

Quasi-3D Approach For Evaluating Fluvial Erosion and Protection Measures in Meandering Alluvial Rivers

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ABSTRACT

The dynamic interaction between hydrologic events and fluvial geomorphology significantly influences sediment transport and bank stability in meandering alluvial rivers. This study investigates the erosion processes in Bear Brook Creek, Ontario, using a quasi-three-dimensional (quasi-3D) modeling approach that integrates one-dimensional (1D) JP Meander and two-dimensional (2D) FaSTMECH IRIC models. By focusing on the interaction between channel hydraulics and river morphodynamics, the research quantifies critical shear stress and soil erodibility through empirical formulations validated with site-specific data. To improve the applicability of these formulations, the study regionally normalizes the equations to account for local soil types and environmental conditions, providing more accurate predictions of erosion risk compared to generic policy-based guidelines. This approach highlights the need for technically driven modelling to determine the appropriate engineering interventions such as slope armoring and Iowa vanes to mitigate erosion. The findings contribute to enhanced riverbank management practices, highlighting the use of a coupled modelling technique to circumvent limited site data.

INTRODUCTION

Municipal drainage systems, comprising networks of swales and ditches, are crucial for water management and flood prevention in rural agricultural areas. These systems often interface with natural or modified watercourses, where agricultural interventions such as dredging can significantly alter environmental fluvial dynamics, in turn affecting localized fluvial geomorphology. Dredging operations, while beneficial for agricultural practices by modifying channel slopes, sedimentation levels, and course alignments, can pose significant threats to the structural integrity of nearby roadways due to natural meandering processes.

Fluvial erosion of riverbanks presents considerable challenges, impacting socioeconomic activities through the loss of arable land, damage to critical infrastructure, and increased risk in geomorphologic changes due to uncertain hydrologic events along basins and floodplains. As designers develop practical solutions, various government institutions such as the Ontario Ministry of Natural Resources develops policy based guides such as the Water Section's technical guide, River and Stream Systems: Erosion Hazard Limit to create qualitative baselines for these alluvial conditions. This technical guide based in Ontario, Canada outlines physical and environmental conditions that classify geologic and environmental characteristics of meandering streams. While policy-based approaches such as this are invaluable for establishing standardized hazard and mitigation procedures, they are limited by their respective qualitative approaches by omitting the characterisation of dynamic hydraulic responses due to the regional hydrology. Specifically, these qualitative approaches do not adequately address specific hydraulic behaviour, mechanistic boundaries, limitations, hydrological effects, and interdisciplinary considerations. To bridge this gap, it is important to employ detailed hydraulic evaluations to characterise specific hazards in order to determine the most appropriate mitigation solution. Using coupled one-dimensional (1D) and two-dimensional (2D) modeling techniques, quasi-three-dimensional (quasi-3D) hydrodynamic condition can aide in quantifying the extent of these geo-environmental hazards on a case by case basis. This approach provides an improved and more accurate understanding of fluvial processes with respect to their impact on specific infrastructure resilience cases.

BACKGROUND

This study addresses the critical issue related to the meander migration due to excess bank erosion, which continuously encroaches near a municipal rural road in the township of Bourget, Clarence-Rockland, Ontario, Canada. Recent slope instability have indicated combinations of slope failure and landslides despite mitigation efforts. As the bank continues to recede, slope failure paths continue to encroach towards the middle of an existing roadway which jeopardizes both safety and the lifecycle of the municipal asset. This in turn necessitates a thorough understanding of the underlying processes to evaluate potential mitigation strategies based on geomorphologic evolution. To this end, various hydraulic models are employed to simulate river flow, meandering, and stress behavior, focusing on software such as IRIC JP Meander, IRIC FaSTMECH, numerical methods, and flow characterisation modelling to identify governing hydraulic effects along the bank. Standard practice and approaches to this engineering problem typically involves the analysis of slope instability with respect to the infrastructure's life cycle assessment. However, when environmental impacts remain undetermined, a hydrodynamic analysis in an unassessed area can be used to propose mitigation solutions through Dynamic-process conservation areas Florsheim, Mount, and Chin, (2008).

The primary objective of this research study is to explore hydraulic modelling tools given limited data, and to quantify the applied shear stresses to estimate bank erosion rates. This study utilizes coupled hydrodynamic models with the excess shear equation to estimate bank erosion and mitigation measures that are critical to the infrastructures life cycle. A detailed analysis of the design advantages and limitations of slope armoring, Iowa vanes (or submerged river vanes), head control structures, and bend way weirs are evaluated to assess their efficacy in reducing the excess shear stresses below the thresholds required to maintain slope stability.

STUDY AREA

Bear Brook Creek, situated within the South Nation River Watershed in Eastern Ontario, Canada, serves as the focal point for this study. The creek, a tributary of the South Nation River, flows through the City of Clarence-Rockland and plays a crucial role in the local hydrology, impacting

both agricultural lands and ecological habitats. Spanning a catchment area of approximately 4,200 square kilometers, Bear Brook Creek traverses a diverse landscape that includes agricultural fields, forested areas, and urban developments. The creek's meandering patterns, with connecting municipal drain features are vital for maintaining the region's biodiversity and wetlands ecosystem.

The hydrology of BearBrook Creek is characterized by variable flow conditions throughout the year, influenced by precipitation patterns and land use practices. Higher flows typically occur during spring runoff, while lower flows are common in the summer and fall. The presence of agricultural drains and urban stormwater systems further affects the flow regime, leading to rapid runoff and reduced groundwater recharge. Hydrometric data, obtained from Environment and Climate Change Canada's stream gauge at BearBrook near Bourget, provides valuable insights into the daily water levels and discharge, essential for understanding the creek's hydraulic behavior.

The terrain within the study area is relatively flat, with an elevation change of about 80 meters over its 180-kilometer length. This gentle slope contributes to the creek's susceptibility to both flooding and drought conditions. High-resolution terrain data, derived from the Ontario Digital Terrain Model (Lidar-Derived), provides a detailed representation of the bare-earth terrain, which is critical for developing accurate hydraulic models and assessing erosion processes, (GeoHub,2024).

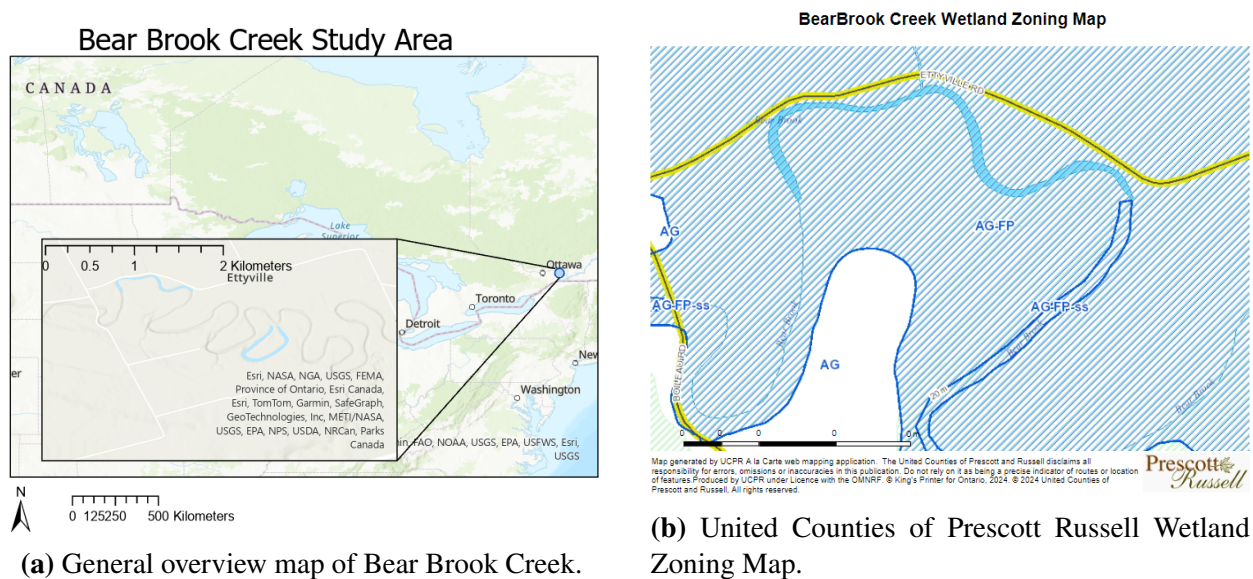


Fig. 1. Maps showing the study area of Bear Brook Creek 45.426764°N, -75.112408°W.

Soil characteristics in the Bear Brook Creek area are a mix of silty clay loam and cohesionless silty sand, particularly along the creek banks. These fine-grained soils have low to moderate hydraulic conductivity, influencing water retention and drainage capabilities. The soil composition also includes moderate to high levels of organic matter, which enhances soil structure and moisture retention, supporting plant growth and soil stability. The presence of fine-grained, cohesionless soils along the creek banks increases the erosion potential, especially during high flow conditions, necessitating effective erosion control measures.

Both hydrologic and soil conditions are interdependent in understanding channel behavior. Key terrain defining features of the meander channel, such as bank depth, soil erodibility, environmental loads, maximum discharge, and flow rates within the channel, are critical for floodplain management and habitat preservation. The bankfull width, depth, and discharge rates provide insights into the creek's ability to transport sediment and manage flow energy during peak discharge events. Understanding these parameters is key to define effective erosion control measures while mitigating the creek's impending geo-hazards, where relevant hydrologic properties are given in table 1.

TABLE 1. Hydraulic Characteristics for BearBrook Creek from the 25-Year Bearbrook Stream Gauge Data from Environment & Climate Change Canada, (2024) .

Description	Value	Unit
Catchment area	4,200	km ²
Length	180	km
Average depth	3	m
Average width	12	m
Slope	0.00044	m/m
Minimum discharge	0.071	m ³ /s
Maximum discharge	128.0	m ³ /s
Dominant discharge	2.974	m ³ /s
Avrage discharge	5.1	m ³ /s
Average discharge during flooding events	28.795	m ³ /s
Return Period for Bankfull Events	1 in 425	days

CASE STUDY

The BearBrook case study involves applying geotechnical, hydraulic and sediment transport theories to characterise and model the erosion along BearBrook creek to produce feasible mitigation measures. The model involves coupling the 1D IRIC JP Meander model with the 2D IRIC FaSTMECH river flow model to determine the stress and flow profile along the eroding river segment, tangent to Ettyville Road. The JP Meander model takes advantage of meander migration theory to identify areas where the sinuosity of the river encroaches into the easement and municipal roadway. Identified meander migration zones are coupled and validated in FaSTMECH using shear stress profiles and erosion potential, taking advantage of mesh interpolation using the depth averaged RANS formulation. In order to first quantify the physical behaviour of the channel, the governing hydrologic behaviour is established to provide a basis and input to the case study.

Hydrology of BearBrook Creek

The 4,200 square kilometer drainage basin comprises of a network of rivers, channels, drainage ditches and municipal drains within the rural setting. From the Environment and Climate Change Canada Bearbrook station, (2024), the stage and water elevation data is shown as an example to showcase the flow regime in 2023, as noted below in Figure 2.

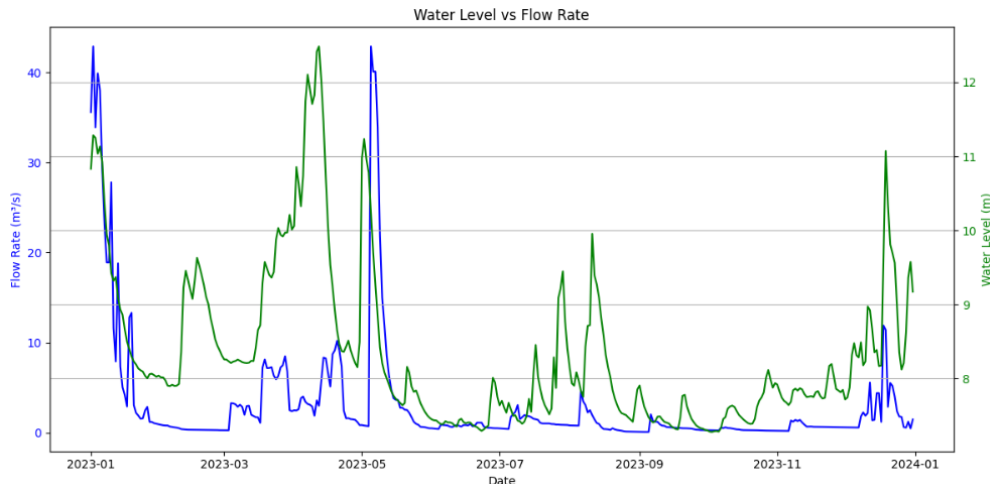


Fig. 2. Water Level vs flow rates in 2023 using Environment and Climate Change Canada data from the BearBrook Creek Station, (2024)

Using the 25 year historical data from the stream gauge, the yearly runoff hydrograph showcases the seasonality of the flow regime. Per the Köppen–Geiger climate classification Clarence–Rockland contains humid continental climactic conditions, classified as (dfb) within the classification system. This entails mild summers, long winters with frequent periods of very cold temperatures and clear skies. Immense weather variability on yearly basis and large annual temperature ranges show comparable conditions as (dfc) climates, and is represented through variabilities in the runoff hydrograph.

Due to the cold region conditions, the 25 year runoff hydrograph shows signs of yearly spring freshets between the months of march and may, with an anomalous event in 1998, conventionally known as the ice storm of 1998. Further variations in spring runoff are extended as far as July and as early as January as a result of the extreme variations in temperature regime, which can be denoted by the hydrograph in Figure 3.

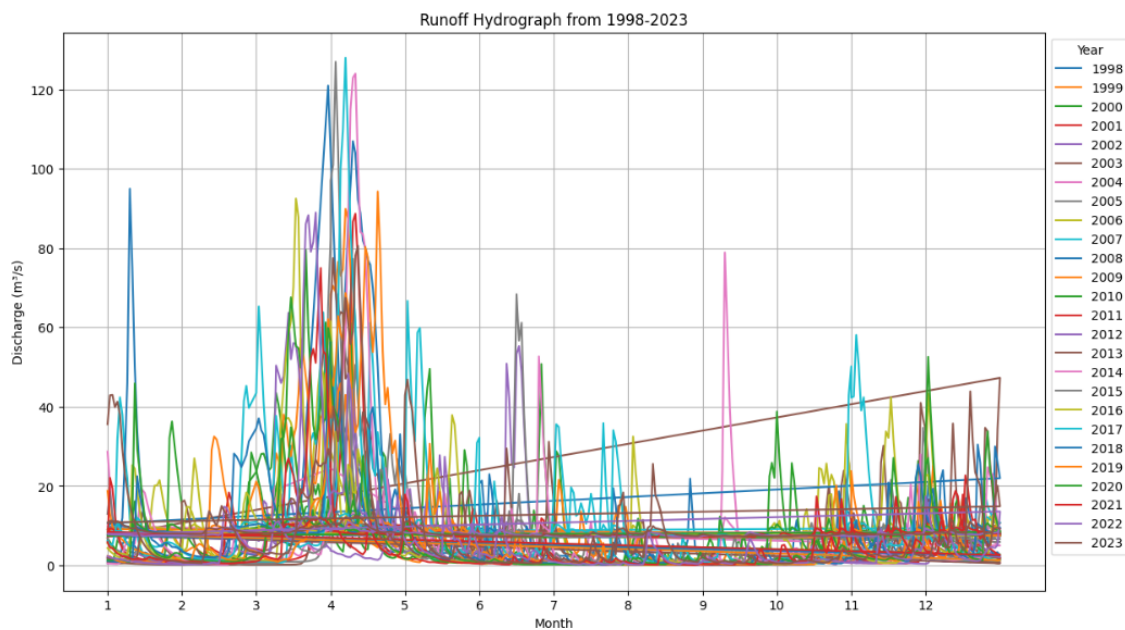


Fig. 3. 25 Year Runoff Hydrographs using Environment and Climate Change Canada data from the BearBrook Creek Station's Stream Gauge, (2024)

Quantifying these extreme variations is key to determining the excess shear stress induced flow events, and can be characterised by the return period of extreme spring freshet flow events, as shown below in Figure 4. As flow rates respective to stage are directly proportional to the applied shear

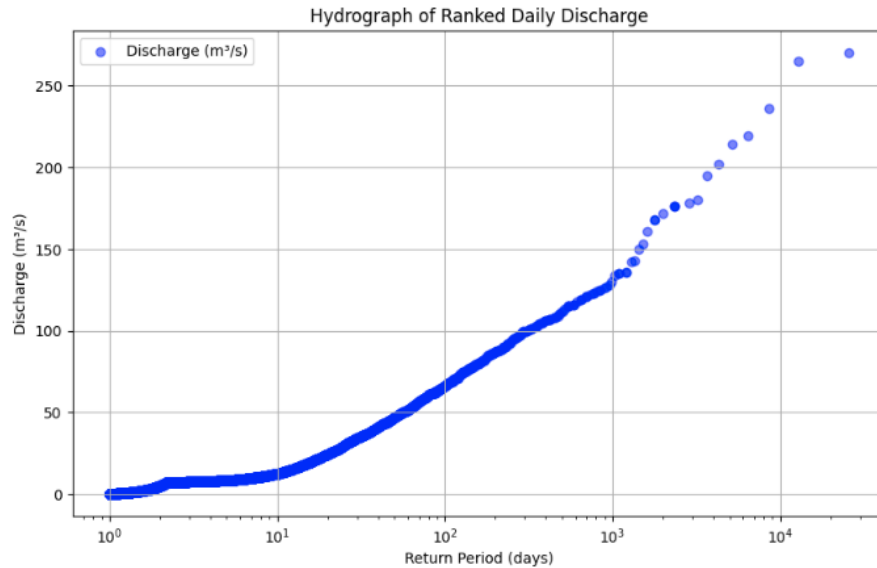


Fig. 4. Discharge Return Period using Environment and Climate Change Canada data from the BearBrook Creek Station's Stream Gauge, (2024)

shear stress of currents, the corresponding flow rates which cause an excess shear greater than the critical shear stress of a soil material denotes the erodibility at a location within the river bank.

Channel Characteristics of Bearbrook Creek

The meander zone of Bearbrook Creek comprises of multiple cross sections spanning from 10m to 15m in width, with a mean channel width of 12m. Most channel cross-sections comprise of a compound channel with an over bank section. The main channel is typically 10m-12m in width with a secondary compound width of up to 15m. The overbank section for the 5 cross-sections range between 5 and 10 meters in width from the toe of the primary channel section however, the top of the widest eroding bank section is but 1m from an existing roadway, as shown in Figure 6, 7 and Figure 8, illustrating the over-bank cross section in the problematic areas.

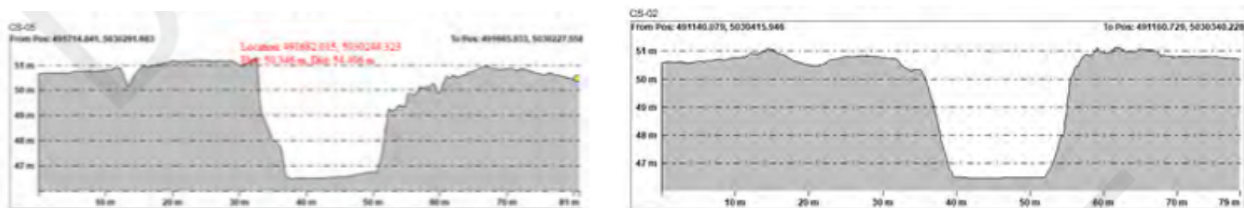


Fig. 5. Cross Slope Sections 5 and 2 (Stantec Consulting Ltd., 2023)

The river itself has a length of 180 km, with a drop in elevation of 80m, presenting an average slope of 0.000444 m/m, and locally within the section, an average bed slope of 0.0004m is present (South Nation Conservation Authority, 2014). The water level recorded is roughly 3m in height although bank full flows occur during spring freshets and low flow events reduce the stage substantially during droughts.

Given the channel geometry, various bank cut sections exhibit various soil conditions. The conditions vary from stiff brown or brown clay to sandy silt conditions. The soil layers comprise of a lower clay layer with an upper silty sand or clayey silt overburden layer. For this reason, the erosion rate at location 1 and location 4 differ, where their locations are shown below in Figure 9.



Fig. 6. Geotechnical Exploration Terrain Map (Stantec Consulting Ltd., 2023)

The most sensitive and erodible layer is located cross section 1, comprising of an upper layer of clay stiff grey clay. The second, prone area is located at cross-section 5, comprising of silty sand with some clay. Given these are fine grained materials, Figures 10 and 11 below show the grain size distribution for 2 boreholes along the riverbanks in these locations.

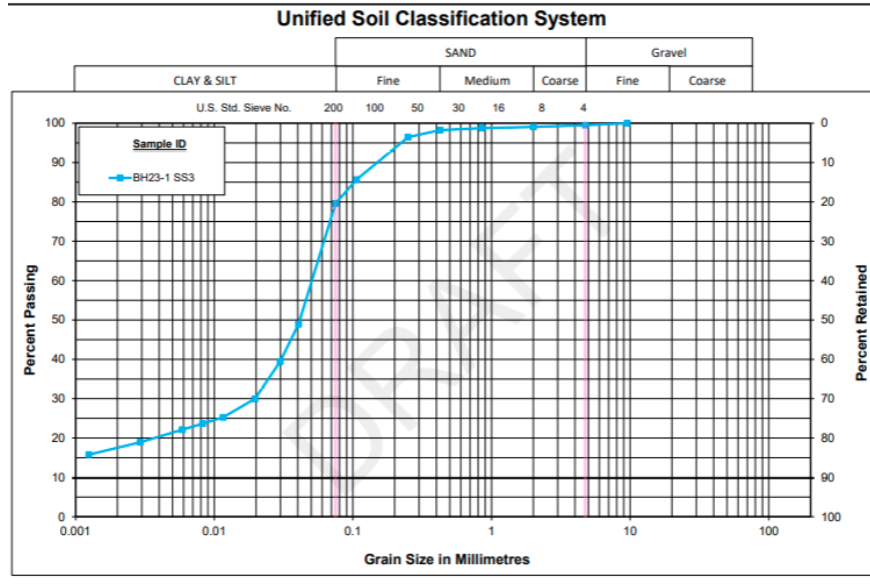


Fig. 7. Clayey Sand (CL) Grain Size Distribution (Stantec Consulting Ltd., 2023)

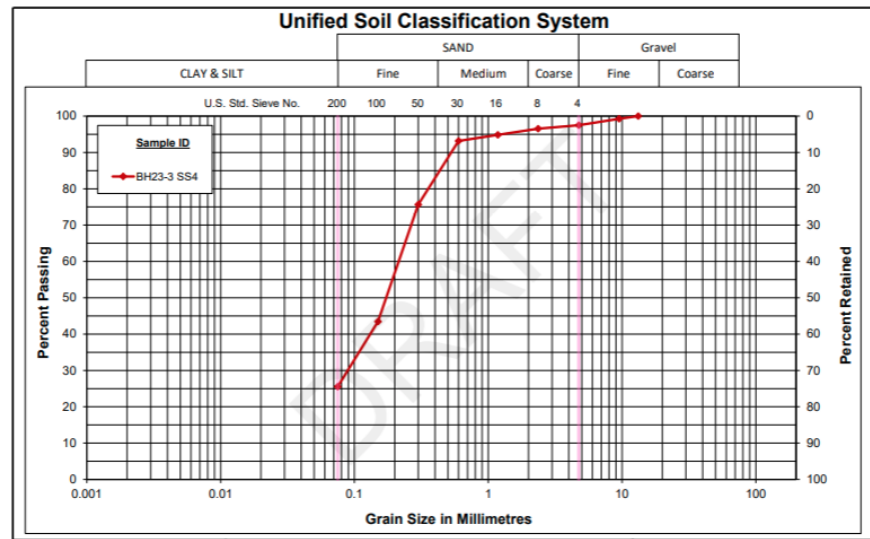


Fig. 8. Silty Sand (SM) Grain Size Distribution (Stantec Consulting Ltd., 2023)

The primary soil characteristics of the channel show that for the clayey sand section, it has median grain size of 0.03mm, a SC content of approximately 82 %, and a PC content of 25%. For the silty sand material, the median grain size is 0.167 mm, an SC content of 25%, and a PC content of 0%. The two soil types present within the channel show both well graded and poorly graded conditions which affect the erosion and stability of the slope. Since these are fine grained materials

for an alluvial bedform, acceptable Manning's n values of 0.02 to 0.04 are used (Chow, 1959).

Given the two grain sizes, employing the Darcy-Weisbach friction coefficient with the Manning's equation, the resulting drag coefficient (C_d) falls within the range of 0.016 for fine grained Clay's given $n=0.02$ conditions, to 0.065 for silty sands with a Manning's n of 0.04. Since each location contains both silty sands with clay content, to be conservative a (C_d) value of 0.065 is used in the following analysis.

1D JP Meander Modelling

Given the channel characteristics and the overview of the applicable theory to meander migration, IRIC JP Meander presents a useful tool for modelling migration patterns based on limited data. Using geometry by importing channel centre lines from Google earth, the JP meander solver simulates meander migration based on a model fitted FD value in this case being 300, a roughness scaling value (C_f) of 2, denoting the maximum mannings coefficient of 0.04 compared to the minimum value of 0.02, an overburden median grain size of 0.167 mm, and the previously noted channel geometry. The friction scaling is defined by:

$$C_f = \frac{\tau}{\rho U^2} = \frac{pgHS}{\rho U^2} \quad (1)$$

where τ is the bed shear stress, ρ is the water density, g is gravitational acceleration, H is the average depth, S is the average downstream slope, U is the vertically averaged velocity. The calibration of the model is based on recent, but historical LIDAR imaging dating to 1985, and the meander migration FD value is modified to fit the migration pattern. To simplify computation, the model utilizes friction scaling along a vertically averaged velocity, computed from the bed to the surface flow conditions. These conditions rely on the Strickler coefficient (k_s) as it relates to the roughness length of a material, a property related to the previously noted Mannings equation, shown below in equation (32). To integrate (k_s) and n to compute the average velocity of the meandering channel, the averaged velocity U is computed using the integral form of the logarithmic law of the wall:

$$U = \int_{bed}^{surface} \frac{\sqrt{gHS}}{\kappa} \ln \left(\frac{z}{k_s/30} \right) dz \quad (2)$$

where z is the elevation from the bed, κ is von Karman's constant, k_s is the roughness length. By relating the bed shear stress to both mannings n and the scaling coefficient, U is reduced to:

$$U = \left(\frac{gHS}{C_f} \right)^{1/2} \quad (3)$$

Given that U is dependent on channel curvature and meander length along a curvature the relation can be differentiated with respect to the change along channel distance ds , we get the relation below;

$$\frac{d\psi_0}{ds} = \frac{d}{ds} \left(\frac{b_i}{\bar{r}} \right) = b_i \frac{d}{ds} \left(\frac{1}{\bar{r}} \right) = -b_i \frac{1}{\bar{r}^2} \frac{d\bar{r}}{ds} \quad (4)$$

Since $\frac{d\bar{r}}{ds}$ depends on the erosion rate and flow induced applied shear stress, the erodibility equation is related as shown below:

$$\frac{d\bar{r}}{ds} = k_d(\tau_a - \tau_c)^a \quad (5)$$

Where the meander migration can be expressed as:

$$M = \epsilon \frac{\partial \theta}{\partial s} \quad (6)$$

Thus, the change in maximum centerline curvature is:

$$\frac{d\psi_0}{ds} = -b_i \frac{1}{\bar{r}^2} k_d(\tau_a - \tau_c)^a \quad (7)$$

Where in the case of center line migration, the critical shear stress is evaluated based on the bed shear stress as noted by the Shields equation. By combining Equation (5) and (6), the curvature dependent migration can be reduced to:

$$M = k_d(\tau_a - \tau_c)^a \frac{2\pi}{\lambda_m} \sin\left(\frac{2\pi}{\lambda_m}s\right) \quad (8)$$

Considering the governing equations as they relate to equation (63), the JP Meander solver and grid generation tool is employed for varying channel geometries, changes in wavelengths, and additional conditions to simulate the meander migration patterns (Johannesson and Parker, 1988). In the model employed for this study, the first condition within the scenario is to set FD to the optimized values in order to simulate natural stream response without any modifications. The second condition is to apply a boundary condition, such as the slope armouring scenario in order to evaluate the constrained meander migration behaviour. This condition is important as it relates to the operations and maintenance of the site. The site regularly undergoes rip rap protection on an ad-hoc failure based cycle. Because of this, the meander segments are exposed to both conditions. Showing both boundary condition scenarios allows the analysis to start the meander migration model from the state where the slope's armouring has finally degraded, in which future conditions can be shown.

Figure 12 below showcases the natural meander migration of the channel under a $P_{50\%}$, 5.1 m³/s discharge for the 25-year historical flows, illustrating incremental changes in channel sinuosity while also showcasing the critical meander year in which the half width b of the channel intersects the tangential road shoulder.

When applying the steady state mean flooding flow of 28.8 m³/s without any erodibility areas, the meander pattern in Figure 13 emerges, showcasing the future potential hazardous areas due to meander migration by 2050.

Comparing the meander lengths to the natural meander pattern based on the average flow rate in Figure 12, the average flooding meander length for that 5 year increments in between 2025 and 2030 is 6.67m at cross-section 2, and 6.34 m at cross-section 5. Utilizing the same conditions in the case where the bank is assumed to be protected against meander migration, the added erodibility boundary conditions limit the extent of the meander migration to the toe of the slope as shown in Figure 14.

IRIC JP Meander Migration Model for Bear Brook Creek

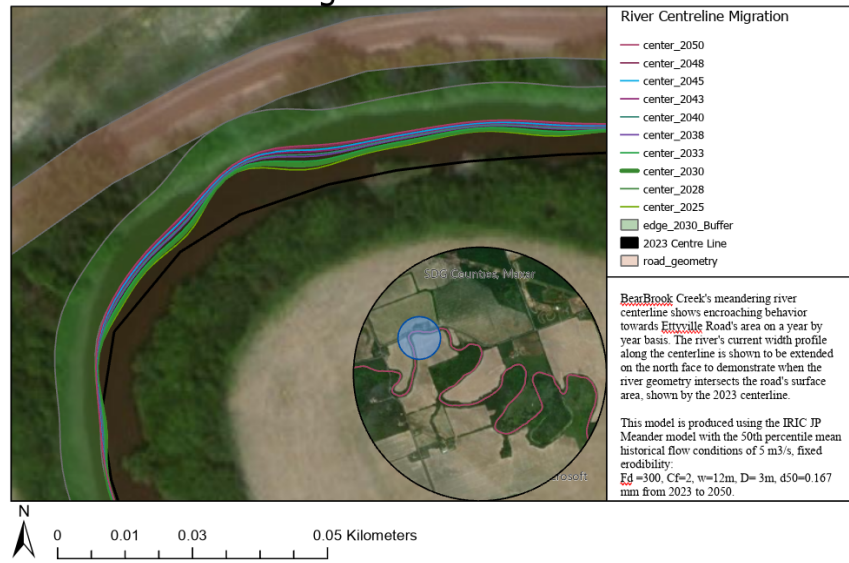


Fig. 9. Natural Meander pattern for Bearbrook Creek

IRIC JP Meander Migration Model for Bear Brook Creek

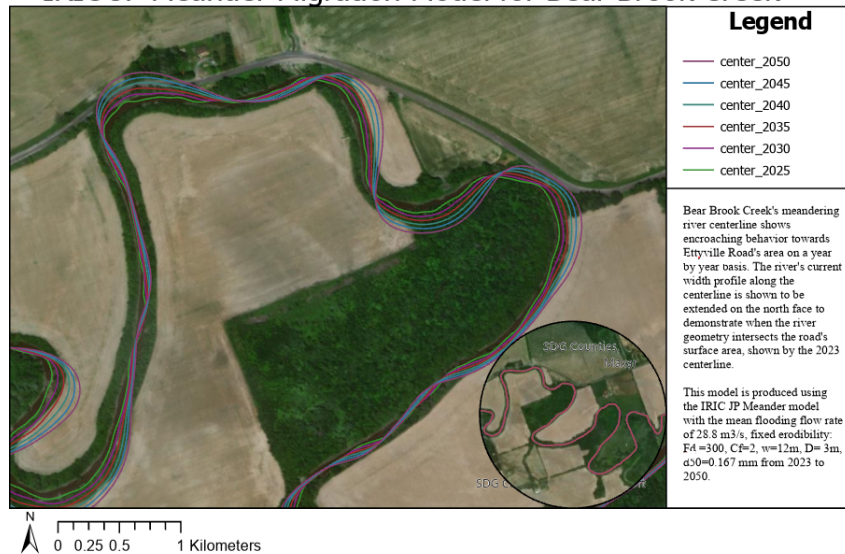


Fig. 10. Average flooding flow rate 28.8 m³/s Meander pattern for Bearbrook Creek

Applying slope armouring in the same case as the average flooding flow rate shows an estimated reduction in meandering, to 1.5m at cross-section 2 and 1.6m at cross-section 5, as the slope is considered non erodible.

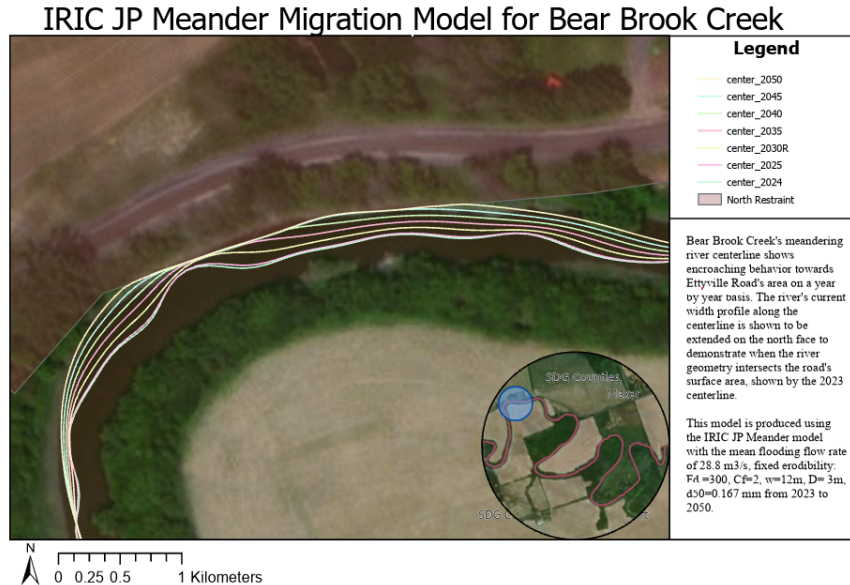


Fig. 11. Average flooding flow rate 28.8 m³/s Meander pattern for Bearbrook Creek

2D FaSTMECH Modelling

Using the FaSTMECH for hydraulic modelling, measured inputs such as a 12m channel width and an average flow depth h of 3 meters, Manning's roughness coefficients n were set at 0.02 and 0.04, and the channel slope was determined to be 0.000444 m/m. Drag coefficient values were considered as 0.016 and 0.064. Given the LIDAR resolution of 0.5 meters, a grid mesh size of 0.5m x 0.5m was generated, with shape functions (N_i and N_j) having lengths of 0.5 meters. The evaluated stream length was 2000 meters, resulting in a grid with 4000 element lengths and a width of 24 elements. The upstream stage was set at 50 meters in elevation, with a constant stage analysis, and river stages ranged from 50 meters to 46.6 meters. The FaSTMECH model results showed consistency with the IRIC JP Meander model regarding meander migration patterns. Cross-sectional profiles, at the problematic areas cross sections 1 and 5, showed scouring at the outer bends, which corresponded to the maximum amplitude of the bends. These areas also showed increased bed elevations, likely due to sediment deposition from eroding banks. The calculated Froude number of 2.85 indicated a weak hydraulic jump across these sections. In deeper channel areas near the banks, while maintaining a constant stage, the flow developed into a supercritical state from the inner bends toward the outer bends. The model, assuming a constant flow rate with a

variable bed profile, indicated that outer bends experience lower shear stress than the center of the stream around bends on average, but with greater shear stress distribution along the water surface due to a reduction in the Froude number and corresponding velocity drop.

The FaSTMECH model, utilizing the RANS equations for channel curvature, was implemented without sediment transport or eddy analysis (Nelson). Figures 12 (a),(b),(c),(d), show the development of velocity and stress along the streambank. Three problematic areas identified by both the meander migration model and numerically predicted shear stresses were consistent with field observations, showing higher shear stresses and velocity gradients compared to the rest of the channel, indicative of potential erosion sites. Its important to note that in this study, the middle bend is not considered as it is currently a municipal drain outlet.



(a) Median flow rate Velocity Distribution.



(b) Median Flow rate Shear Stress Distribution



(c) Critical Shear Stress Velocity Distribution



(d) Critical Shear Stress Distribution

Fig. 12. IRIC FaSTMECH Constant Flow, Mesh Derived Velocity and Stress Distribution Model.

As shown above, these meander zones show an uneven stress distribution along the cross-section, showing lower shear stresses at the bank compared to the center line flow of the channel. In previous studies such as Wynn et al. 2003, and Islam and Matin, 2022, center line flows are used in the excess shear formulation to estimate erosion from the respective hydraulic models. Using the water surface profiles, table 2 and table 3 below show how the near bank stresses compare to the center line stresses at cross sections 2 and 5. Since CS-02 is approximately 12m wide and

Deviation of Near Bank Shear Stresses (Pa) from Centerline Stresses at CS-02					
$\tau_{5.1m^3/s}$	%deviation	$\tau_{28.8m^3/s}$	%deviation	$\tau_{24.8m^3/s}$	%deviation
0.0784	67%	2.89	64%	2.1	65%
0.179	26%	5.95	25%	4.4	26%
0.182	24%	5.93	25%	4.39	26%
0.241	0%	7.73	0%	5.73	0%
0.119	51%	3.82	52%	2.83	52%
0.167	31%	5.43	32%	4.023	32%
0.175	27%	5.59	30%	4.14	30%
0.125	48%	3.99	50%	2.95	50%

TABLE 2. Shear Stress and Percentage Deviation for Different Flow Rates at Cross-Section 2. Calculated shear stress values for each channel geometry are 0.241 Pa, 7.93 Pa and 5.93 Pa respectively.

has boundary condition values of 0, the results produces 8 flow values spaced 1m apart across the channel, while CS-05 is 13m wide and under the same mesh boundary conditions, produces 9 values of shear stress across the section, shown in table 3.

Deviation of Near Bank Shear Stresses(Pa) from Centerline Stresses at CS-05					
$\tau_{5.1m^3/s}$	%deviation	$\tau_{28.8m^3/s}$	%deviation	$\tau_{24.8m^3/s}$	%deviation
0.0324	61%	1.16	56%	0.853	56%
0.0522	37%	1.69	36%	1.25	36%
0.0593	28%	1.89	4%	1.41	28%
0.0788	4%	2.53	0%	1.88	4%
0.0824	0%	2.63	0%	1.95	0%
0.0828	0%	2.64	0%	1.96	1%
0.0826	0%	2.64	0%	1.95	0%
0.0802	3%	2.56	0%	1.89	0%
0.0569	31%	1.81	31%	1.35	31%

TABLE 3. Shear Stress and Percentage Deviation for Different Flow Rates

Applying the results from the meander and stress models using IRIC JP Meander and Fastmech, the bank erosion behaviour can be further numerically modelled using empirical techniques to physically determine the extent of the recession.

State of the Art of Riverine Erosion

Starting with understanding the concept of erosion, erodibility defined as a key dependent metric which quantifies the inherent rate of resistance of soil materials. A useful approach to determining erosion is coupling the concept erodibility with applied environmental forces and intrinsic resistive stresses, one method of which involves the use of the excess shear stress equation. This approach is fundamental in predicting the erosion rate of fine-grained soils in riverbanks and channels given by the following equation:

$$\epsilon = k_d(\tau_a - \tau_c)^a \quad (9)$$

Originally derived from Partheniades (1965), Hanson and Cook (1997), and others, they estimate that erosion occurs when the applied shear stress τ_a exceeds the critical shear stress τ_c of a given soil material considers the concept of relating driving forces to resistive forces considering a materials mechanical properties. The erodibility coefficient k_d quantifies the soil's susceptibility to erosion, while the exponent a is empirically determined and assumed to be 1. To empirically quantify the excess stresses which drive the erosion rate, the force balance is broken down into components such as the soils ability to resist shear stresses, and applied environmental loads acting as applied shear stresses. Critical shear stress can be estimated using empirical relationships that relate soil properties to τ_c through various past investigations such as: **Smerdon and Beasley (1961):**

$$\tau_c = 3.54 \times 10^{-2} D_{50}^{1.28} \quad (10)$$

$$\tau_c = 0.493 \times 10^{-0.0182P_c} \quad (11)$$

These equations relate τ_c with the median particle size (D_{50}) and percent clay content (P_c) of the

soil, respectively. **Neil (1973):**

$$\tau_c = 0.76090\gamma(S_G - 1)D_{50}^{2/3}d^{1/3} \quad (12)$$

Where specific gravity of soil (S_G) and specific weight of water γ , median particle size (D_{50}), and flow depth d to empirically quantify soil parameters. Due to the hydraulic implications in this approach, it cannot exclusively be considered a property of soil but presents a multi parametric approach to modelling shear stresses. Further research empirically approaches shear stress behaviour by material type, and in the cases below as a function of grain size, and silt clay content. **Swamee and Mittal (1976):**

$$\tau_c = 0.155 + \frac{0.409d^2}{\sqrt{1 + 0.177d^2}} \quad (13)$$

Julian and Torres (2006):

$$\tau_c = 0.1 + 0.1779(SC) + 0.0028(SC)^2 - 2.34 \times 10^{-5}(SC)^3 \quad (14)$$

This polynomial equation estimates τ_c based on the proportion of silt and clay (SC) in the soil, showing a direct relationship between soil composition and critical shear stress. As the Shields' diagram is traditionally used to estimate τ_c for noncohesive soils based on particle size and specific gravity, this empirical relationship becomes useful when strictly evaluating the % fines in the grain size distribution of various alluvial river bed materials. Due to the misalignment of parameters across empirical methods, the accuracy and applicability are affected when applied on conditions which do not fully conform to the bounds of the prescribed equations, and in the case of Julian and Torres, leading to the underestimation of τ_c for fine-grained sediments.

The submerged jet test device, developed by Hanson (1990), provides a practical method for determining τ_c and k_d in situ. This device measures the depth of scour caused by a jet of water impinging on the soil surface, and the erosion parameters are estimated by fitting the observed data to the excess shear stress equation.

Osman and Thorne (1988) presented a method for estimating the lateral bank erosion rate based on τ_c and the erodibility coefficient k_d :

$$k_d = 2.5\tau_c^{-0.44} \quad (15)$$

Hanson and Simon (2001) expanded on this work by conducting jet tests to determine τ_c and k_d for highly erodible loess in the midwestern U.S. They found an inverse relationship between τ_c and k_d , which they expressed as:

$$k_d = 0.2\tau_c^{-0.5} \quad (16)$$

On the other hand, k_d may be empirically predicted using known values of τ_c . Based on submerged JET findings, such empirical relationships between τ_c and k_d have been constructed, and they are conventionally modelled using inverse power law relations Hanson & Simon, (2001); Thoman & Niezgoda, (2008); Wynn et al., (2004). Erodibility co-efficient, k_d was computed by the formulae based on the researches performed by Wynn et al. (2004), Simon et al. (2010) and Karmaker and Dutta (2011). Different equations developed for k_d which have been used in this study are given below:

$$k_d = 3.1\tau_c^{-0.37} \quad (\text{Wynn et al. 2004}) \quad (17)$$

$$k_d = 1.62\tau_c^{-0.838} \quad (\text{Simon et al. 2010}) \quad (18)$$

$$k_d = 3.16\tau_c^{-0.185} \quad (\text{Karmaker and Dutta 2011}) \quad (19)$$

In the study conducted on the Jamuna River in Bangladesh, a comprehensive approach was used to estimate the critical shear stress and soil erodibility. The empirical models developed by Osman and Thorne (1988) and Hanson and Simon (2001) were applied and validated against local conditions using an inverse power law averaging method, yielding the equation presented by Islam et al., 2022:

$$k_d = 2.5\tau_c^{-0.464} \quad (20)$$

The principle of excess shear stress (τ_a) traditionally considers only the average hydraulic stress.

$$\tau_a = \rho g d S \quad (21)$$

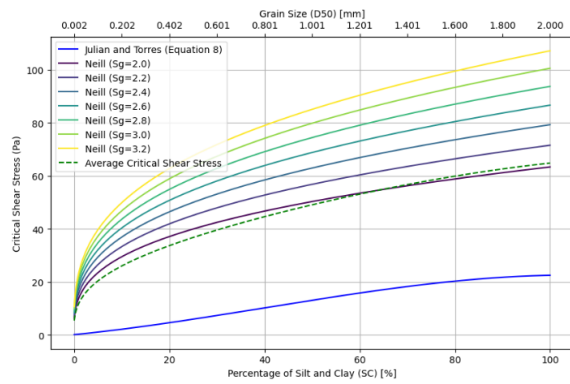
where τ_a is the average applied shear stress at the section midpoint (Pa), ρ is the density of water (kg/m^3), g is the acceleration due to gravity (m/s^2), d is the height of water above the midpoint (m), and S is the slope of the energy grade line (approximated by channel slope) (Clark and Wynn, 2007).

However, a more accurate depiction of applied shear stresses can be determined using channel hydraulic theory by applying the results from the meander and stress models using IRIC JP Meander and Fastmech to determine the extent of the recession.

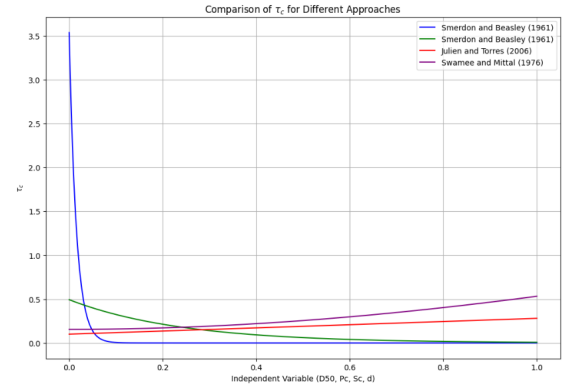
Bank Erosion

Using the theory of excess shear stress within the concept of bank erosion, equations (10), (11), (12), (13) and (14) all govern the minimum and maximum τ_c behaviour of given soils, derived from empirical tests. All empirical formulae of τ_c were derived by investigating in situ and laboratory jet test results for a given set of parameters. The limits of each empirical relation are all bound by specific sites, native soil shear strengths, and the range of pressures tested with the jet plate test. Depending on soil type, operational limits of the jet plate test apparatus, and field or laboratory conditions, not all tests have pressure ranges starting from 0. Since much of the work subsequent to both Hansen and Simon, (2001), and Osman and Thorne, (1988), have used a lower limit of 0.6 Pa, the lower limit of fitting the empirical relations must not exceed the lower limit to avoid error and maintain the empirical validity. Furthermore, some empirical relations focus on only specific soil types, in turn reducing an empirical range close to the upper limit of subsequent tests. For example, Julian and Torres, (2006) focuses on a jet test stress range of 3.50 Pa to 3.53 Pa, while Clark and Wynn, (2007), performed 142 multi angle in situ jet tests at 25 different sites with stress ranges of 0.6 Pa to 30 Pa. Using the τ_c equations from each derivation thus requires careful attention to empirical limits.

The study published by Islam and Matin, (2022), employed an inverse power curve fitting technique to determine the average shear stress of the excess shear stress equation (1), however the upper bound employed to calculate k_d using the results of all empirical formulations for τ_c was given by using the maximum computed τ_c . Using the governing equation, it is applied to equations (16), (17), (18) and (19) to yield equation (20). Fundamentally, equation (20) provides an average shear stress relationship to characterise bank erosion however, while employing the exponential function $k = a \cdot b^{-k}$, the weight of the function a was fixed using a value of 2.5. The value was chosen in their study because prior literature had different values for various types of soils and in this case, it is because the material is silty sand like in Osman and Thorne (1988). However, Osman and Thorne was also a baseline for Wynn et al., (2004), and as a result of the varying soil types from the study, using 2.5 for one soil type skews the inverse power relationship as stresses increase. By plotting the values of τ_c below in Figure 16 below, the varying developments of stress are presented.



(a) Water Level vs flow rates in 2023 using Environment and Climate Change Canada data.



(b) Bedform average critical shear stress from given data from site investigation data.

Fig. 13. Soil Critical Shear Stress Comparisson of Empirical Equations (10), (11), (12), (13), and(14).

Further more, as various formulae utilize one or two soil parameters, their relationships are only applicable if the parametric data is available, or comparable. Due to this constraint, employing Neil et al., (1973) allows for the specific gravity of the material with respect to grain size and flow

depth to be evaluated considering the gravitational forces of water, determining a more accurate representation of the materials critical shear stress, although at the bed or the toe of a bank due to the depth parameter d . By evaluating specific gravity of the soil in BearBrook Creek having a S.G of 1.84 for brown clay, 1.94 for stiff grey clay and 2.0 for silty sand, while considering the other parameters, the limit of critical shear stress for the excess shear formulation is 5.93 Pa for the upper silty layer and 20.6 Pa for the stiff grey clay layer. Using this shear stress limit, Figure 17 below shows the inverse power average fitting of each empirical relation for edibility.

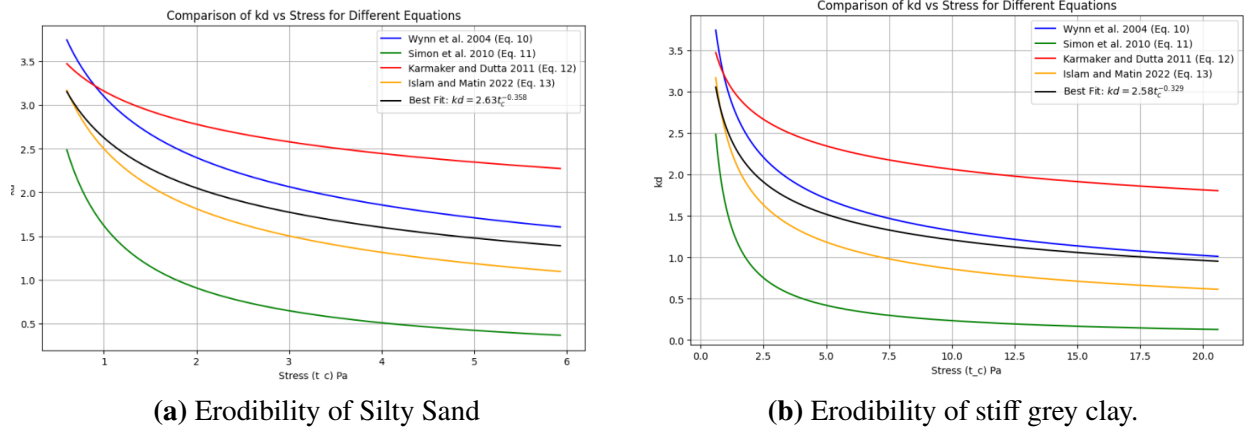


Fig. 14. Comparison of empirical k_d formulae from Wynn et al., (2024) ; Simon et al., (2010) ; Karmaker & Dutta, (2011) ; Islam and Matin, (2022), given a shear stress of 5.93 Pa and 20.6 Pa.

The comparison in Figure 13 shows a best fit fit with $a=-2.63$, $b = \tau_c$, $k = 0.353$. Notably, the site conditions show silty sand, stiff brown and grey clay. Since this relationship is not built on fixing $a=2.5$ based on the soil type, the a and k values are optimized to best fit the average. Furthermore, as Wynn et al., applies a pressure range up to 30 Pa, and when the chart is extended beyond the critical shear stress limit of this study to Wynn's et al., 2001 upper limit, the best fit begins to converge with Wynn's formulation, showing a stronger correlation for R^2 . The convergence suggests the average shear stresses of all methodologies with respect to the empirical erodibility formulae are consistent with Wynn et al., results simply due to the number and range of tests performed by their study. However, since this site has an upper limit for a critical shear stress limit of 5.93 Pa, this limit is employed to fit the site conditions for Bearbrook creek, resulting in Bearbrook Creeks characteristic

erodibility equation below:

$$k_d = 2.63\tau_c^{-0.353} \quad (22a)$$

$$k_d = 2.58\tau_c^{-0.329} \quad (22b)$$

As the methodology in this study is modelled after the results of Islam and Matin, (2022), the same R^2 coefficient of 0.79 is obtained, showing a strong correlation between the empirical relation while validating the method by their study. It also reinforces the variability between the employed parameters induce only 21% error rate. It is important to note that because the best fit converges with Wynn et al. while the other erodibility relations converge to their limit, the R^2 increases to 0.81 as the range of critical shear stress increases. However, as the range decreases below 0.60, the relationship breaks down due to the minimum stress test ranges performed by the in situ jet test. Considering equation (64) for both critical soil materials, using the fitted equation, the silty sand using the above erodibility rating shows signs of very erodible to erodible soil, while stiff grey clay presents itself as a moderately resistant soil material as shown below in Figure 15.

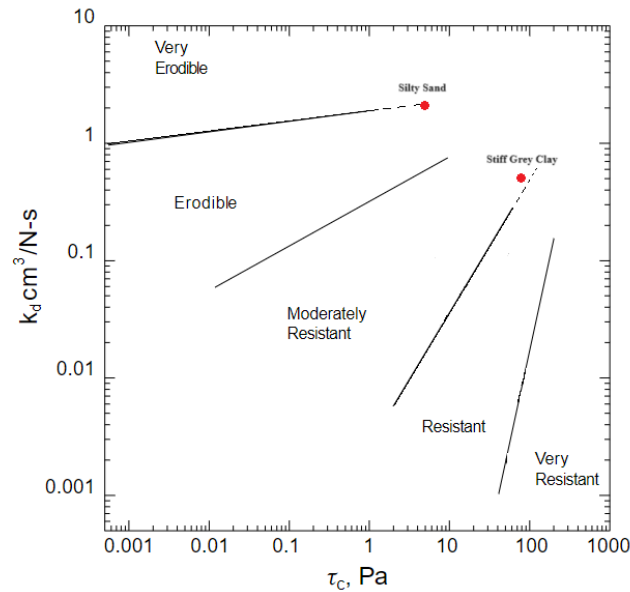


Fig. 15. Erodiblity Monograph For Silty and stiff Grey Clay (Hanson and Simon, 2001)

While erodibility can be determined given flow induced shear stress, the mean flooding flow erodibility are shown in table 4 below.

Silty Sand:	$D_{50} =$	0.167 mm,	$P_c = 0$	$S_c = 25$		
Equation	τ_c (Pa)	$k_d(9)$	$k_d(12)$	$k_d(10)$	$k_d(11)$	$k_d(\text{This study})$
(2)	3.50196	1.9497	1.39759	0.566739	2.50605	1.67771
(3)	0.493	4.02725	3.47144	2.93028	3.60172	3.38288
(5)	5.93187	1.60424	1.09441	0.364401	2.27325	1.38946
(6)	0.166379	6.91943	5.7458	7.28174	4.40336	4.98921
Grey Clay:	$D_{50} =$	0.03 mm	$P_c = 25$	$S_c = 82$		
(2)	3.53931	1.94261	1.39073	0.561721	2.50113	1.67136
(3)	0.172921	5.93415	5.6439	7.05017	4.37225	4.92885
(5)	20.613	1.01188	0.614041	0.183311	1.80539	0.889917
(6)	0.155004	6.17924	5.93774	7.72696	4.46143	5.1172

TABLE 4. Soil edibility results for the fitted relationship between 0.6 and 5.93 Pa, and 0.6 Pa and 20.6 Pa

To accurately determine the bank erosion given the daily flow data, by applying equation (1) and summing the daily erosion with their respective rates due to excess shear, we get the yearly bank erosion for each material shown below in Table 5 where the average yearly erosion for silty sand is 1.03 m and for Brown clay it is 0.486 m.

Year	Silty Sand Erosion (m)	Stiff Grey Clay Erosion (m)
2013	0.908670	0.428625
2014	0.750640	0.354081
2015	0.434581	0.204994
2016	0.869162	0.409989
2017	2.172906	1.024972
2018	1.066699	0.503168
2019	1.185222	0.559076
2020	1.343251	0.633619
2021	0.474089	0.223630
2022	0.632118	0.298174
2023	1.501281	0.708162

TABLE 5. Erosion results for Silty Sand and Stiff Grey Clay from 2013 to 2023

Considering the above results compared to the recorded landslides in the inner bend of cross-section 3 and the outer bend of cross-section 5, the results are within the range of the estimated yearly excess shear erosion. The figure below depicts two recorded landslides along the noted banks. As cross-section 5 contains less silt than clay material, this is indicative of possible silty

sand erosion from the landslide as the erosion is approximately 4m, shown in Figure 19. The landslide in cross-section 2 shows an increase in channel depth, possibly why the erosion is lower than estimated.

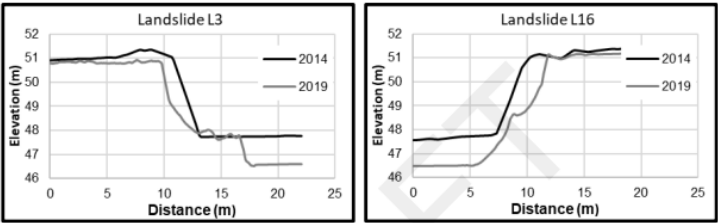


Figure 5-2: Examples of topographic profiles along landslide features, and their evolution between 2014 and 2019.

Fig. 16. Cross-Section 2 and 5 lateral landslides (Stantec Consulting Ltd., 2023)

Given the estimated bank erosion results using the excess shear stress equation it is important to note that recession from bank erosion does not account for the 3D slope instability, and the distribution of stresses along the face with respect to depth of the bank at various stages, resulting an estimated erosion length in table 5. Furthermore, as erodibility is a volumetric empirical estimation of the rate of decay along a bank section and does not account for different types of failure. However, using the coupled quasi-3D hydraulic bank erosion model, Figure 20 represents the estimated erosion after the last site measurement in 2022 to 2023 given updated data.

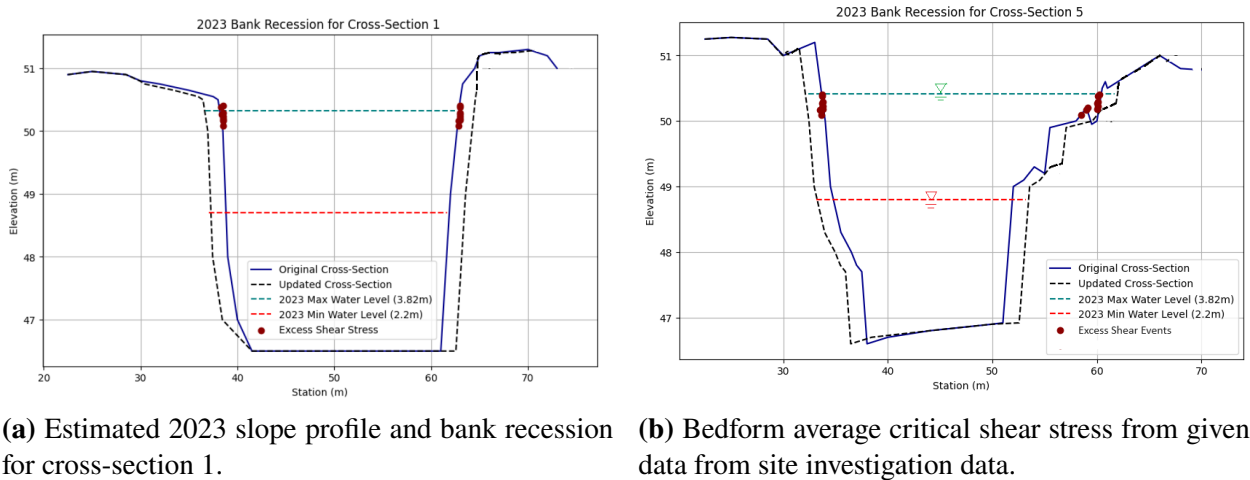


Fig. 17. Estimated 2023 slope profile and bank recession for cross-section 5.

Mitigation Measures

To effectively address the meandering and erosion challenges at Bear Brook Creek, a comprehensive approach involving multiple mitigation measures is essential for determining appropriate recourse. Based on site history, protection measures historically involved applying various Rip Rap installations which eventually erode due to the high variability in water level.

The channel's primary modes of deterioration involve toe scouring around the bends, flow-induced shear stress at various stages leading to global bank recession, and landslides due to the erodible nature of the overburden material and top layer along the bank. Due to the volatility of the hydraulic behavior, multiple mitigation solutions such as slope armoring (riprap) and submerged vane structures are presented to discuss how they can reduce the meandering effects. Structures such as bendway weirs, head control structures, and traditional weirs could be useful in controlling hydraulic behavior, but they present detrimental effects, possibly outweighing their benefits. Bendway weirs stabilize channels by also widening of the channel, which may lead to unintended consequences such as increased bank shear stress, sediment transport, and deposition downstream, potentially exacerbating erosion in other areas (U.S. Army Corps of Engineers, Bureau of Reclamation, and Tennessee Valley Authority, 2019) . For this application, the river width cannot increase since the road side slope at risk has a lower elevation than the inner bend, thus the problem would not lead to positive results. Head control structures and traditional weirs can significantly alter flow regimes and sediment transport dynamics, potentially leading to upstream sedimentation and downstream erosion. Since the catchment has a very low slope, flow control structures risk flooding upstream areas, causing backwater flooding during events. Therefore, the focus of the mitigation strategy is on riprap and river vanes is chosen due to their ability to directly mitigate shear stress and erosion at specific locations without causing significant alterations to the overall flow and sediment transport patterns of the river. Riprap provides immediate protection against toe scouring and bank erosion by stabilizing the bank material, while river vanes can effectively redirect flow away from vulnerable areas, reducing shear stress and preventing further bank deterioration.

The Province of British Columbia's Ministry of Environment Riprap Design and Construction

Guide proposes a minor repair framework which includes evaluating river training within the bank erosion framework when other modes of repair present themselves as problematic. This site has already been subject to riprap protection periodically, and cross section 5 shows signs of scouring at the toe. When the riprap solution fails the framework involves reassessing the feasibility of slope armouring, as well as the possibility of river training, sheet piling and other approaches as shown in the Figure 20 below.

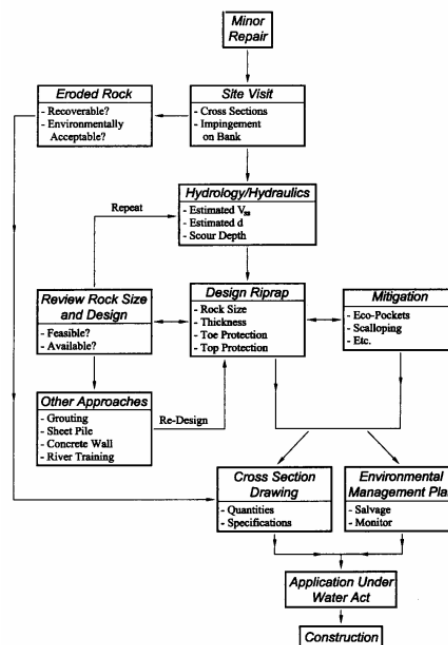


Fig. 18. Riprap Design and Construction Guide, Ministry of Environment, (2000)

Slope Armouring

Slope Armouring (riprap protection) is an essential tool in improving slope stability, and in river engineering it is used primarily to protect riverbanks from erosion caused by hydraulic forces. It involves the placement of large rocks or boulders along the bank, creating a revetment that absorbs and dissipates the energy of flowing water. This method increases the slope's strength and resistance to shear stress, providing a robust defense against erosion. Riprap is designed to stabilize the bank by providing a durable and resistant layer that protects the soil from the erosive forces of flowing water, thereby minimizing the potential for soil erosion and slope failure.

Riprap offers several advantages, such as its durability, flexibility, ease of installation, and environmental benefits. It is highly resistant to weathering and hydraulic forces, making it a long-lasting solution. The structure can adapt to minor changes in the underlying soil and flow conditions without significant loss of stability. Additionally, riprap can be installed relatively quickly and does not require specialized equipment or extensive technical knowledge. When properly designed, riprap can create diverse habitats, supporting aquatic life by providing various water depths and flow conditions. However, riprap also presents some disadvantages. Large rocks can alter the natural appearance of the riverbank, potentially conflicting with visual or recreational values. Over time, periodic inspections and potential repositioning or replacement of rocks are necessary to maintain the integrity of the revetment. Furthermore, effects such as toe scouring, vegetation growth, and improper sizing of rocks can accelerate the degradation and utility of rip rap protection. Toe scour, where the base of the revetment is undermined by water flow, can be mitigated by using a launching apron or a toe trench filled with rock to provide a flexible base. Vegetation growth between the rocks can destabilize the structure, requiring regular maintenance to manage invasive plants and tress. Using inadequately sized rocks can lead to failure, so ensuring proper rock size through detailed design calculations is crucial.

Designing an effective riprap revetment involves several key calculations to ensure stability and performance under various hydraulic conditions. Below are the essential equations and steps involved in the design process outlined by the Riprap Design and Construction Guide of British Columbia (Northwest Hydraulic Consultants Ltd., 2000) :

Coefficient of Velocity C_v :

$$C_v = 1.283 - 0.2 \log_{10} \left(\frac{R}{W} \right) \quad (23)$$

where R is the bend centerline radius, and W is the water-surface width.

Local Depth-Averaged Velocity V_{ss} :

$$V_{ss} = C_v \times V_{avg} \quad (24)$$

where V_{avg} is the mean flow velocity.

Regime Depth y_f :

$$y_f = \left(\frac{q_f^2}{F_{b0}} \right)^{1/3} \quad (25)$$

where q_f is the flood discharge intensity, and F_{b0} is derived from Blench's chart based on the median diameter of bed material (Blench, 1959).

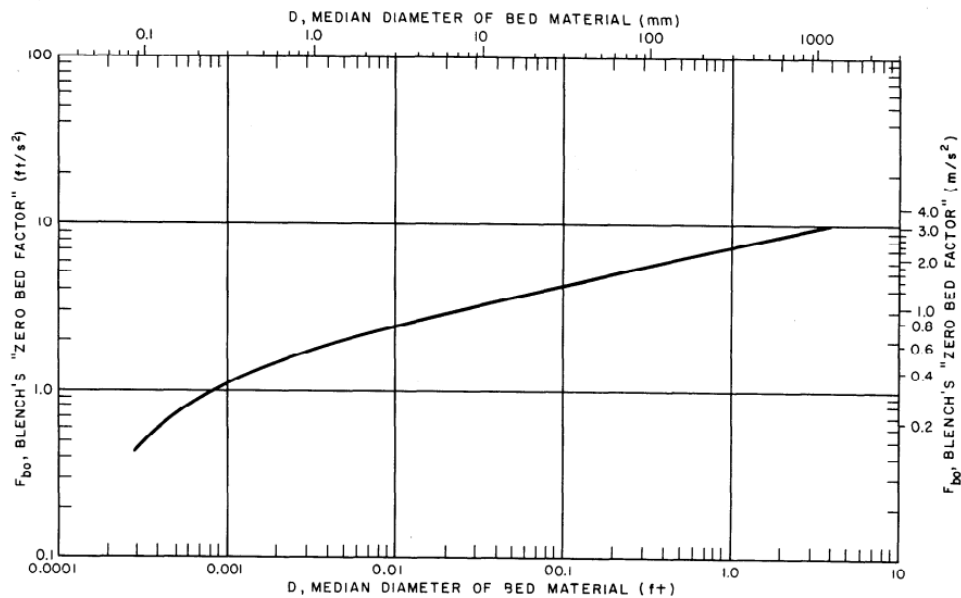


CHART FOR ESTIMATING F_{b0} (AFTER BLENCH)
Figure 9. - Chart for estimating F_{b0} (after Blench, 1969).

Fig. 19. Chart for Estimating F_{b0} , Figure 9, (Pemberton and Lara, 1984)

Maximum Scour Depth:

$$D_s = Z \times y_f \quad (26)$$

with Z being the multiplier based on flow conditions (1.5 to 2.8).

Required Rock Size D_{30} : The required rock size is determined based on local hydraulic conditions using velocity and depth relationships.

Median Rock Size D_{50} :

$$D_{50} = 1.25 \times D_{30} \quad (27)$$

Weight of Stone W :

$$W = \frac{\pi D^3 \gamma_s}{6} \quad (28)$$

where D is the b-axis dimension, and γ_s is the specific weight of the rock.

Thickness of Riprap Layer T :

$$t = 1.5 \times D_{50} \quad (29)$$

Volume of Rock Required per Meter V_T :

$$V_T = 3.35T \times D_s \quad (30)$$

where D_s is the predicted depth of scour.

The design process for riprap at the specified location involves detailed calculations to ensure the stability and effectiveness of the revetment. The coefficient of velocity (C_v) was determined to be 1.159, leading to a local depth-averaged velocity (V_{ss}) of 0.224 m/s as the design considers the critical flow conditions along the bank of 24.8 cms. The regime depth (y_f), based on the critical flow conditions, was calculated to be 0.435 m. Considering the potential for maximum scour depth, the design identified a maximum scour depth of 0.652 m. To mitigate these conditions, the design specified a required rock size (D_{30}) of 0.61 m, which corresponds to a median rock size (D_{50}) of 0.7625 m.

Given these parameters, the weight of each stone was calculated to be approximately 0.315 kg given an approximate specific weight of 2.65. To ensure adequate protection, the riprap layer's cover thickness was determined to be 1.144 m. Based on this thickness, the volume of rock required per meter length of the bank was calculated to be 2.498 cubic meters. For a total affected bank length of 35 meters, the total volume of rock required is approximately 87.429 cubic meters for each of the two banks. This preliminary design ensures that the riprap can feasibly protect the riverbank from erosion, considering the specific hydraulic conditions and characteristics of the site. Since this primarily addresses the hydraulic parameters, additional slope stability is required to determine the slope design for this solution.

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conditions using governing equations for vane induced sediment transport, described by River Training and Sediment Management with Submerged Vanes by (Odgaard, 2009), as shown below:

Lift and Drag Forces:

$$F_l = \frac{1}{2} \rho u^2 A c_l(\alpha) \quad (31)$$

$$c_L = \frac{2\pi\alpha}{1 + \frac{L}{H}} \quad (32)$$

Where L and H represent the selected vane parameters.

$$F_d = \frac{1}{2} \rho u^2 A c_d(\alpha) \quad (33)$$

$$c_D = \frac{1}{2\pi} \frac{L}{H} c_L^2 \quad (34)$$

Bed Shear Stress: The vane-induced transverse bed shear stress (τ_m) is given by:

$$\tau_m = \lambda \beta_s \frac{F_l}{A_v} \quad (35)$$

Where β_s is defined in equation (48), λ is the vane interaction coefficient, and A_v is the vane area. Because α is a critical parameter for determining the drag along the vane induced by circulation, minimizing the attack angle increases the performance of circulation however, it also reduces the applied force, in turn, the shear stress exerted on the bed. The optimization of the attack angle thus directly affects the sediment transport. Odegard et al., (1988), has proven that recommended vane structures be installed at attack angles ranging from 10-20 degrees however, from the bank erosion results, since the critical shear stress is very low due to the soil type, a smaller attack angle of 10 degrees can be used for the vane layout to induce the required sediment transport effects for deposition, and thalweg scouring.

Given the channel's hydraulic properties, we calculate the design parameters as shown by the following equations:

Friction parameter (m):

$$m = \frac{\kappa L}{\sqrt{8gSd}} \quad (36)$$

where κ is the von Karman constant (0.4).

Sediment Froude number (F_p):

$$F_p = \frac{u}{\sqrt{gD}} \quad (37)$$

where u is the average flow velocity and D is the median grain size.

Depth-width ratio (d_b/b):

$$\frac{d_b}{b} \quad (38)$$

Width-radius ratio (b/r_c):

$$\frac{b}{r_c} \quad (39)$$

Maximum change of depth for the deposited berm ($d_m - d_v$) : $d_m = 1.9d_o$ (40) where d_o is the original depth.

As there is a range of possible designs based on the aspect ratio, size and attack angle of the vane, the relative geometry and site constraints allow for vane geometries to be tested using the following parameters: - Height (H): Various values (0.36 m, 0.9 m, 1.4 m)- Length (L): Calculated based on height - Angle of attack (α): Various values (10, 15, 20 degrees).

$$0.12 < \frac{H}{d} < 0.48 \quad (41)$$

Relative vane submergence (T/d):

$$\frac{T}{d} \quad (42)$$

Vane height-length ratio (H/L):

$$\frac{H}{L} \quad (43)$$

Based on the design procedures, the channel geometry and calculated parameters determine the number of vanes per array (N), longitudinal spacing (δ_s) = $L \times N$, lateral spacing (δ_n)= $3H$, and

vane to bank distance (δ_b)= (δ_n - L) /2. Table 6 below shows 3 possible vane configurations based on various possible vane sizes and aspect ratios.

Parameters	(H=0.36, α = 10)	(H=0.6, α =10)	(H=1.0, α = 10)
Vane length, L (m)	2.88	4.8	8
Number of vanes per array, N	4	2	2
Vane submergence, T(m)	2.64	2.4	2
Lateral vane spacing, δ_n (m)	2.4	4	6
Longitudinal vane spacing, δ_s (m)	14.4	14.4	16
Vane-to-bank distance, δ_b (m)	4.56	3.6	2.4
Height of berms, $d_o - d_v$ (m)	2.7	2.7	2.7

TABLE 6. Comparative Analysis of Vane Design with 10-degree Angle of Attack

Given the possible solutions, the structures must be feasible while minimizing the number of vanes within arrays. Given the 3 solutions, option 2 with a height of 0.6m and 4.8m in length is the most optimal design here.

DISCUSSION

The bank erosion study of Bear Brook Creek using a coupled 1D JP Meander, 2D FaSTMECH model and a numerical bank erosion model provides hydrologically driven estimates into erosion processes and provides a foundation for feasible mitigation measures. This study successfully demonstrates the capability of quasi-3D modeling techniques to predict meander migration and bank erosion under varying hydrologic and environmental conditions. Key findings from this study highlight the critical influence of hydrology and soil parameters along a stream bank to determine the river morphology due to erosion.

Theoretical Background and Study Rationale

In modelling the morphological evolution, JP Meander model shows that lower FD values correspond to increased meander migration, effectively capturing natural sinuosity changes with limited data while higher values correspond to lower levels of migration. By leveraging channel characteristics and theory to determine cross-sectional flow conditions, the model eliminates the need for input frictional values, utilizing an FD value and a scaling coefficient C_f . This approach ensures that the solutions converge within the range of Manning's n , given the grain size parameters,

FD, flow, and C_f values. Despite its dependency on boundary conditions, the model's calibration using a C_f value of 2 ensures comparability and scales the roughness to 0.032, the average C_d value in this case.

Modeling Approaches and Empirical Analysis

FaSTMECH, on the other hand, couples 1D water surface elevations with 2D channel geometries, providing detailed velocity and shear stress fields. The model emphasizes the importance of 2D and 3D modeling, as mean velocity analyses often overlook terrain variations evident in meander bends where erosion occurs. This is shown by comparing the thalweg shear stresses versus the near-bank shear stresses, which showed a reduction up to 67 %. Comparing mean flow to near-bank velocities and stresses reveals lower shear stresses and velocities at outer bends due to attack angles and higher elevations (sand bars) on inner bends. However, the water surface profile reveals that the outer bends in regions with landslides level the bed profile, or increase the bed elevation, which locally increases the near-bank shear stress, but not to a level beyond the stresses at the centerline flows. What this reveals is landslides caused by hydraulic action cause sediment deposition at the toe, which increases the bank shear stress, in turn increasing the erosion rates which leads to higher meander migration. This is shown in the JP Meander model when the meander centrelines become more spaced apart as time increases. This increased shear stress response showing in the FaSTMECH model further validates the meander migration and channel mean shear stresses predicted by the JP Meander model. The model as it is dependent on the quality of the mesh and inputs, produces a mesh error was a maximum of 7 cfs, primarily at the second bend, showing a 25% error around one bend section. As the model converges on meshed values based on roughness and flow depth, the landslides which deposit sediment at the outer bend affect the flows, creating larger errors. Specifically, around the second meander bend, there is a municipal drain outlet which also deposits more sediment, in turn, increasing the surface profile.

Empirical Erosion Rates and Critical Shear Stress Equations

Empirical erosion rates derived from critical shear stress equations strongly correlate with observed data, indicating varying degrees of erodibility for silty sand and brown clay along Bear Brook

Creek. The predicted yearly bank erosion rates align closely with field observations, confirming the reliability of the coupled modeling approach. This study's results show an improvement over previous works by providing a more accurate, site specific k_d equation for silty sand, and for stiff grey clay:

$$k_d = 2.63\tau_c^{0.353}, k_d = 2.58\tau_c^{-0.329}$$

with the same R^2 coefficient as Islam et al of 0.79. The R^2 increases to 0.81 when the range of applicability is extended to 0.6 to 20 Pascals, similar to Wynn's work. However, within the range of 0.6 to 5.93 Pascals, the results approach those of Islam et al., who based their upper shear stress limit on the Julian and Torres equation with an upper limit of 3.53 Pa. This study employed the same approach to determine the actual maximum critical shear stress for the soil type, differing from Islam's values based on varying soil conditions. The Neil equation was also considered, but it depends on full channel depth, including the water weight. Thus, near-bank shear stress uses Neil's maximum value to quantify the bounds of the problem.

Comparison of Historical Data with Predicted Values

Comparing the calculated erosion using the historic data from the last 5 years to the predicted values shows notable differences. The predicted erosion lengths for average flooding flow rates of 28.8 m³/s, as illustrated in Figures 13 and 14, suggest significant meander migration at specific cross-sections. For instance, meander lengths are predicted to be 6.67 m at cross-section 2 and 6.34 m at cross-section 5 over a 5-year period. When protective measures like slope armoring are applied, these values reduce substantially, highlighting the importance of implementing such mitigation strategies. The total erosion from 2018 to 2023 for silty sand was calculated as 6.202660 m, and for stiff grey clay, it was 2.925829 m. These values are indicative of the significant erosion potential in the absence of mitigation measures.

Cold Climate Impacts on Erosion

This erosion model only considers one environmental load being the current force. Further loads include the impact of cold climate conditions, debris loads such as trees, ice, and wave loads, influencing applied shear stresses and erosion rates. Applying thermal effects in the excess shear

stress equation would provide a more accurate representation of environmental loads. Freeze-thaw cycles weaken critical shear stress, while adfreeze strength in winter conditions increases critical shear stress. Quantifying freeze-thaw events and ice build-up would further improve model accuracy since the applied stress in the excess shear stress equation theoretically represents all applied loads counteracting the critical shear stress of the soil material.

Site-Specific Mitigation Measures

Considering the two problematic areas in question (cross-section 2 and 5), mitigation measures are required to prevent the loss of critical roadway infrastructure. Cross-section 2 has approximately 6-8 meters in clearance to the road shoulder while cross-section 5 has 4-5 m. According to the "Low Water Response" paper by South Nation Conservation, they state a slope proximity hazard limit of 5 m from a river. Both cross-sections have similar material being silty sand and stiff grey clay; however, cross-section 5 has a significant amount of clay on the upper layer. Although Cross-section 5 is within the hazard limit area, it has potentially less erodibility as shown by the lower stresses in the FaSTMECH results, thereby elongating the period of erosion. Cross-section 2 on the other hand has a soil composition dominated by silty sand which is more erodible. As shown by the JP Meander results, future erosion can be up to 6.67 m, while historical flow data indicates that without protection, the river has the potential to meander between 6.2 and 2.9 m depending on the soil layer being eroded. Past landslides show slipping failures in individual events, receding the bank by approximately 4 m (the silty sand layer at cross-section 5), and in another case, approximately a 3 m bank recession due to a landslide at cross-section 3. Comparing the potential meander between the JP Meander and numerical results, there presents a 7% difference in estimated erosion. In this cross-section, it indicates that the numerical model supported by the FaSTMECH model is accurate with respect to the meander migration model.

Recommendations for Mitigation

When considering mitigation measures, slope armoring is recommended as a short-term solution at both bends. However, cross-section 2 has potential for river training to mitigate slope remediation. The reason why river training is preferred is that the sensitive soils should not be disturbed by bank

cuts in order to build a rock slope. Since high flow events displace and wash away riprap protection, training the river to avoid excess shear erosion is preferred. Overall, this study evaluates bank erosion models while improving and characterizing these empirical formulae for the Bear Brook Creek site, while applying quasi 3D flow modeling to estimate bank erosion at cross-sections 2 and 5 along Bear Brook Creek, while presenting two possible solutions to mitigate the problem.

CONCLUSION

It was observed that bank shear stress was greatly influenced by the magnitude of discharge for given soil properties. From the study, it was found that shear stresses on the bank showed fluctuating patterns depending on the river flow, due to changes in water surface profiles due to sediment deposition after landslides around the meandering bends. A discharge rate of 5.1 m³/s was identified as the mean flow velocity, a discharge of 24.8 m³/s point in which excess shear stress induces the riverbank to erosion, the average flooding event discharge is 28.8 m³/s and the maximum recorded discharge is 128 m³/s.

In this study, the critical shear stress (τ_c) for the susceptible silty sand layer in the selected reach of Bear Brook Creek varied from 0.155 to 5.93 Pa, and the erodibility coefficient (k_d) ranged from 1.39 to 4.99 cm³/N.s. From the numerical solution, values for current induced bank shear stress showed an average of 0.241 Pa to 700.76 Pa while the average flood event has a shear stress of 7.93 Pa, which exceeds the calculated τ_c . These stresses are derived from centerline flows, which in the hydrodynamic simulation showed a near bank shear stress reduction of up to 67% of the center line shear stresses. The bank erosion results however utilize stream gauge data, and represent a conservative estimate of the bank erosion along cross-section 2 and 5. Therefore, these findings demonstrate that the riverbank is vulnerable to erosion under flooding flows. Analyzing τ_c and k_d values, it was found that the bank materials of the selected reach in Bear Brook Creek fall into the category of 'erodible' to 'very erodible.'

From empirical estimates, maximum erosion rates varied from 1.02 to 2.17 m/year due to the variation in obtained results for τ_c and k_d . The predicted annual average fluvial erosion rate was found to range between 0.223 and 2.17 m/year for the stiff grey clay and silty sand layers

respectively, showing an average erosion rate of 1.03 m/y for silty sand and 0.486 m/y for brown clay. As the layer thickness for both materials vary across the bank the actual erosion driven by slope failure is dependent on a slope analyses however, the hydraulic induced stress provides a quasi 3-D excess shear stress model to evaluate the erosion behaviour. Therefore, from the predicted erosion rates, it can be concluded that the riverbank of Bear Brook Creek under study is highly erodible, while both cross sections can benefit from submerged vanes for river training and slope armouring for stability.

Further Research Directions

Future research should focus on analyzing the behavior of saturated cyclically frozen ground along riverbanks to better understand the reduction in strength at both the top and bottom layers. Considering flow impacts along the bank, more detailed factors within the excess shear stress equilibrium between the resisting forces and driving forces should be investigated to improve the model. Furthermore, the impacts of climate change, including extreme weather events and fluctuating flow conditions, should be integrated into the models to predict long-term erosion trends with higher accuracy. Additionally, incorporating more detailed data such as bathymetry, nearby stream gauges, temperature sensors, seismometers, site exploration reports, and overburden material analyses will enhance the robustness and accuracy of the models. These improvements will provide a more comprehensive understanding of the erosion processes and support the development of more effective and sustainable management practices.

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