EVALUATION OF ALLUVIAL RIVER STABILITY: CASE STUDY OF RAIA RIVER

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ABSTRACT

As a result of increasing economic growth of the country, areas within river catchment are being developed into new commercial, industrialization and housing purposes. Effect of this rapid urbanization has accelerated its impact on the hydrology and geomorphology. These developments have caused dramatical increase in the surface runoff and the behaviour of their sediment output hence resulting higher sediment yield.

Since any flood mitigation works would likely affect channel modification, knowledge of predicting the geometry changes involving the sediment transport movement to maintain the channel stability and design capacity are significant and necessary.

Comparisons on several empirical design methods including regime theory and mathematical model (FLUVIAL-12) were carried out for Raia River, a major tributary of Kinta River, in order to evaluate the appropriate method, which will minimize the morphological changes in river channel.

The results indicate that mathematical model (FLUVIAL-12) comprising component of water and sediment routing is capable of predicting instability effects such as significant erosion and sedimentation along river channel.

The simulation results from FLUVIAL-12 also indicate that this model is capable of producing stable design section based on maintaining the maximum section capacity and high bank stability.

1.0 INTRODUCTION

1.1 Characteristics of Natural Rivers

Materials comprising the beds of alluvial rivers have an important influence on river geometry. Bed slopes at the headwaters of rivers are steep, and the bed material is relatively coarse. In general, both river slope and bed material sizes decrease in the downstream direction.

Figure 1 shows bed material of Rio Grande River range in size from boulders and cobbles to silts and clays, generally decreasing in a downstream direction. On the Rio Grande, median particle size decreases from 0.5 mm at Otowi, New Mexico, to 0.14 at a point 200 miles downstream.

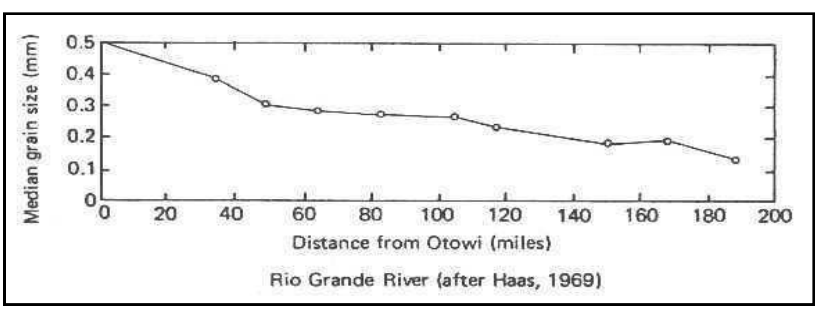

Figure 1: Grain Size Degradation of Rio Grande River (Peterson, 1986).

1.2 Bank Materials

Bank material normally changes with distance along a stream. It is important to note that banks are generally not composed of uniform materials throughout their height, but rather are stratified with layers of gravels, silts, sands and clays.

River banks may generally be classified as cohesive, noncohesive, and stratified (composed of layers of materials of different size, permeability, and cohesion characteristics).

1.3 Definition of Stable Section

Lane (1955) presented an excellent definition of stable or regime channels as follows;

"*A stable channel is an unlined earth canal for carrying water, the banks and bed of which are not scoured by the moving water and in which objectionable deposits of sediment do not occur*".

Thus from the definition, small amount of erosion and deposition may occur within river channels but for a long period of time, bank and bed will attained toward stability.

1.4 Method of Designing Stable Channel

Various procedures for designing stable alluvial channels have been developed in the past. These methods may be generally categorized as belonging to one of the following design criteria;

a) Maximum Permissible Velocity

The maximum permissible velocity or the nonerodible velocity is the greatest mean velocity that will not cause erosion of the channel body. In other word the maximum velocity, which the alluvial material can withstand without movement of the granular particles.

b) Permissible Tractive Force

Permissible tractive force is the maximum unit tractive force that will not cause serious erosion of the material forming the channel bed on a level surface. The theory relates the shearing force of the fluid on the banks to the geometry of the cross section and the weight of the individual particles.

c) Regime Theory

In the regime theory approach, relationships for the channel width, depth and slope were established based on measurements from stable alluvial canals and rivers in India and Pakistan.

2.0 MATHEMATICAL MODEL (FLUVIAL-12) FOR CHANNEL DESIGN

FLUVIAL-12 (Chang 1982, 1984, 1985) model has been formulated and developed for water and sediment routing in natural and man-made channels.

Briefly, this model, for a given flood hydrograph, simulates time and spatial variations in flood level, sediment transport, and bed topography. In the prediction of river-channel changes, scour and fill are tied in with width variation and the effect of secondary currents under the changing channel curvature. In the model, scour and fill are computed on the basis of longitudinal imbalance in sediment discharge.

3.0 METHODOLOGY

3.1 Case Study of Raia River

Figure 2 shows Raia River study area started from Kampong Tanjung Bridge (Ch. 2800 m) extends upstream for a distance of about 2.8 km.

Severe damages had occurred to several channel section during 1999 flood event (Figure 3).

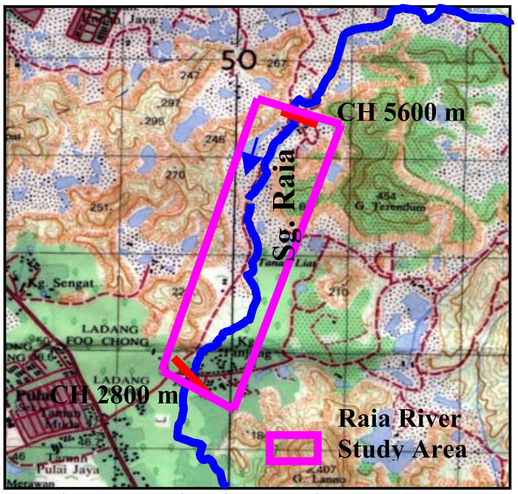

Figure 2: Raia River Study Area (DID Kinta/Batang Padang, 2001).

Figure 3: Water Level (45.950 m) at Kampong Tanjung Bridge During 1999 Flood Event (DID Kinta/Batang Padang, 2002).

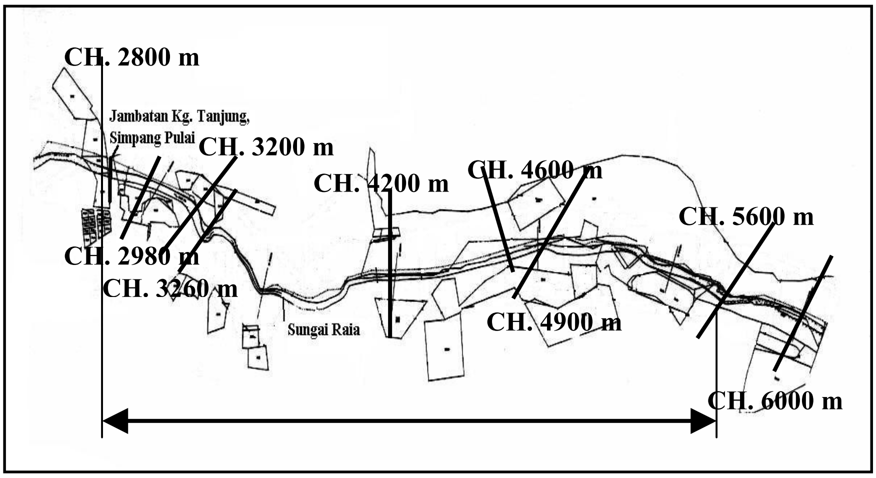

Figure 4: Study Reach of Raia River (DID Kinta/Batang Padang, 2001).

Bed and bank material as well as cross sections profile at selected gauging station shown in Figure 4 were taken for comparison with the simulated results.

3.2 Study Procedures

Simulation of mathematical model FLUVIAL-12 were carried out by using several input data such as hydrograph, rating curve obtained from DID. Bed materials for upstream and downstream were obtained from gauged station.

Cross sections used in the simulation process were obtained from DID survey plan in 1999 (for calibration purposes).

A model should be calibrated before it is used to simulate existing and hypothetical conditions. Calibration may be viewed as an iterative process in which an initial values of model parameters are set, the model is operated, the simulated output are compared with measured output, and if necessary, changes are made in input parameters, thus initiating the second cycle of the process. The process is repeated until a satisfactory correlation is obtained between simulated and measured output.

3.3 Calibration Process

For this study, highest water level 45.950 m on 6 January 1999 (Figure 3) obtained from gauging station at Kampong Tanjung Bridge and hydrograph profile (Figure 5), which generated the maximum discharge of 129.98 m3/s, were used for calibration purposes.

From the calibration process, the input parameters such as Manning's n, sediment transport equation and bank erodibility factor, which correlated to the highest water level, were identified.

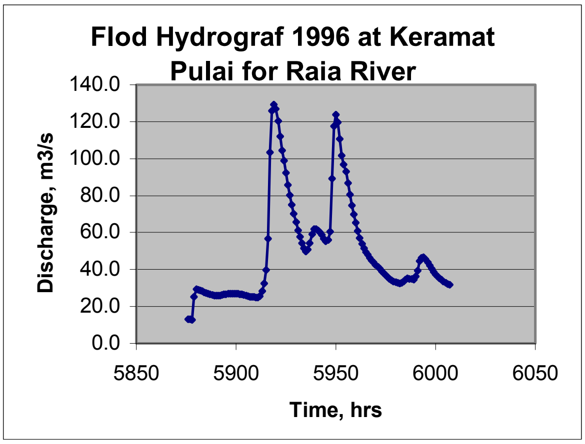

Figure 5: Flood Hydrograph Recorded at Gauging Station Keramat Pulai, Kampung Tanjung Bridge (DID Malaysia).

3.4 Calibration Results

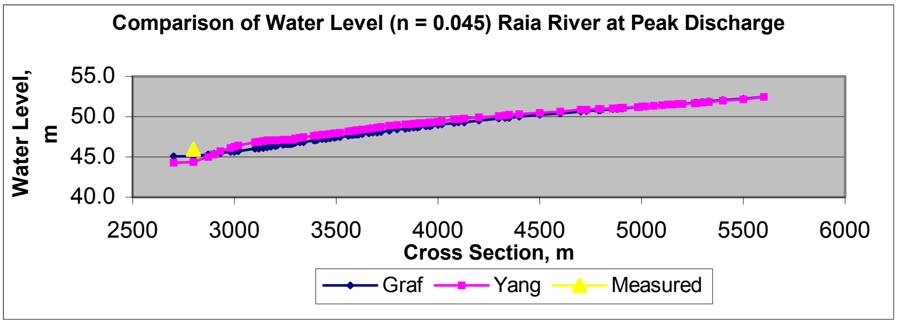

Figure 6: Simulated and Measured Water Surface Profile at Peak Discharge.

Selected simulated results during the calibration process are presented in Figure 6.0 and Table 1.0. In the model simulation, Graf equation for the sediment transport and Manning's n = 0.045 produces a close agreement with the measured water surface profile.

Table 1.0: Comparison of Water Level Using Six Sediment Transport Equation and Manning's n Coefficient.

No.	Sediment Tansport Equation	Chainage 2800 m Predicted Level Manning's n = 0.025
1.	Graf	44.250 m
2.	Yang	44.050 m
3.	Engelund-Hansen	44.210 m
4.	Parker	44.120 m
5.	Ackers-White	44.330 m
6.	Meyer-Peter-Muller	44.260 m
		Manning's n = 0.045
1.	Graf	45.070 m
2.	Yang	44.390 m
3.	Engelund-Hansen	44.810 m
4.	Parker	44.690 m
5.	Ackers-White	44.350 m
6.	Meyer-Peter-Muller	44.380 m
	Highest Water Level 6 January 1999	
1.	Measured Level	45.950 m

4.0 DESIGN CONFIGURATION OF RAIA RIVER

4.1 Selected Cross Section

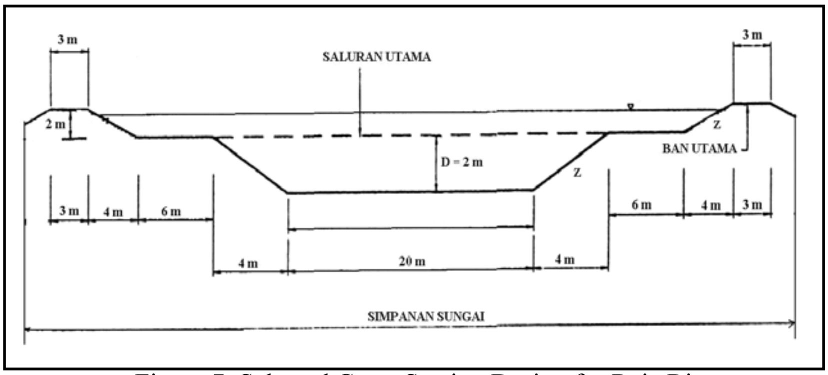

Figure 7: Selected Cross Section Design for Raia River.

Appropriate cross section was identified and selected to convey maximum discharge and furthermore the important task is to minimize the instability problem. Due to inadequate area at site, preliminary cross section as shown in Figure 7.0 was chosen.

4.2 Hydrology

a). Hydrograph

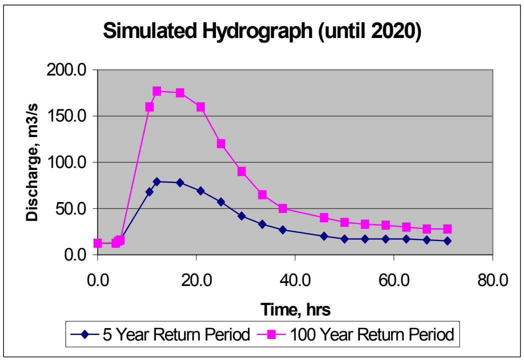

Figure 8: Simulated Hydrograph Until 2020 for Raia River.

Figure 8 shows predicted hydrograph based on landuse until 2020 that were used for design process

b). Flow Rating Curve

Rating curve for the simulation process was derived from downstream section using Manning's formula (Figure 9).

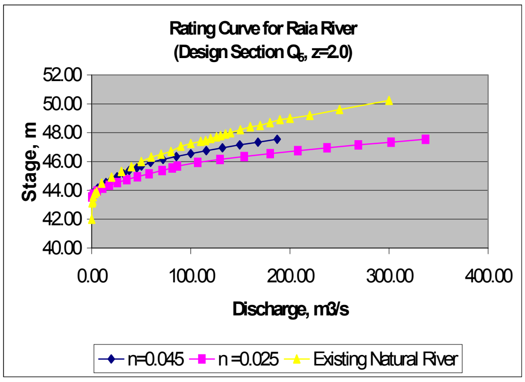

Figure 9: Flow Rating Curve for Raia River.

4.3 Bed Material

Bed material samples from each section i.e. downstream and upstream of design reach were used for simulation process (Figure 10). Each sample is divided into five size fractions, and the size for each fraction is represented by its geometric mean diameter.

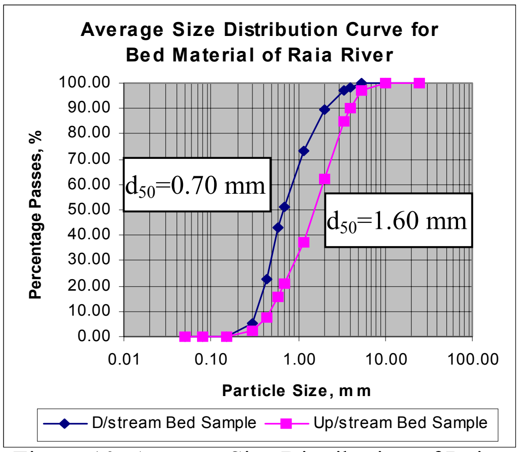

Figure 10: Average Size Distribution of Raia River Bed Material.

4.4 Simulation Process

The mathematical model, FLUVIAL-12 was employed to simulate and identified the instability problem occurred in the design reach especially at riverbank. A total of 89 cross sections were employed to represent the riverbed geometry. Graf's equation for sediment transport was used for this sand-bed river. Parameters used in the simulation process are as follows;

a). Different design cross section were used in the simulation process ranging from side slope of z = 2.0, z =1.5 and z = 1.0. The purpose of this process is to identify which section produces the best stable section that have a minimum erosion and sedimentation in the channel.

b). Comparison of two bank erodibility factor of Fh = 1.0 and Fh = 0.5 were also used to established the various changes occurred at the section and bank.

c). Roughness in terms of Manning 's n obtained from calibration results of 0.045 and 0.025 were used in the model process to identify the variation in the channel capacity.

5.0 SIMULATION RESULTS

Selected simulated results using mathematical model are presented as follows;

5.1 Bed Level Changes at Peak Discharge

Figure 11 and Table 2 indicate that the cross section side slope z = 2.0 demonstrate a minimum changes in bed level compared to cross section side slope z =1.5 and z = 1.0.

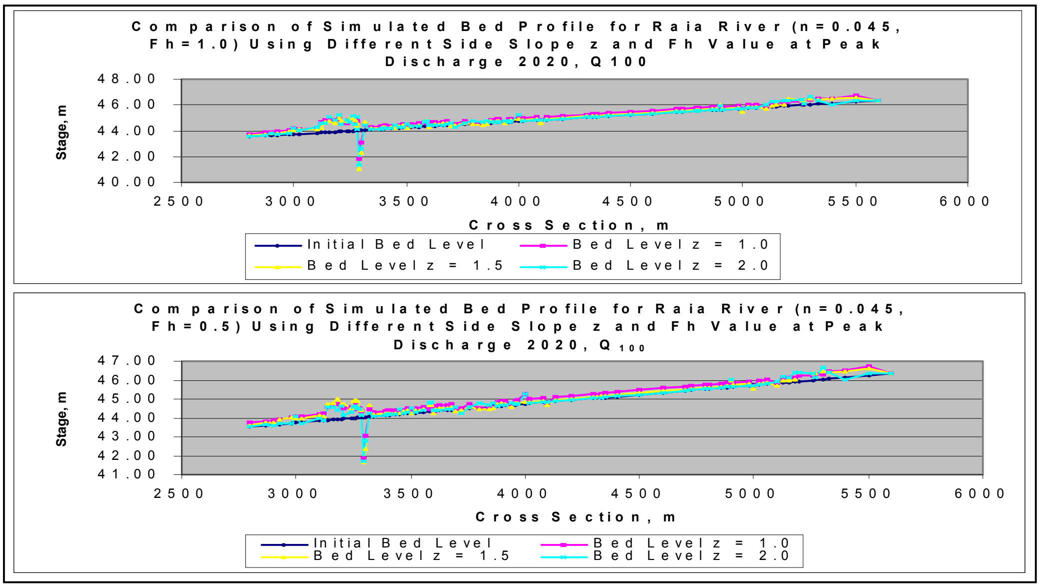

Figure 11: Simulated of Bed Level Changes Raia River (n = 0.045) Using Different Side Slope and Fh Value at Peak Discharges 2020, Q_{100}.

Table 2: Bed Level Variations of Selected Cross Section Raia River (n = 0.045) at Peak Discharges 2020, Q_{100}.

Cross Section, m	Initial Bed Level, m	Bed Level, z = 1.0		Bed Level, z = 1.5		Bed Level, z = 2.0	
		n=0.045, Fh=1.0	n=0.045, Fh=0.5	n=0.045, Fh=1.0	n=0.045, Fh=0.5	n=0.045, Fh=1.0	n=0.045, Fh=0.5
4900	45.630	45.950	45.880	46.000	45.980	45.920	46.010
4600	45.330	45.570	45.570	45.420	45.410	45.330	45.340
4200	44.940	45.160	45.160	45.010	45.010	44.940	44.940
3260	44.000	44.890	44.820	45.000	44.930	45.110	44.610
3200	43.940	44.920	44.640	44.960	44.740	45.200	44.620
2980	43.720	44.010	44.030	43.990	44.050	43.780	43.700

Figure 12.0 and Table 3.0 also indicate that the cross section side slope z = 2.0 demonstrate a minimum changes in bed level compared to cross section side slope z = 1.5 and z = 1.0 although different Manning's n were used. This also shows that the cross section side slope z = 2.0 produce a high degree of stability in the channel section.

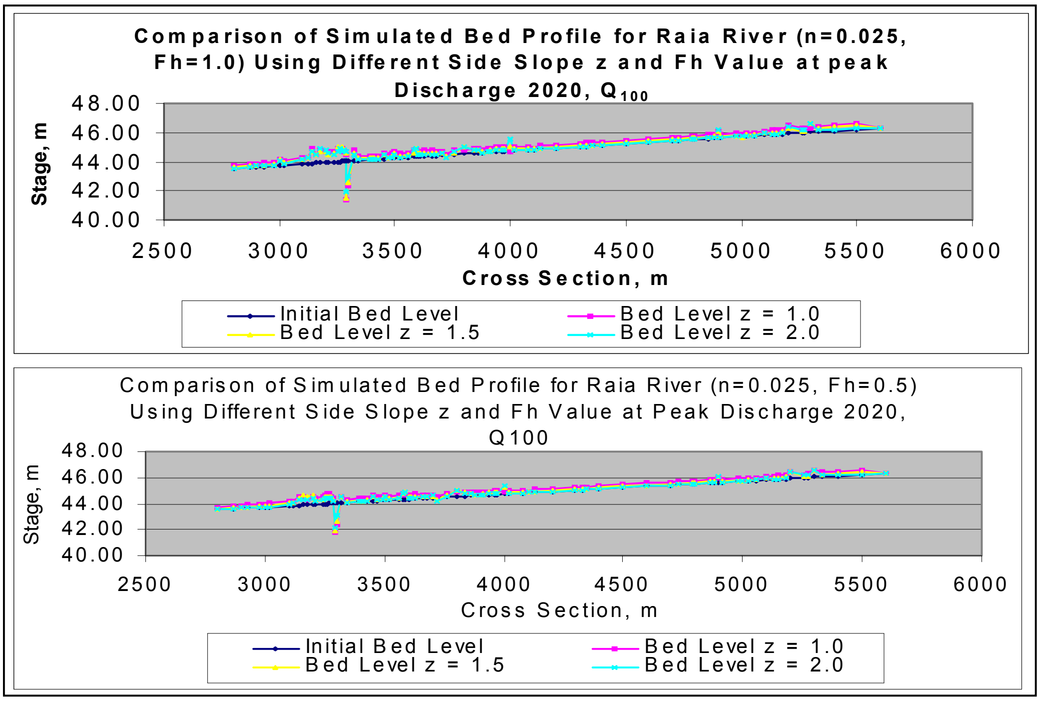

Figure 12.0: Simulated of Bed Level Changes Raia River (n = 0.025) Using Different Side Slope and Fh Value at Peak Discharges 2020, Q_{100}.

Table 3.0: Bed Level Variations of Selected Cross Section Raia River (n = 0.025) at Peak Discharges 2020, Q_{100}.

Cross Section, m	Initial Bed Level, m	Bed Level, z = 1.0		Bed Level, z = 1.5		Bed Level, z = 2.0	
		n=0.025, Fh=1.0	n=0.025, Fh=0.5	n=0.025, Fh=1.0	n=0.025, Fh=0.5	n=0.025, Fh=1.0	n=0.025, Fh=0.5
4900	45.630	46.060	46.010	46.130	46.140	46.140	46.110
4600	45.330	45.570	45.560	45.420	45.410	45.330	45.340
4200	44.940	45.170	45.160	45.010	45.020	44.950	44.950
3260	44.000	44.830	44.810	44.910	44.480	44.930	44.460
3200	43.940	44.700	44.590	44.670	44.600	44.840	44.450
2980	43.720	43.910	43.970	43.870	43.800	43.740	43.730

5.2 Water –Surface Profile Changes at Peak Discharge

Figure 13 and Table 4 clearly shows that waters-surface profile for cross section side slope z = 2.0 is below design bund level compared to cross section side slope z = 1.5 and z = 1.0 which above design bund level.

These indicate that cross-section side slope z = 2.0 at Manning's n = 0.045 is capable of carrying maximum discharge of 177 m^3/s compared to the other two cross section.

Figure 14 and Table 5 also shows that waters-surface profile for all cross section are below design bund level using Manning's n = 0.025.

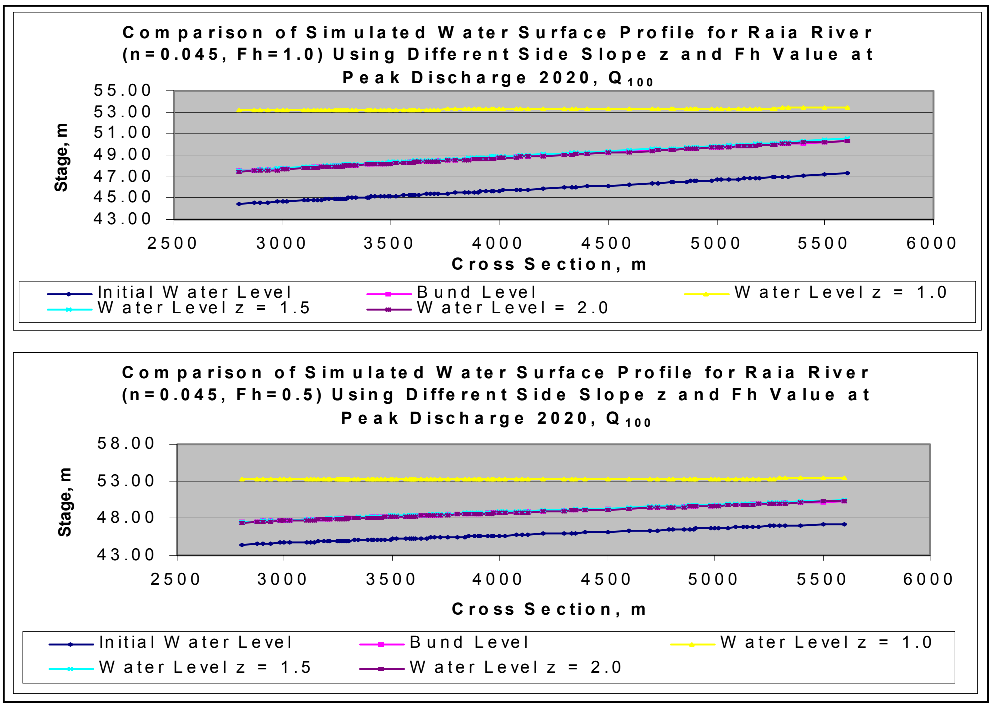

Figure 13: Simulated of Water-Surface Profile Changes Raia River (n = 0.045) Using Different Side Slope and Fh Value at Peak Discharges 2020, Q_{100}.

Table 4: Water-Surface Profile Variations of Selected Cross Section Raia River (n = 0.045) at Peak Discharges 2020, Q_{100}.

Cross Section, m	Bund Level, m	Initial Water Level, m	Water Level, z = 1.0		Water Level, z = 1.5		Water Level, z = 2.0	
			n=0.045, Fh=1.0	n=0.045, Fh=0.5	n=0.045, Fh=1.0	n=0.045, Fh=0.5	n=0.045, Fh=1.0	n=0.045, Fh=0.5
4900	49.630	46.570	53.340	53.340	49.760	49.760	49.600	49.590
4600	49.330	46.270	53.320	53.320	49.460	49.450	49.300	49.290
4200	48.940	45.880	53.290	53.290	49.060	49.060	48.920	48.900
3260	48.000	44.940	53.240	53.240	48.070	48.070	47.960	47.930
3200	47.940	44.880	53.240	53.240	48.000	48.000	47.870	47.860
2980	47.720	44.660	53.230	53.230	47.740	47.740	47.620	47.630

Table 5.0: Water-Surface Profile Variations of Selected Cross Section Raia River (n = 0.025) at Peak Discharges 2020, Q_{100}.

Cross Section, m	Bund Level, m	Initial Water Level, m	Water Level, z = 1.0		Water Level, z = 1.5		Water Level, z = 2.0	
			n=0.025, Fh=1.0	n=0.025, Fh=0.5	n=0.025, Fh=1.0	n=0.025, Fh=0.5	n=0.025, Fh=1.0	n=0.025, Fh=0.5
4900	49.630	46.570	48.850	48.850	48.740	48.740	48.660	48.650
4600	49.330	46.270	48.550	48.450	48.450	48.440	48.360	48.360
4200	48.940	45.880	48.180	48.170	48.080	48.070	48.000	47.990
3260	48.000	44.940	47.220	47.200	47.170	47.100	47.050	47.020
3200	47.940	44.880	47.110	47.110	47.050	47.030	46.950	46.960
2980	47.720	44.660	46.860	46.850	46.770	46.770	46.710	46.710

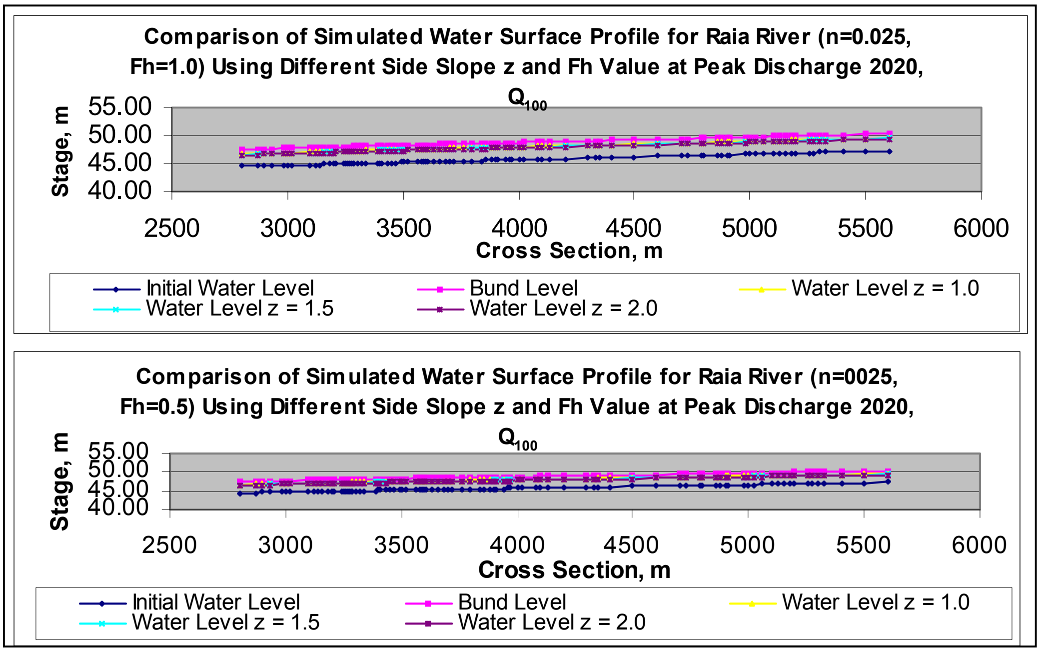

Figure 14.0: Simulated of Water-Surface Profile Changes Raia River (n = 0.025) Using Different Side Slope and Fh Value at Peak Discharges 2020, Q_{100}.

5.3 Cross-sectional Changes

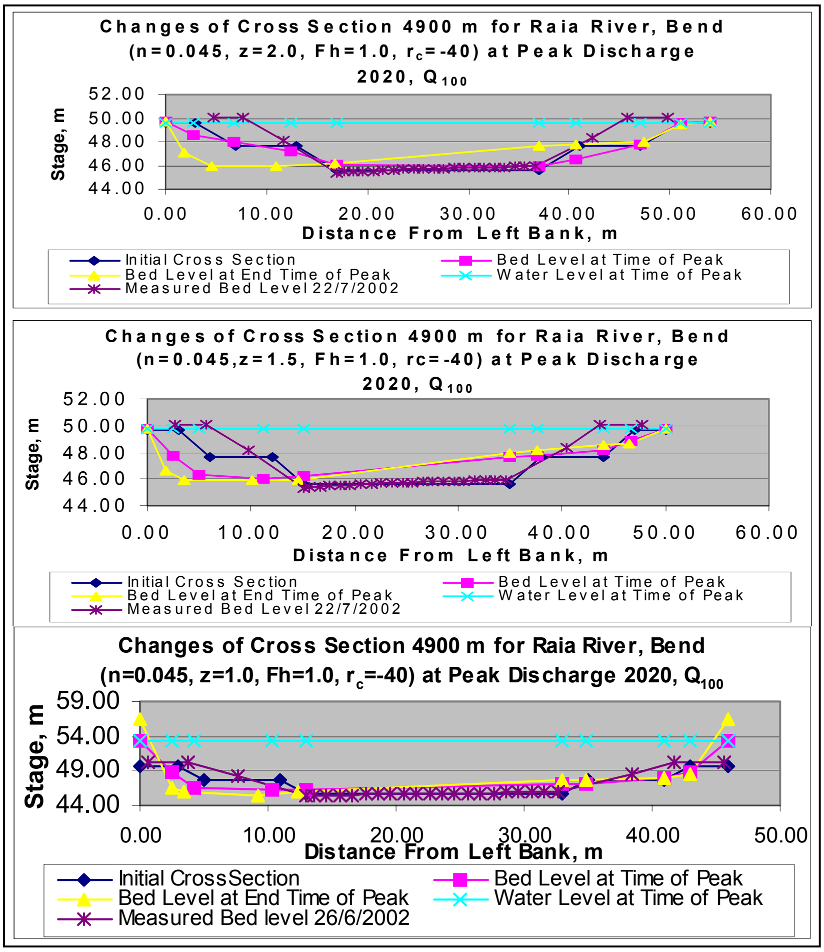

Figure 15: Simulated Cross-sectional Changes (n = 0.045) at Section 4900 m Using Different Side Slope z Compared to Existing Natural Raia River.

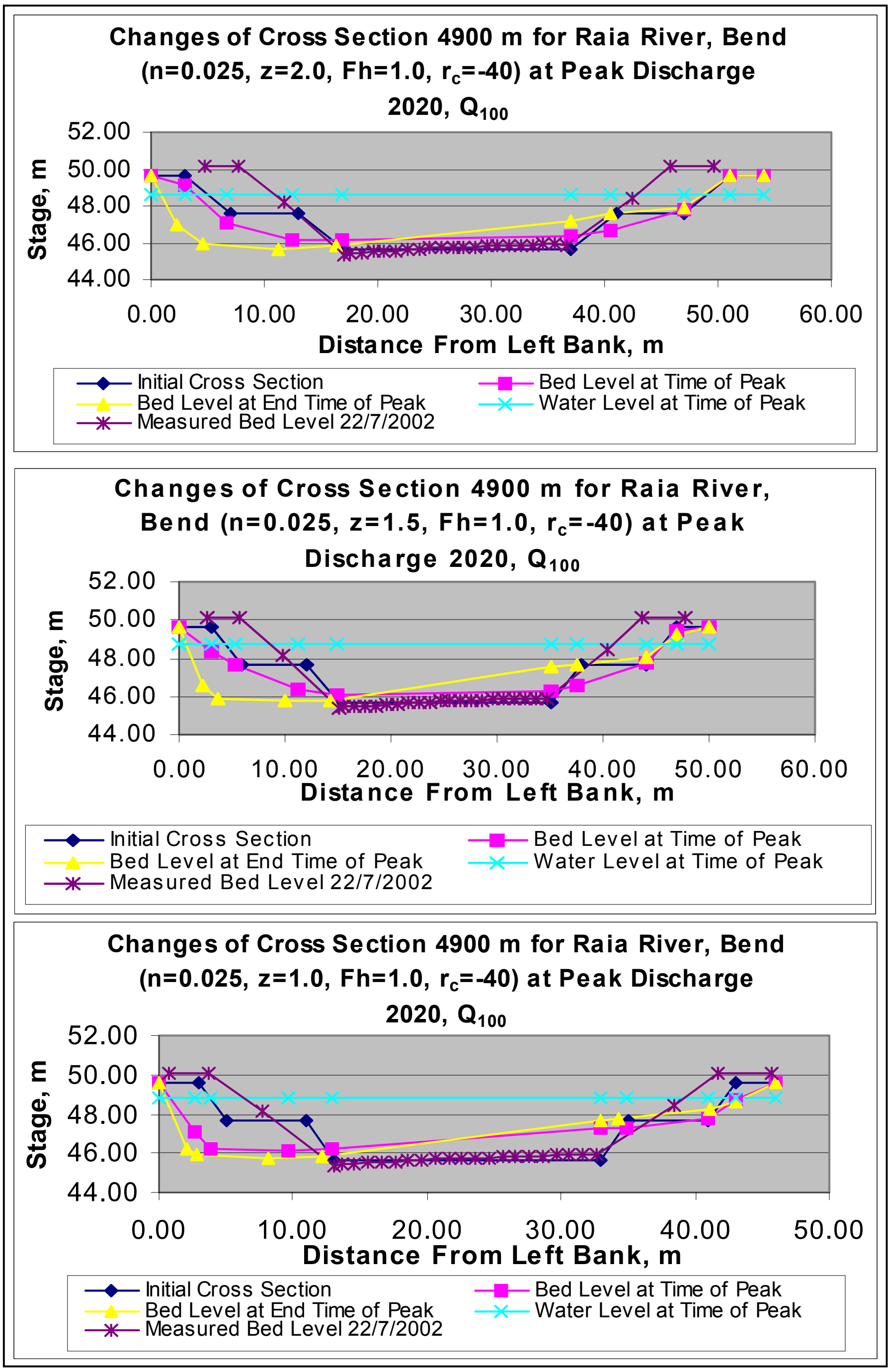

Figure 16.0: Simulated Cross-sectional Changes (n = 0.025) at Section 4900 m Using Different Side Slope z Compared to Existing Natural Raia River.

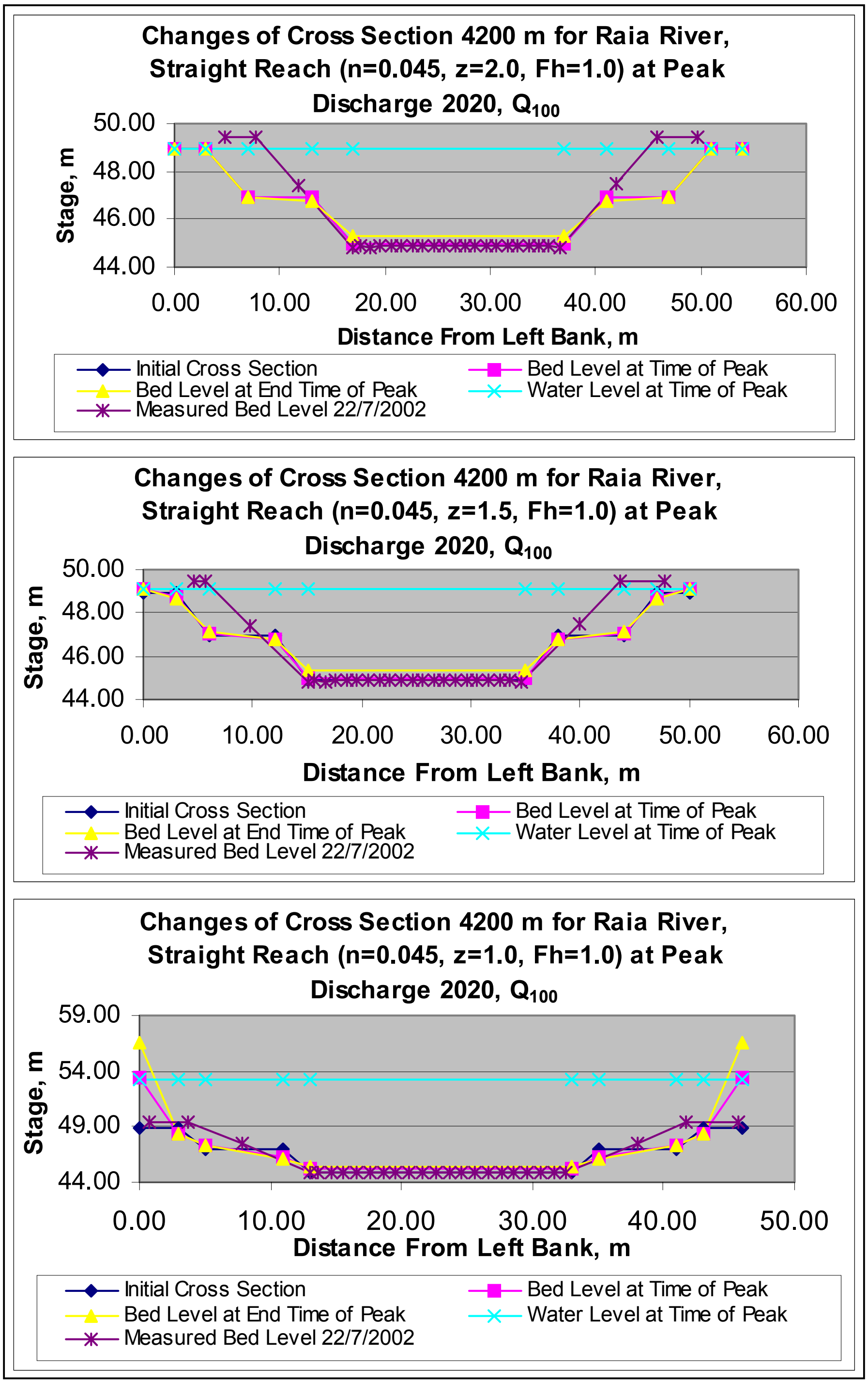

Figure 17.0: Simulated Cross-sectional Changes (n = 0.045) at Section 4200 m Using Different Side Slope z Compared to Existing Natural Raia River.

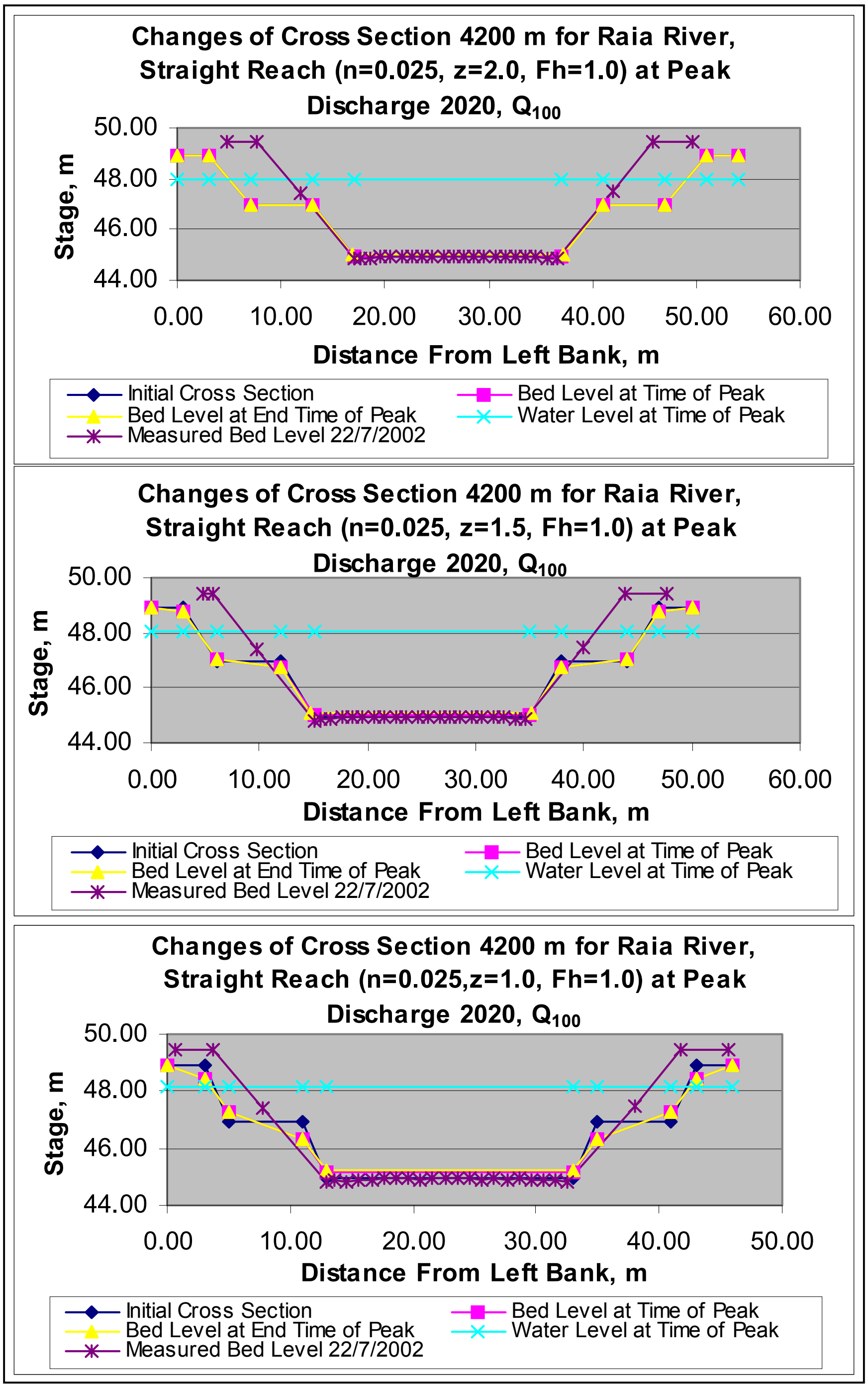

Figure 18.0: Simulated Cross-sectional Changes (n = 0.025) at Section 4200 m Using Different Side Slope z Compared to Existing Natural Raia River.

Selected simulated cross-sectional changes during the beginning, the peak discharge and end of flood are shown in Figure 15, 16, 17 and 18. These simulated cross-sectional profiles are compared with measured profile and are found that the cross section side slope z = 2.0 produce an agreement with the measured profiles. The figures above show that the straight reach demonstrates a high degree of stability than the curved reach. Although approximate sediment equilibrium is predicted for the design flood, the channel bed of a bend can still be subject to significant changes, as shown in Figure 15 and 16. This indicates that river bed scour is related to the flow curvature with the maximum scour occurring at the bend exit.

6.0 SUMMARY AND CONCLUSIONS

A mathematical model for water and sediment routing through alluvial channels was employed to simulate riverbed changes and instability problem during a specified flow, thereby providing the necessary information for the design or other bank protection work.

Simulated results show cross section with side slope of z = 2.0 is capable of carrying maximum discharge of 177 m^3/s and also demonstrate a minimum changes in bed level and high degree of stability.

Simulated results also show that channel-bed scour is affected by the channel curvature. The scour depth increases as flow enters a bend; maximum scour is generally reached at the bend exit, followed by a gradual decrease in transverse bed slope and scour depth with the decline in spiral motion.

So in designing channel section with bend, avoid having a smaller value of radius of curvature, r_c or provide an effective protection along the critical bend (Figure 19).

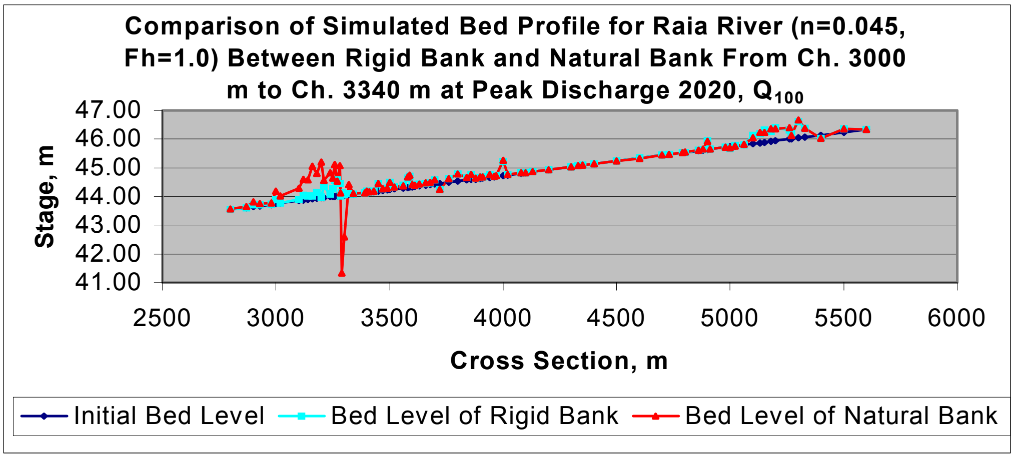

Figure 19: Simulated Bed Level Changes of Natural and Rigid Bank at Section 3000 m to 3340 m Raia River.

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