ONE DIMENSIONAL SIMULATION OF FLOOD LEVELS IN A TROPICAL RIVER SYSTEM USING HEC-2 MODEL

Thamer Ahmed Mohammed[1], Salim Said[2], Mohd. Zohadie Bardaie[3] and, ShahNor Basri[4]

Abstract

In the present study, the HEC-2 static hydraulic model was used to predict the flood levels along Linggi River in Seremban town, Malaysia. HEC-2 model is based on numerical solution of the one dimensional (1D) energy equation for the steady gradually varied flow using the iteration technique. Calibration and verification of the HEC-2 model were conducted using the recorded data for the Linggi River. After calibration, the model was applied to predict the water surface profiles for Q_{10}, Q_{30}, and Q_{100} along the watercourse of the Linggi River. The predicted water surface profiles were found to be in agreement with the recorded water surface profiles for Linggi River. The value of the maximum absolute error between the predicted water surface profile and the recorded water surface profile for a stretch of 600 m of Linggi River was found to be 100 mm while the minimum absolute error was 20 mm only. Accuracy of the computed water surface profiles for a river using HEC-2 model is affected by stream geometry, accurate Manning coefficient of roughness and stations interval. Since the modern survey technologies can give an acceptable accuracy in determining cross-sectional geometry of a river, so the sensitivity analysis of the model was limited mainly to the impact of the Manning coefficient of roughness.

Keywords

Flood simulation, Linggi River, HEC-2, calibration, Verification

[1] Associate Professor, Department of Civil Engineering, Faculty of Engineering, Universiti Putra Malaysia, 43400 UPM, Serdang, Selangor, Malaysia, Phone +603-89466352, FAX +86567129, *thamer@eng.upm.edu.my*

[2] Professor, Department of Civil Engineering, Faculty of Engineering, Universiti Malaysia Sarawak, 94300 Kota Samarahan, Sarawak, Malaysia, Phone +6082671000 ext. 557, FAX +6082672317, *ssalim@eng.unimas.my*

[3] Professor, Vice Chancellor, Universiti Putra Malaysia, 43400 UPM, Serdang, Selangor, Malaysia, Phone + 603-89466408, Fax +603-89466425, mzbd@eng.upm.edu.my

[4] Professor, Department of Aerospace Engineering, Faculty of Engineering, Universiti Putra Malaysia, 43400 UPM, Serdang, Selangor, Malaysia, Phone +603-89466388, FAX +603-86567099, sbb@eng.upm.edu.my

INTRODUCTION

Flood normally happen when the river flows are large enough to cause flooding of those areas that is less often covered by water than the main channel of the flowing river. The municipal and rural developments that are located in the flooded area will be partly or fully damaged. Flooding in the river system of a tropical region is mainly due to excessive rainfall in the basin. The worst flood in Malaysia was recorded in 1926 which has been described as having caused the most extensive damage to the natural environment. Subsequent major floods were recorded in 1931, 1947, 1954, 1957, 1967, and 1971. Floods of lesser magnitude also occurred in 1973, 1979 and 1983 (Ann 1994). As a result of advances in the numerical methods and computer technologies, many mathematical models were developed and used for hydraulic simulation of the flood. The hydraulic simulation of the flood in a river system usually includes the prediction of the flood width and depth along a river watercourse. This type of information is essential because it will help engineers to take precautionary measures in their designs to minimize the total flood damage especially at the downstream end. Hydraulic models that are used in the simulation can be classified into dynamic hydraulic models and static hydraulic models. This classification was based on the concept and the approach used in the formulation of these models. Static hydraulic models for computing water surface profile in prismatic and non-prismatic channels were developed by Ishikawa (1984). Dynamic hydraulic models were developed by Lyness and Myers(1994), Molls and Chaudhary (1995) and Sturm and Sadiq (1996). Nik (1996) applied both HEC-2 static hydraulic model and MIKE 11 dynamic hydraulic model to predict the water surface elevation in Klang River, Malaysia and a difference of 5% was obtained between the two models. The effect of bed resistance on river rhine during flood was studied by Julien et al. (2002). Hall et al. (2005) conducted sensitivity analysis to flood inundation model calibration. In the present study, the HEC-2 static hydraulic model was calibrated, verified and then applied to predict the water surface profiles along the watercourse of the Linggi River system.

MODEL FORMULATION

The hydraulic simulation of the flow in a river, stream, or a drain is useful for many water resources projects. Knowledge about the water surface profile in nonprismatic channels is important specifically for flood plain management, flood mitigation and for analysis and design of the river crossing. In this study, the flow along nonprismatic channel was hydraulically simulated using mathematical model in order to predict the water surface profile along river watercourse in a tropical region. The hydraulic model used is known as HEC-2. The model is based on numerical solution of the one-dimensional energy equation which is applied for flow of water between two sections of a river reach. In the HEC-2 model both major and minor losses in energy occurred in a river reach were considered since these two types of energy losses are effective. The energy loss for the flow in a river is due to the friction loss, eddy loss and any other possible minor losses. To explain the mathematical algorithm, it is convenient to refer to the water surface for a natural channel above a datum at the two-end sections as shown in Figure 1. When the energy principles are applied for the two sections, we get the following equation:

$$z_2 + d_2 + \frac{\alpha_2 V_2^2}{2g} = z_1 + d_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \qquad (1)$$

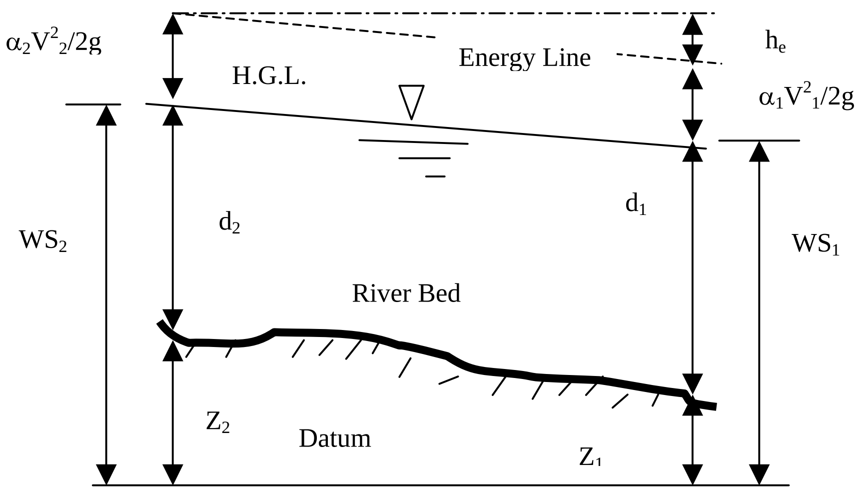

Figure 1: Profile of Natural River Reach with Two Stations

and $$WS_1 = d_1 + z_1 \qquad (2)$$

$$WS_2 = d_2 + z_2 \qquad (3)$$

Substituting eqn (2) and eqn (3) into eqn (1) we get:

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} \qquad (4)$$

where WS_1, WS_2 are water surface elevations from a datum for section 1 and section 2 respectively, d_1, d_2 are water depths at section 1 and section 2 respectively, z_1 , z_2 are the channel bed elevations above a datum at section 1 and section 2 respectively, V_1 , V_2 are average velocities (total discharge / total area of the flow at the section) at section 1 and section 2 respectively , α_1, α_2 are velocity weighting coefficients at section 1 and section 2 respectively, g is the acceleration due to gravity, and h_e is the energy loss in the reach.

Chow (1959) defines the energy loss in the reach of a river as the combined friction loss and eddy loss:

$$h_e = h_f + h_l \qquad (5)$$

where h_f is the friction loss and h_l is the eddy loss.

The eddy loss h_l is appreciable in nonprismatic channels and there is no available rational method of evaluating this loss. The eddy loss depends mainly on the velocity head change and may be expressed as shown below:

$$h_l = \theta(\alpha_2 \frac{V_2^2}{2g} - \alpha_1 \frac{V_1^2}{2g}) \tag{6}$$

where θ is the eddy loss coefficient.

For gradually converging and diverging reaches, θ = 0 to 0.1 and 0.2, respectively. For abrupt expansions and contractions, θ is about 0.5. For Prismatic and regular channels, the eddy loss is practically zero, or θ =0. For nonprismatic channel, friction loss can be described by the following formula:

$$h_f = L \bar{S} f \tag{7}$$

The discharge-weighted reach length L in Equation (7) is computed by weighting lengths in the left overbank, channel, and right overbank with their respective flows at the end of the reach. This length is described by HEC (1991) as below:

$$L = \frac{L_l \bar{Q}_l + L_c \bar{Q}_c + L_r \bar{Q}_r}{\bar{Q}_l + \bar{Q}_c + \bar{Q}_r} \tag{8}$$

A representative friction slope $\bar{S}_f$ is expressed as follows:

$$\bar{S} f = (\frac{Q_{T1} + Q_{T2}}{K_{T1} + K_{T2}})^2 \tag{9}$$

where: L_1, L_c, L_r are reach lengths specified for flow in left over bank , main channel and right overbank, respectively, $\bar{Q}_l, \bar{Q}_c, \bar{Q}_r$ are arithmetic average of the flows at the ends of the reach for left overbank, main channel, and right overbank, respectively, Q_{T1}, Q_{T2} is the value of the total discharge at section 1 and 2 respectively, K_{T1}, K_{T2} is the composite or total conveyance for section 1 and 2 respectively.

By substituting Equation (8) and Equation (9) into Equation (7), we get:

$$h_f = (\frac{L_l \bar{Q}_l + L_c \bar{Q}_c + L_r \bar{Q}_r}{\bar{Q}_l + \bar{Q}_c + \bar{Q}_r})(\frac{Q_{T1} + Q_{T2}}{K_{T!} + K_{T2}}) \tag{10}$$

Total energy loss in a river reach h_e can be obtained by substitute Equation (10) and Equation (6) into Equation (5):

$$h_e = (\frac{L_l \bar{Q}_l + L_c \bar{Q}_c + L_r \bar{Q}_r}{\bar{Q}_l + \bar{Q}_c + \bar{Q}_r})(\frac{Q_{T1} + Q_{T2}}{K_{T1} + K_{T2}})^2 + \theta(\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}) \tag{11}$$

The total conveyance of a river section can be described as below:

$$K_{Ti} = \frac{Q_{Ti}}{\sqrt{S_i}} \qquad (i=1,2,3....N) \tag{12}$$

If the river section is divided to N number of subsections, the total conveyance is the sum of the conveyance for the subsections as shown below:

$$K_{Ti} = k_1 + k_2 + k_3 + ... + k_N \tag{13}$$

where k_1,k_2, k_3, .. ,k_N are the conveyance for the subsection number 1, 2, 3, …,N

To simplify the calculation, natural section is divided into three main subsections namely the right, central and left as shown in Figure 2. Equation (13) can be simplified into the following form:

$$K_{Ti} = k_l + k_c + k_r \tag{14}$$

where k_l, k_c and k_r are the conveyance of the left subsection, central channel, and right subsection respectively.

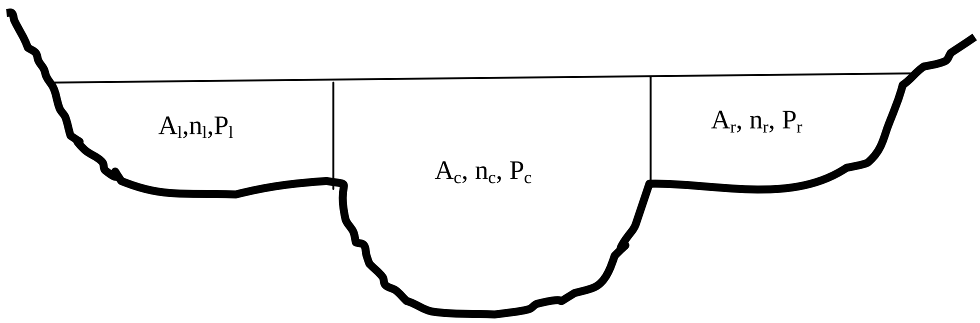

Figure 2: Division of the Flooded Natural Cross Section into Discrete Elements

From Manning formula, the conveyance of each subsection can be written as:

$$k_j = \frac{1}{n_j} A_j R_j^{2/3} \qquad (j=l, c, r) \tag{15}$$

where: l,c, and r denote the left subsection, central subsection and right subsection
By substituting Equation (15) into Equation (14), we get:

$$K_{Ti} = \frac{1}{n_l} A_l R_l^{2/3} + \frac{1}{n_c} A_c R_c^{2/3} + \frac{1}{n_r} A_r R_r^{2/3} \tag{16}$$

where n_l, n_c, and n_r is Manning coefficient of roughness for left overbank, central channel, and right overbank respectively.

The velocity coefficient α at any river section can be written as:

$$\alpha = (\frac{A_T^2}{K_T^3})(\frac{k_l^3}{A_l^2} + \frac{k_c^3}{A_c^2} + \frac{k_r^3}{A_r^2}) \tag{17}$$

where A_T is the total area of cross section, A_l, A_c, A_r are flow area at left overbank, main channel and right overbank, respectively.

In Equation (17), the difference in velocity heads between the main channel and the overbank sections are taken into consideration. The average velocity at a section can be described by:

$$V_i = \frac{Q_{Ti}}{A_{Ti}} \tag{18}$$

By substituting Equation (18), Equation (17) and Equation (11) into Equation (4) and after simplifying, we get:

$$WS_2 = WS_1 + \left(\frac{1-\theta}{2g}\right)\left(\left(\frac{Q_{T1}^2}{K_{T1}^3}\right)\left(\frac{k_{l1}^3}{A_{l1}^2} + \frac{k_{c1}^3}{A_{c1}^2} + \frac{k_{r1}^3}{A_{r1}^2}\right) - \left(\frac{Q_{T2}^2}{K_{T2}^3}\right)\left(\frac{k_{l2}^3}{A_{l2}^2} + \frac{k_{c2}^3}{A_{c2}^2} + \frac{k_{r2}^3}{A_{r2}^2}\right)\right) + \left(\frac{L_l \bar{Q}_l + L_c \bar{Q}_c + L_r \bar{Q}_r}{\bar{Q}_l + \bar{Q}_c + \bar{Q}_r}\right)\left(\frac{Q_{T1} + Q_{T2}}{K_{T1} + K_{T2}}\right)^2 \tag{19}$$

COMPUTATIONAL METHOD

Equation (19) describes the HEC-2 model which can be used to predict the water surface profile along a river watercourse for known values of streamflow, and the Manning coefficient of roughness. On the other hand the section geometry along the river watercourse must be defined in the model computations. Solving Equation (19) numerically using the iteration technique performs computations of the HEC-2 model, but it is difficult and time consuming to do the numerical computations manually. The Hydrologic Engineering Center (HEC) in California, USA produced HEC-2 computer package which can be used to perform all the necessary computations needed to simulate the flood in a river system. The numerical implementation of Equation (19) can be summarized by the following steps:

1. Assume a water surface elevation at the upstream cross section WS_2 for subcritical flow in the river channel while the SW_1 is known.
2. Based on the assumed water surface elevation, determine the corresponding total conveyance. The determinations of the areas and the conveyance for subsections are important for model application.
3. Solve Equation (19) for SW_2 and compare the computed value of SW_2 with the value assumed in step 1; repeat steps 1 to 3 until the value agree within 0.01m accuracy. The calculated SW_2 will be used as SW_1 for the computation of the water surface elevation to the next upstream section.

LINGGI RIVER

Linggi is the major river in the state of Negeri Sembilan, Malaysia. Linggi River discharges to the sea (Straits of Malaca) through an outlet in Port Dickson, which is located approximately 53 km downstream from the city of Seremban which is the commercial and administrative capital and the major city of the state of Negeri Sembilan. Seremban is located approximately 70 km south of Kuala Lumpur at latitude 2.75 degree north and longitude 101.9 degree east. It is situated alongside the main Kuala Lumpur - Singapore highway and the railway line. The upstream basin of the Linggi River system is located in Seremban town. The Linggi River system passes through residential, commercial, industrial and agricultural areas within the Seremban town. There are several areas within the city of Seremban that experience flooding due to the high flow in the Linggi River basin during the rainy season. It is important to control the flooding in Seremban town to reduce flood damage. According to the survey data the average slope of the Linggi River is 1/500 while the length it length until the control point is 5688 m.

DATA ACQUISITION

The data needed for the present study was obtained from the Hydrology Section of the Drainage and Irrigation Department (DID) in Kuala Lumpur, Malaysia. The data acquired can be categorized as: (a) the streamflow records, (b) the stage records, (c) the longitudinal section and (d) the cross sections at 50 m intervals.

MODEL CALIBRATION AND VERIFICATION

The calibration of HEC-2 model involves the accurate estimation of the empirical hydraulic coefficients so that the flow events simulated on the model will produce as closely as possible to the comparable natural events. On the other hand it is necessary to use the boundary condition of the studied watercourse in the model. For backwater computation of the subcritical flow, the water level at downstream control section is considered as a boundary condition to the HEC-2 model. This can be tackled by using the rating curve at this section. The other boundary condition that must be involved is tributary inflow to the main river. In the calibration process, the consideration of various values of the incoming flow from tributaries to the main river will help the modeller to get accurate estimation for roughness coefficients along the main watercourse. For Linggi River, the eddy loss coefficient, θ and Manning coefficient of roughness, n were estimated based on field measurement of the water surface profile of Linggi River for a stretch of 600 m for different discharge values including all boundary conditions. The estimation is based on the application of the energy equation and values of the eddy loss coefficients for Linggi River were found within the range of values given by Chow (1959). The Manning formula was used to estimate the Manning coefficient of roughness along the watercourse of the studied stretch for Linggi River at different water levels. The estimated values of the Manning roughness coefficient for the Linggi River were varied from 0.03 to 0.032 for the central channel only. For grass turfed banks the value of the Manning roughness coefficient varied from 0.032 to 0.04 depending on the condition of the bank. The measured water surface profile for the Linggi River was used in the verification process of the HEC-2 model. The recorded discharge of Linggi River was equal to 36.2 m^3/s and value of the Manning coefficient of roughness for main watercourse of the studied stretch was 0.032. The HEC-2 model used to predict the water surface profile for 600 m stretch of Linggi River and the predicted water surface profile was compared with the measured one as shown in Figure 3. The absolute error in the predicted water surface profile for Linggi River was computed and the maximum absolute error was found to be 100 mm while minimum absolute error was 20 mm only.

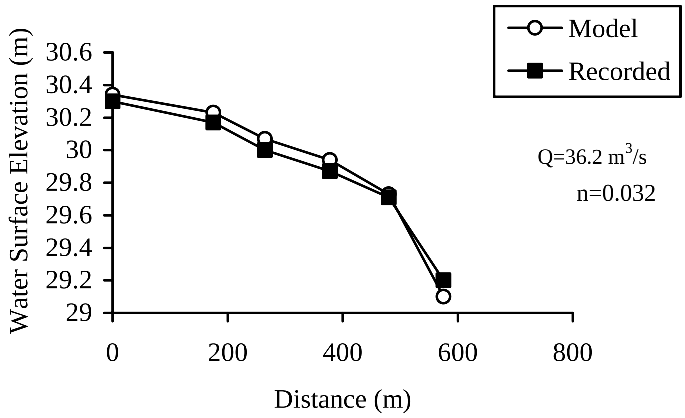

Figure 3: Measured and Predicted Water Surface Elevation of the Linggi River

SENSITIVITY ANALYSIS

Accuracy of the computed water surface profiles for a river using HEC-2 model were mainly affected by the accuracy of stream geometry, Manning coefficient of roughness and interval between stations along a river. Since the modern survey technologies can give an acceptable accuracy in determining cross-sectional geometry of a river, so the sensitivity analysis was limited to study the impact of Manning coefficient of roughness and intervals between the cross sections or stations on the accuracy of the predicted water surface profile for Linggi River using HEC-2 model. For constant interval of 50 m between stations along a stretch of 5.0 km of Linggi River, values of the Manning coefficient for main channel, right and left banks were increased every time by 0.001, 0.002, 0.003, 0.004, 0.005, and 0.006 and it found that this increment significantly affected the predicted water surface profile for Linggi River. The average increment in the predicted water level due to this change in Manning coefficient of roughness was found to be 2.5 m. Figure 4 shows the impact of the variation in the Manning coefficient of roughness on the predicted water surface elevations at 1 km intervals along the studied stretch of the Linggi River. A difference of 51 cm, 37 cm, 7 cm, 3 cm in the predicted water surface elevation at the most upstream section were obtained by running the HEC-2 model for a stretch of 5688 m from Linggi River using intervals of 1000 m, 500 m, 200 m, and 100 m receptively. The predicted water surface profile for Linggi River at 50 m intervals was used as base line for the above comparison.

MODEL APPLICATION

The water surface profile for the Linggi River was predicted using Q_{100}, Q_{10}, and Q_2 respectively. The value of the 100-year reoccurrence interval (ARI) flood for Linngi River is 100 m^3/s while the floods 10-year and 2-year reoccurrence floods are 57.2 m^3/s

and 32.7 m^3/s respectively. Cross sections at 50 m interval along the watercourse of the Linggi River for a reach of 5.688 km were used in the input data to the HEC-2 model. The value of Manning coefficient of roughness being used for central channel was 0.030 while a value of 0.035 was used for both the right overbank and left overbank. Figure 5 shows the predicted water surface profiles for Linggi River.

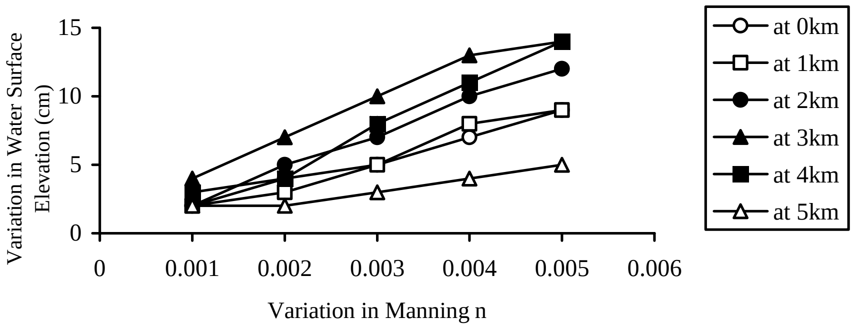

Figure 4: Effect of Manning Roughness on the Predicted Water Surface Elevation Along the Linggi River

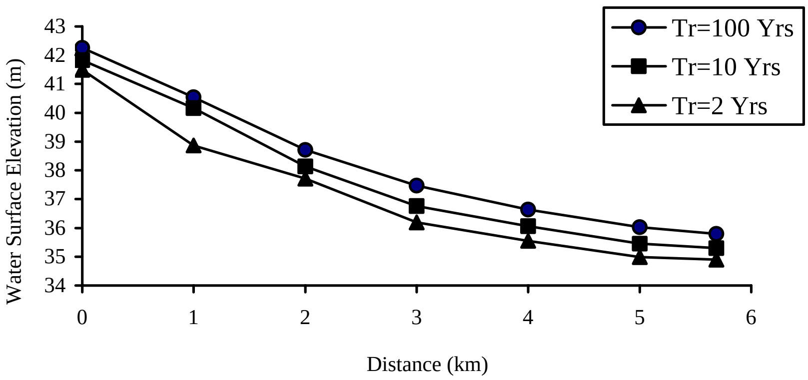

Figure 5: Predicted Water Surface Elevation for the Linggi River

CONCLUSION

The application of the HEC-2 model to the Linggi River system in Seremban, Malaysia showed that the predicted water surface profile and recorded water surface profile were in agreement. The absolute error in the predicted water surface profile for Linggi River was found to be ranging from 100 mm to 20 mm only (within 5%). So, the HEC-2 model can be applied successfully to simulate the water surface profile along the watercourse of tropical river systems with a reasonable error.

REFERENCES

Ann, O.C. (1994). "A Review of Irrigation, Drainage and Flood Control Projects In Malaysia" Proceedings of National Conference on Environmental Impact Assessment for Irrigation Drainage and Flood control, Kuala Terengganu, Malaysia, 1-14.

Chow, V.T. (1959). Open Channel Hydraulics. McGraw-Hill Company.

Hall, J. W., Tarantola, S., Bates, P. D., and Horritt, M. S. (2005). " Distributed Sensitivity Analysis of Flood Inundation Model Calibration" ASCE, Journal of Hydraulic Engineering, 131(2), 117-126.

Hydrologic Engineering Center. (1991). "HEC-2 Water Surface Profiles: User's Manual " U.S. Army Corps of Engineers, Davis, California, U.S.A.

Ishikawa, T. (1984). "Water Surface Profile of Stream with Side Overflow" ASCE, Journal of Hydraulic Engineering, 110(12), 1830-1840.

Julien, P. Y., Klaassen, G. J., Ten, W. B., and Wilbers, A. W. (2002). " Case Study: Bed Resistance of Rhine River During 1998 Flood" ASCE, Journal of Hydraulic Engineering, 128(1), 46-54.

Lyness, J.F. and Myers, W.R. (1994). " Velocity Coefficients for Overbank in a Compact Compound Channel and Their Effect on the Use of One Dimensional Flow Models" Proceedings of 2nd International Conference on Hydraulic Modelling, Stanford upon Avon, U.K.

Molls, T. and Chaudhry, M. H. (1995). " Depth-Averaged Open-Channel Flow Model" ASCE, Journal of Hydraulic Engineering, 121(6), 453-465.

Nik, A. (1996). "Klang River Improvement Works" Report submitted to the Department of Irrigation and Drainage, Kuala Lumpur, Malaysia.

Sturm, T. W. and Sadiq, A. (1996). "Water Surface Profiles in Compound Channel With Multiple Critical Depths" ASCE, Journal of Hydraulic Engineering, 122(12), 703-708.