in water depth at Sect14 was relatively small. It was 0.110 m whilst Sect17 had 0.342 m. Water profiles of maximum and minimum discharges along the reach are shown in Figure 4.

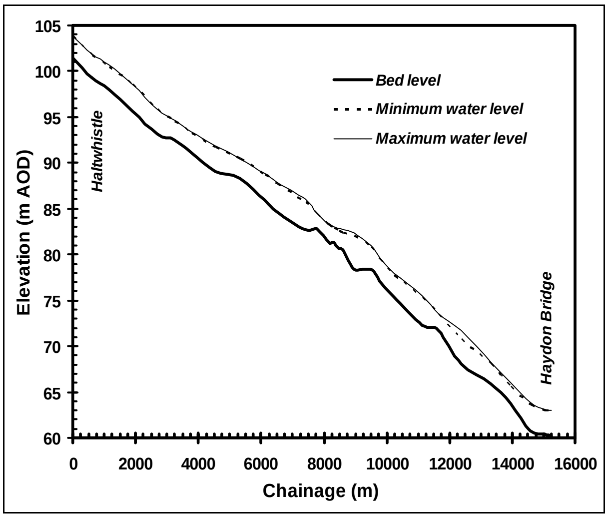

Fig. 4. Maximum and minimum stage for unsteady run from time 1.00 to time 36.00 hours

Careful consideration must be taken into account at points where the difference between the maximum and the minimum water depth are considerably high. This is particularly important in order to design proper but economical flood protection in the right places along the reach. The highest difference of water depth was at Sect25 in which the largest fluctuation range of stage was found. The maximum and the minimum stage at this section were 66.061 m and 65.651 m AOD respectively. This resulted in a notable 0.410 m difference in water depth between maximum and minimum category. The simulation also suggested the velocity vary between each section. The largest difference occurred at Sect26 with the maximum and the minimum velocities were 3.109 m/s and 2.356 m/s respectively.

Conclusions and Recommendations

Conclusions

The maximum number of nodes and the maximum number of unit for the version of ISIS Flow were important factors in hydraulic calculation. To ensure adequate spatial resolution, two interpolated sections were added because of sparse cross sections data particularly in the condition that channel properties vary radically between sections.

In the situation where the boundary data consists of two types of boundaries, flow time boundary (QTBDY) at the upstream and head time boundary (HTBDY) at the downstream end of the network, only certain values have been accepted for unsteady run. After various attempts were made for simulations, particularly expansion of cross sections where poor model convergence occurred, the maximum values of discharge and stage from rating curve were accepted. These values were used as initial condition for unsteady run where an initial six steady iterations were calculated to smooth the transition from the steady methods.

The fluctuation ranges of stage vary for each node. The steady run produced the range of water depth between 35 % less to 50 % increased from the average value of 2.484 m. In this run the maximum water depth occurred at Sect17 and the minimum at Sect14. The unsteady run also produced the maximum and the minimum water depths at the same sections found in the steady run with a wider fluctuation range of water depths. The range started from 50 % less to 53 % increased from the average value of 2.617 m in the maximum category and from 34 % less to 66 % increased from the average value of 2.429 m in the minimum category. The maximum discharge at Sect17 created a notable 4.028 m in water depth.

The variation in the fluctuation ranges allows a proper flood protection scheme to be applied. Detailed information of stage from each node is also very beneficial in the process of designing an economical flood protection plan.

Table 4. Maximum and minimum of all variables for unsteady run using Direct Method from time 1.00 to time 36.00 hours.

Label	Bed level (mAOD)	Maximum			Minimum		
		Discharge (m^3/s)	Stage (mAOD)	Velocity (m/s)	Discharge (m^3/s)	Stage (mAOD)	Velocity (m/s)
Sect1	101.430	251.153	103.797	2.417	251.153	103.764	2.368
Sect2	99.260	251.156	101.841	2.462	250.886	101.824	2.446
Sect3	98.040	251.158	100.712	2.198	250.079	100.492	2.001
Sect4	95.680	251.160	98.565	3.017	248.410	98.542	2.981
Sect5	93.160	251.164	95.875	2.865	246.627	95.823	2.779
Sect6	92.470	251.165	94.675	1.789	244.233	94.571	1.680
Sect7	91.070	251.164	93.334	2.427	241.787	93.225	2.358
Sect8	89.050	251.168	91.901	1.774	239.992	91.760	1.635
Sect9	88.300	251.165	90.509	2.016	239.770	90.457	1.982
9010	85.910	251.168	88.813	2.122	241.261	88.755	2.064
Sect10	84.700	251.166	87.880	2.426	241.137	87.769	2.375
Sect11	82.710	251.166	86.054	2.337	240.828	85.689	1.953
Sect12	82.780	251.166	84.543	2.835	240.919	84.445	2.717
Sect13	81.320	251.169	83.291	1.483	239.909	83.221	1.396
Sect14	81.340	251.166	83.063	1.453	237.996	82.953	1.317
Sect15	80.700	251.165	82.836	1.552	234.998	82.584	1.257
Sect16	80.420	251.165	82.719	1.245	231.850	82.393	1.043
Sect17	78.400	251.164	82.428	1.850	227.753	82.086	1.583
Sect18	78.400	251.162	80.975	2.880	224.418	80.798	2.710
Sect19	77.080	251.163	79.679	3.414	223.181	79.506	3.237
Sect20	75.400	251.162	77.970	2.605	221.686	77.737	2.496
Sect21	72.290	251.160	75.553	2.421	225.470	75.255	1.957
Sect22	71.700	251.163	73.566	2.875	227.671	73.406	2.776
Sect23	68.090	251.164	71.730	2.074	223.635	70.718	1.371
Sect24	65.930	251.164	68.342	3.270	209.089	68.054	3.024
Sect25	63.720	251.161	66.061	2.282	199.278	65.651	2.001
Sect26	60.750	251.163	63.829	3.109	189.266	63.625	2.356
26027	60.413	251.170	63.092	1.306	182.835	63.062	0.961
Sect27	60.240	251.172	63.019	0.945	181.320	63.019	0.681

Recommendations

The simulations suggest that the results were obviously influenced by channel properties such as cross sectional data and selected initial conditions for simulations. It is therefore a sufficient data set and parameters related to the reach are necessary to improve the simulation and to get the more accurate result. However, parameters values should not be altered to improve stability without a technical justification based on the physical situation. The section spacing may also need to be reduced to avoid the small depth problem when modelling shallow flows if the averaging rule for the friction slope is fixed within the model.

It is also suggested that the selection of time step for unsteady runs must be carefully considered as too large time step caused the necessary detail in the hydrograph being missed or numerical instabilities occurred and the use of too small time step lead to excessive computation times. Sudden changes in chainage should also be avoided, since they may induce additional errors.

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