EXPERIMENTS, MODEL DEVELOPMENT AND BANK STABILITY SIMULATIONS TO ASSESS BANK EROSION RATES AND POTENTIAL MITIGATION STRATEGIES

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April 2013

Prepared as part of agreement #60-6408-8-088 (Enhanced Stream-Corridor Modeling Tools for Adaptive Management of Tahoe Basin Streams; P003) with:

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This research was supported using funds provided by the Bureau of Land Management through the sale of public lands as authorized by the Southern Nevada Public Land Management Act.
TABLE OF CONTENTS

TABLE OF CONTENTS ........................................................................................................ II
LIST OF ABBREVIATIONS AND UNITS ........................................................................ VII
LIST OF ABBREVIATIONS AND UNITS ........................................................................ VII
CONVERSION FACTORS AND TEMPERATURE .............................................................. IX
INTRODUCTION AND BACKGROUND ....................................................................... 1
1. Research on Bank Processes ...................................................................................... 3
2. Development of BSTEM-Dynamic ............................................................................ 3
3. Effectiveness of Erosion-Control Strategies .............................................................. 3
1. RESEARCH ON BANK PROCESSES ....................................................................... 5
1.1 EXPERIMENTS TO QUANTIFY THE EFFECT OF FAILED BLOCKS ON BANK
STABILITY AND EROSION RATES ............................................................................. 6
1.1.1 Entrainment of Failed Blocks ......................................................................... 6
1.2 QUANTIFYING ROOT-REINFORCEMENT OF SPECIES FOUND IN THE UPPER
TRUCKEE AND TROUT CREEK WATERSHEDS ....................................................... 17
1.2.1 Methodology ..................................................................................................... 17
1.2.2 Results of root tensile strength testing and root densities for different species .... 19
   Addition of Tahoe species to BSTEM Static and BSTEM Dynamic ......................... 20
1.2.3 Results of BSTEM runs with and without vegetation ....................................... 24
2. DEVELOPMENT OF BSTEM-DYNAMIC2.1 STREAMBANK-EROSION PROCESSES. 26
2.1 STREAMBANK-EROSION PROCESSES ................................................................ 27
2.1.1 History and Structure of the Bank Stability and Toe-Erosion Model (BSTEM) ...... 28
2.1.2 Hydraulic Erosion Sub-Model .......................................................................... 28
   Sediment Entrainment and the Shields Curve ......................................................... 28
   Mechanisms of Cohesive Sediment Erosion ............................................................ 29
   Predicting the Distribution of Near-Bank Shear Stress ........................................... 31
2.1.3 Geotechnical/Mass Failure Sub-Model ............................................................. 33
   Quantifying the Resisting and Driving Geotechnical Forces .................................. 34
2.1.4 Root Reinforcement Sub-Model (Riproot) ......................................................... 35
2.1.5 Assessing the Potential for Geotechnical Failure ............................................. 37
   Horizontal Layer Method ....................................................................................... 37
   Cantilever Shear Failure Algorithm ....................................................................... 38
   Locating the failure plane that minimizes $F_s$ ......................................................... 38
2.1.6 Modeling Movement of the Groundwater Table ................................................. 39
2.1.7 Data Requirements ........................................................................................... 40
2.2 VALIDATION OF THE NEAR-BANK GROUNDWATER MODEL ......................... 44
2.2.1 Metholodgy ...................................................................................................... 44
2.2.2 Results ............................................................................................................. 45
3. EFFECTIVENESS OF EROSION CONTROL STRATEGIES3.1 BANK-STABILITY
MODELING FOR STREAM RESTORATION ............................................................... 49
3.1 BANK-STABILITY MODELING FOR STREAM RESTORATION ......................... 50
3.1.1 Methodology ................................................................................................... 51
3.1.2 Model Runs to Explore Current Loading Rates And Potential Mitigation Techniques
................................................................................................................................. 60
3.2 INVESTIGATING THE USE OF ENGINEERED LOG JAMS FOR REDUCING STREAMBANK EROSION................................................................. 68
3.2.1 Methodology ............................................................................. 68
3.2.2 Results ....................................................................................... 70
4. SUMMARY AND CONCLUSIONS ........................................................ 72
REFERENCES .................................................................................. 76
LIST OF FIGURES

Figure 1. Map showing the locations of the sites modeled within BSTEM on the Upper Truckee River and Trout Creek. ................................................................. 4
Figure 2. Photograph of UTR3 showing failed blocks at the bank toe.......................... 6
Figure 3. Comparison of frequency distributions of the intermediate particle diameter of failed blocks deposited at the bank toe of UTR3 .......................................................................................................................... 7
Figure 4. Schematic of jet-test device (from Hanson and Simon, 2001). ........................... 8
Figure 5. Mini-jet (~0.12 m diameter) including foundation ring, submergence tank, rotating head, outlet, water delivery connections, gauge, valve, outlet, snap clamps, and depth gauge..... 9
Figure 6. Relation between root biomass and critical shear stress of failed bank materials showing increased resistance with increasing root density.............................................. 10
Figure 7. Variation in average boundary shear stress at UTR3 over the period of field study. The critical shear stress of various block conditions are shown for comparison. Plotted critical shear stress values are from Table 1......................................................................................................................... 11
Figure 8. Flow depth at UTR3 required to attain an average boundary shear stress capable of entraining failed blocks under various vegetative conditions (See Table 1). ......................... 12
Figure 9. Cumulative erosion at UTR3 under the two modeling scenarios. Note: Brown line represents bare, in situ bank-toe material; green line represents bank toes containing vegetated blocks. ........................................................................................................... 14
Figure 10. Relative contributions of hydraulic (dark blue), geotechnical (beige) to total erosion (green) for the two modeling scenarios; bare, in situ material (top graph), and vegetated blocks (bottom graph). Cumulative erosion is expressed in m$^3$ ................................................................................... 15
Figure 11. Bank profiles of the modeled bank at UTR3 showing 1.4 m of top-bank retreat between the summers of 2008 and 2009. Simulated retreat rate for the case where blocks were accounted for was 1.2m. ......................................................................................................................... 16
Figure 12. Measuring the tensile strength of Geyer’s willow roots on a streambank of the Upper Truckee River, CA, using the Root Puller. ................................................................. 18
Figure 13. Close-up photographs showing the way the load cell and roots are connected during the root pulling process. ......................................................................................... 18
Figure 14. Comparison of Tahoe riparian species root-tensile strength curves with those obtained for other riparian species across the USA. ............................................................................. 19
Figure 15. Screenshot showing dropdown box containing new species added to the RipRoot algorithm for the Tahoe basin.................................................................................................................. 20
Figure 16. Average number of roots crossing a meter squared failure plane as a tree matures, as used in RipRoot (Modified from Pollen-Bankhead and Simon, 2009). ........................................ 21
Figure 17. Cumulative growth curves used in RipRoot for shrubs, perennial grasses and trees. 23
Figure 18. Growth pattern used in RipRoot for annual grasses to represent growth and die-back of roots over the seasons. ........................................................................................................... 23
Figure 19 Types of streambank failures........................................................................... 27
Figure 20 Shields diagram for incipient motion (modified from Buffington, 1999). The y-axis is defined by equation 1 and the x-axis is defined by equation 2 .................................................. 29
Figure 21 Three modes of cohesive sediment erosion: a) surface erosion of bed aggregates; b) mass erosion of the bed; c) entrainment of fluid mud (from Mehta, 1991, fig.1, p.41) ................. 31
Figure 22 Segmentation of local flow areas and hydraulic radii. ......................................... 32

IV
Figure 23. Monitoring site at Trout 1, showing (left) tensiometer locations and logger housing, and (right) the pressure transducer set up to measure water surface elevation.

Figure 24. Tensiometer and precipitation data from Trout1 (above) and UTR3 (below) monitoring sites.

Figure 25. Before and after profile from groundwater validation run at the Trout 1 site.

Figure 26. Before and after profile from groundwater validation run at the UTR3 site.

Figure 27. Comparison of groundwater output from BSTEM compared to monitored groundwater data over the same time period at the Trout 1 site.

Figure 28. Comparison of groundwater output from BSTEM compared to monitored groundwater data over the same time period at the UTR3 site.

Figure 29. Field photos showing (left) the BST device and (right) jet test devices being used on Trout Creek.

Figure 30. Schematic representation of borehole shear tester (BST) used to determine cohesive and frictional strengths of *in situ* bank materials. Modified from Thorne *et al.* (1981).

Figure 31. Flow stage data examples for the UTR3 site and the Trout1 site.

Figure 32. Bank geometry, layer depths and bank material properties for Trout 1 site.

Figure 33. Bank geometry, layer depths and bank material properties for Trout 4 site.

Figure 34. Bank geometry, layer depths and bank material properties for UTR Hole 6 site.

Figure 35. Bank geometry, layer depths and bank material properties for UTR 54 site.

Figure 36. Bank geometry, layer depths and bank material properties for UTR3 site.

Figure 37. Geotechnical erosion volumes under existing conditions and various mitigation scenarios. Values are in m3 per m of streambank, over one flow year.

Figure 38. Hydraulic erosion volumes under existing conditions and various mitigation scenarios. Values are in m3 per m of streambank, over one flow year.

Figure 39. Total erosion volumes under existing conditions and various mitigation scenarios. Values are in m3 per m of streambank, over one flow year.

Figure 40. Before and after profiles for existing and mitigated scenarios at UTR 54.

Figure 41. Before and after profiles for existing and mitigated scenarios at UTR Hole 6.

Figure 42. Before and after profiles for existing and mitigated scenarios at UTR 3.

Figure 43. Before and after profiles for existing and mitigated scenarios at TROUT 1.

Figure 44. Before and after profiles for existing and mitigated scenarios at TROUT 4.

Figure 45. Simulation of hydraulic erosion for a 4 m-high silt bank with a slope of 0.005 m/m. This eroded geometry is then exported into the bank-stability sub-model to evaluate geotechnical stability.

Figure 46. Simulation of bank stability for a 4 m-high silt bank with a slope of 0.005 m/m after hydraulic erosion by a bankfull flow. The resulting factor of safety was 1.39, indicating stability. Under drawdown conditions, this bank was unstable, failing along the red line shown in the figure.
Table 1. Results of mini-jets tests conducted along the bank toes of sites on the Upper Truckee River and Trout Creek. Note the significantly higher critical shear stresses for block permeated with grass roots. Data for site UTR3 are shown in bold as these data will be used in subsequent comparative analysis of bank-erosion rates at the site.

Table 2. Iterative modeling results at UTR3 assuming a 100 meter-long reach length. Note:: DD= failure occurred at under drawdown conditions.

Table 3. Average number of roots of different diameters measured for wet and dry meadow annual grass assemblages. All data are number of roots per square meter of streambank.

Table 4. Results of RipRoot algorithm, showing root-reinforcement values for common Lake Tahoe riparian species, growing in different streambank materials.

Table 5. Average cohesion values for bank material types, and percentage increase due to root reinforcement of a 30-year old stand of Geyer’s willow trees.

Table 6. Required input parameters for BSTEM.

Table 7. Default values in BSTEM (bold) for geotechnical properties. Data derived from more than 800 in situ direct-shear tests with the Iowa Borehole Shear Tester except where indicated.

Table 8. Potential alternative means to control the two primary processes that control streambank stability.

Table 9. Volumes of erosion from BSTEM runs at three sites along the Upper Truckee River and two sites along Trout Creek.

Table 10. Summary table of ELJ results for different bank heights and material types.
**LIST OF ABBREVIATIONS AND UNITS**

- \( a \) exponent assumed to equal 1.0
- \( A \) flow area \((m^2)\)
- \( c' \) effective cohesion, in kilopascals; kPa
- \( c_a \) apparent cohesion, in kilopascals; kPa
- \( c_f \) non-dimensional bed roughness coefficient
- \( D_{50} \) median diameter of grains in the bed, in meters; m
- \( F_s \) Factor of Safety
- \( g \) acceleration due to gravity, in meters per square second; \(9.807 \text{ m s}^{-2}\)
- \( h \) head, in meters; m
- \( i \) layer, node, or flow segment index
- \( I \) total number of soil layers
- \( j \) slice index
- \( J \) total number of slices
- \( k \) erodibility coefficient, in cubic centimeters per Newton second; \(\text{cm}^3\text{N}^{-1}\text{s}^{-1}\) or cubic meters per Newton second; \(\text{m}^3\text{N}^{-1}\text{s}^{-1}\)
- \( k_d \) erosion rate coefficient, in meters per second; \(\text{m s}^{-1}\)
- \( L \) length of the failure plane, in meters; m
- \( n \) Manning’s roughness coefficient, in seconds per cubic root of a meter; \(\text{s m}^{-1/3}\). If present, the subscripts \(g\), \(f\) and \(v\) signify the grain, form and vegetal components of the roughness
- \( P \) hydrostatic-confining force due to external water level, in kilonewtons per meter; kN m\(^{-1}\)
- \( R \) hydraulic radius \((\text{area/wetted perimeter})\), in meters; m
- \( \text{Re}^*_{c} \) critical roughness Reynolds number
- \( S \) channel gradient, in meters per meter; \(\text{m m}^{-1}\)
- \( t \) time, in seconds; s
- \( U \) mean flow velocity, in meters per second; \(\text{m s}^{-1}\)
- \( W \) failure block weight, in kilonewtons; kN
- \( z \) water surface elevation, in meters; m
- \( \alpha \) local bank angle, in degrees from horizontal
- \( \beta \) failure-plane angle, in degrees from horizontal
- \( \varepsilon \) rate of erosion, in meters per second; \(\text{m s}^{-1}\)
- \( \gamma \) bulk unit weight of soil, in kilonewtons per cubic meter; kN m\(^{-3}\)
- \( \gamma_w \) unit weight of water, in kilonewtons per cubic meter; kN m\(^{-3}\)
- \( \mu_a \) pore-air pressure, in kilopascals; kPa
- \( \mu_w \) pore-water pressure, in kilopascals; kPa
- \( \nu \) kinematic viscosity of water, in square meters per second; \(\text{m}^2\text{s}^{-1}\)
- \( \rho \) density of water, in kilograms per cubic meter; \(1000 \text{ kg m}^{-3}\)
- \( \rho_s \) density of the sediment, in kilograms per cubic meter; \(2650 \text{ kg m}^{-3}\)
- \( \sigma \) normal stress on the shear plane, in kilopascals; kPa
- \( \tau_c \) critical shear stress, in Pascals; Pa
- \( \tau^*_c \) dimensionless critical shear stress
- \( \tau_s \) soil shearing resistance, in kilopascals; kPa
\( \tau_0 \)  
bed shear stress, in Pascals; Pa. If present, the additional subscripts \( g, f \) and \( v \) signify the grain, form and vegetal components of the bed shear stress.

\( \phi' \)  
effective soil friction angle, in degrees

\( \phi^b \)  
angle describing the increase in shear strength due to an increase in matric suction \( (\mu_a - \mu_w) \), in degrees
### CONVERSION FACTORS AND TEMPERATURE

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<tr>
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<tr>
<td>kilonewton per meter (kN m⁻¹)</td>
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**Temperature:** Water temperature in degrees Celsius (°C) may be converted to degrees Fahrenheit (°F) as follows:

°F = 1.8 °C + 32
INTRODUCTION AND BACKGROUND

The Lake Tahoe Basin has a long history of human interaction and exploitation dating back to the 1850s. Activities such as logging, road construction, mining, overgrazing and urbanization have led to degradation of land and water resources and threaten to do irreparable damage to the lake. In particular are concerns for lake clarity, which have been partly attributed to the delivery of fine-grained sediment emanating from upland, urban and channel sources. Over the past 35 years, a trend of decreasing water clarity, as measured by secchi depth has been documented (Goldman, 1988).

Because lake clarity is related to the very fine particles that remain in suspension and that transport adsorbed constituents, it is essential to identify the magnitude and sources of these fine-grained materials in order to effectively simulate current and future water-clarity conditions in Lake Tahoe using a lake-clarity model. Furthermore, selection of appropriate management strategies must be founded on the identification of the controlling processes and associated source areas of fine sediment. These source areas can be broadly separated into uplands, urban areas and channels. More specifically, upland sources may include slopes, fields, roads, construction-site gullies etc., while channel sources may include channel beds, bars and streambanks. Moreover, the magnitude of sediment production, transport and delivery to the lake varies widely across the basin as a function of differences in precipitation, surficial geology, land use/land cover, and channel instabilities.

A number of studies have been completed in the past 30 years to address sediment delivery issues from various watersheds in the Lake Tahoe Basin. Earlier studies each focused on only a few streams within the watershed (Kroll, 1976; Glancy, 1988; Hill and Nolan, 1991; Stubblefield, 2002). More recent work by Reuter and Miller (2000), Rowe et al. (2002), Simon et al., (2003), and Simon (2008) used suspended-sediment transport data from the Lake Tahoe Interagency Monitoring Program (LTIMP), which brought together data from streams all around the watershed. These works have indicated that the following streams are among the largest contributors of suspended sediment to Lake Tahoe: Incline, Third, Blackwood, and Ward Creeks, and the Upper Truckee River. Most of the sediment is delivered during the spring snowmelt period (predominantly May and June), which correlates well with the spring reduction in secchi depth.

A recent study by Simon et al. (2003) estimated that about 25% of the median annual, fine-grained loading of sediment to the lake was derived from streambank erosion, making this an important source category. In fact, about 20% of all fine sediment delivered to Lake Tahoe was found to come from the banks of the Upper Truckee River and Blackwood Creek. If Ward Creek is also included, this value becomes 22%, with the remainder emanating from other watersheds around the lake.

Bank retreat and the associated sediment loadings from streambank erosion highlight the importance of interactions between hydraulic forces acting at the bed and bank toe, and gravitational forces acting on in situ bank material (Carson and Kirkby, 1972; Thorne, 1981; Simon et al., 1991). Failure occurs when erosion of the bank toe and the channel bed adjacent to the bank have increased the height and angle of the bank to the point that gravitational forces
exceed the shear strength of the bank material. Failures generally occur when the streambank material is at its weakest; when infiltration of water has decreased matric suction and increased pore-water pressure. This often occurs on the recessional limb of storm hydrographs when the bank loses the support provided by the water in the channel (Simon et al., 1999). Failed bank materials may be delivered directly to the flow and deposited as bed material, dispersed as wash load, deposited along the toe of the bank as intact blocks, or as smaller, dispersed aggregates (Simon et al., 1991). If deposited at the bank toe, failed bank material may temporarily increase bank stability by buttressing the bank and protecting in situ bank material from attack and entrainment by the flow. The properties of the failed bank material, in tandem with the hydraulic forces acting at the bank toe, control the residence time of failed bank material (Thorne, 1982).

To evaluate potential reduction in fine-sediment loadings emanating from streambanks, it was necessary to analyze the discrete processes that control streambank erosion under existing and mitigated conditions in terms of the controlling driving and resisting forces that affect steepening by hydraulic erosion and mass-bank stability, controlled by gravity. These processes include hydraulic erosion of bank-toe sediments, mass failure of upper-bank materials and the reinforcing effects of vegetation, if present. These processes can be modeled previously using the Bank-Stability and Toe-Erosion Model (BSTEM) developed by the USDA-ARS, National Sedimentation Laboratory (Simon et al., 1999; 2000). The model has been previously used successfully in the Tahoe Basin to model the influence of riparian vegetation on bank stability along a reach of the Upper Truckee River (Simon et al., 2006).

Model capabilities, however, needed to be improved so as to accommodate a dynamic groundwater table, the role of riparian vegetation on shear stress along the bank face, and the effect of failed material deposited at the bank toe. In addition, simulations of the applicability and impacts of potential erosion-control measures needed to be tested to provide quantitative evaluations of their utility for a range of riverine conditions.

To improve lake clarity it is essential to quantify the magnitude of the load reductions that could be expected from streambank erosion and other source categories such as upland, airborne, urban, and shoreline. A synthesis of the products generated from all of this research and development of a load-reduction strategy for Lake Tahoe will rely heavily on numerical simulations of lake clarity being conducted by the University of California, Davis.

In this report we will discuss the development of a dynamic version of BSTEM, allowing for input of a continuous hydrograph, an integrated near-bank groundwater model, and the root-reinforcement algorithm, RipRoot. The theory behind the development of this model will be discussed, along with runs carried out for three sites along the Upper Truckee River, and two sites along Trout Creek. These sites were modeled under existing conditions, and with various mitigation options aimed at quantifying the potential for reducing sediment loadings. A validation of the groundwater model for one site on the Upper Truckee and one site on Trout Creek is presented, and, in addition, we report on field experiments conducted to assess the impact of failed blocks of material on streambank erosion rates and bank stability. Field data collected on the Upper Truckee River and at Trout Creek to quantify root-reinforcement from wet and dry meadow grasses, and Geyer’s willow trees will also be reported.
OBJECTIVES AND SCOPE

The general research objective was to provide quantitative evaluations of the impact and applicability of various measures to control streambank erosion for the purpose of reducing the delivery of fine-grained sediment to Lake Tahoe. The scope of the project was Trout Creek and the Upper Truckee River (Figure 1) where all field-related work was conducted. Aspects of the project related to BSTEM are applicable to the entire basin. To accomplish the broad research goal, the following sub-objectives were separated into three major categories: (1) research on bank processes and, (2) development of a dynamic version of the Bank-Stability and Toe Erosion Model (BSTEM-Dynamic) with enhanced capabilities, and (3) application of BSTEM-Dynamic to test the effectiveness of erosion-control strategies.

1. Research on Bank Processes

1. Determine the erosion resistance of failed bank material (blocks), particularly those that are permeated with dense network of roots;
2. Develop a mechanistic methodology to account for the reduction in effective stress acting on the bank surface due to the presence of riparian vegetation;
3. Monitor surface-water stage and near-bank groundwater levels using pressure transducers and tensiometers to validate a near-bank groundwater model and support the CONCEPTS modeling effort described in ; and
4. Determine the root strength of selected riparian species for incorporation into the ‘library’ of species within the root-reinforcement sub-model (RipRoot).

2. Development of BSTEM-Dynamic

1. Develop a dynamic version of BSTEM that is capable of conducting continuous simulation of hydraulic and geotechnical processes over a broad range of time scales and time steps where bank geometry is updated after every time step;
2. Implement the effective effective-stress algorithm (from #2 above) within the toe-erosion sub-model;
3. Develop a dynamic, near-bank groundwater sub-model that updates at every time step and that is linked to the bank-stability sub-model within BSTEM;
4. Validate the near-bank groundwater sub-model with field data collected in #3 above;

3. Effectiveness of Erosion-Control Strategies

1. Test the effectiveness of various erosion-control strategies in reducing the delivery of streambank sediment by hydraulic and geotechnical processes; and
2. Test the conditions under which engineered log jams (ELJs) can be used to limit erosion control
Figure 1. Map showing the locations of the sites modeled within BSTEM on the Upper Truckee River and Trout Creek.
1. RESEARCH ON BANK PROCESSES
1.1 EXPERIMENTS TO QUANTIFY THE EFFECT OF FAILED BLOCKS ON BANK STABILITY AND EROSION RATES

1.1.1 Entrainment of Failed Blocks

Previous research on streambank erosion rates along the Upper Truckee River identified the importance of failed blocks that come to rest at the toe of the bank on erosion rates (Simon et al., 2009). In that study, streambank erosion rates were over-estimated because simulations of hydraulic erosion over a series of flow events were resisted by in situ material only. Because these blocks are generally permeated with a dense network of grass roots, it was hypothesized that they would be more resistant to hydraulic erosion by subsequent flows. Research was conducted at one site on the Upper Truckee River (UTR3) and two sites on Trout Creek to quantify the erosion resistance of root-permeated failure blocks.

Hydraulic erosion of failed blocks can occur by either particle-by-particle erosion of the block, resulting in a smaller block, or by entrainment of the entire block as an individual clast (Simon et al., 1999). Measurements of block dimensions deposited along the bank toe at UTR3 (Figure 2) were conducted during the summers of 2008 and 2009. Results showed a significant reduction in block size across 95% of the frequency distribution. The median diameter of failed blocks decreased from 85.5 cm to 57.5 cm (33%) in one year (Figure 3). It is likely that much of this occurred by particle-by-particle erosion in lieu of the deposition of new blocks by a series of failures. The implications of this decrease in size are that the blocks are more likely to be entrained en masse by the flow as they get smaller and entrainment thresholds become within the range of available the shear stresses at the site. In fact, several intact blocks were seen deposited in the thalweg just downstream of the apex of the bend.

Figure 2. Photograph of UTR3 showing failed blocks at the bank toe.
Tests of erosion resistance were conducted using a mini jet-test device. The original submerged jet-test was developed by the Agricultural Research Service (Hanson, 1990; Figure 4) for testing the in situ erodibility of surface materials (ASTM, 1995). This device was developed based on knowledge of the hydraulic characteristics of a submerged jet and the characteristics of soil-material erodibility. In an attempt to remove empiricism and to obtain direct measurements of $\tau_c$ and k, Hanson and Cook (1997) developed analytical procedures for determining soil k based on the diffusion principles of a submerged circular jet and the corresponding scour produced by the jet. These procedures are based on analytical techniques developed by Stein et al. (1993) for a planar jet at an overfall and extended by Stein and Nett (1997). Stein and Nett (1997) validated this approach in the laboratory using six different soil types.

As the scour depth increases with time, the applied shear stress decreases due to increasing dissipation of jet energy within the plunge pool. Detachment rate is initially high and asymptotically approaches zero as shear stress approaches the critical shear stress of the bed material. The difficulty in determining equilibrium scour depth is that the length of time required to reach equilibrium can be large. Blaisdell et al. (1981) observed during studies on pipe outlets that scour in cohesionless sands continued to progress even after 14 months. They developed a function to compute the equilibrium scour depth that assumes that the relation between scour and time follows a logarithmic-hyperbolic function. Fitting the jet-test data to the logarithmic-hyperbolic method described in Hanson and Cook (1997) can predetermine $\tau_c$. k is then estimated by curve-fitting measured values of scour depth versus time and minimizing the error of the measured time versus the predicted time. Both k and $\tau_c$ are treated as soil properties and the former does not generally correlate well with standard soil mechanical indices such as Atterberg limits. Instead, k is dependent on the physio-chemical parameters that determine the

Figure 3. Comparison of frequency distributions of the intermediate particle diameter of failed blocks deposited at the bank toe of UTR3.

Figure 4. Schematic of jet-test device (from Hanson and Simon, 2001).

At the request of the National Sedimentation Laboratory, a miniature version of the jet-test device was developed in 2008 by Dr. Greg Hanson of the USDA-Agricultural Research Service in Stillwater, OK (Figure 5). The mini-jet apparatus consists of an electric submersible 60 liters/second pump powered by a portable A/C generator, a scaled-down 0.12 m- diameter submergence tank with an integrated, rotatable 3.18 mm-diameter nozzle, depth gauge, and delivery hoses.
Results of the mini-jet tests display distinct differences between the resistance of bare bank materials and the failed blocks permeated with grass roots. Erosion resistance ($\tau_c$) for root-permeated blocks are generally about an order of magnitude higher than the bare soils at the three sites tested. The greatest resistance occurs when the block topples and comes to rest with the above-ground biomass facing the flow (‘block top parallel to flow’, in Table 1). In part, this is due to the fact that the grasses bend and cover the block surface. Intermediate resistance is obtained is attained from the roots alone, as with the tests corresponding to ‘block side parallel to the flow’.

Core samples of six blocks were obtained in the field during the summer of 2009 and used to extract the grass roots for laboratory analysis. The biomass of roots in each core was determined and expressed as a density in grams per cubic centimeter ($g/cm^3$). These results were then compared to the critical shear stresses obtained with the mini-jet device and showed reasonably good agreement. (Figure 6). Additional data collection of root cores and associated tests of erosion resistance could lead to an improved mechanistic technique to quantify soil resistance as a function of soil type and root density/biomass.
Table 1. Results of mini-jets tests conducted along the bank toes of sites on the Upper Truckee River and Trout Creek. Note the significantly higher critical shear stresses for block permeated with grass roots. Data for site UTR3 are shown in bold as these data will be used in subsequent comparative analysis of bank-erosion rates at the site.

<table>
<thead>
<tr>
<th>Site</th>
<th>Vegetation?</th>
<th>Condition</th>
<th>Median critical shear stress (Pa)</th>
<th>Erodibility (cm$^3$/N-s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UTR3</td>
<td>No</td>
<td>Bare bank toe</td>
<td>8.69</td>
<td>1.35</td>
</tr>
<tr>
<td>UTR3</td>
<td>Yes</td>
<td>Block side parallel to flow</td>
<td>12.5</td>
<td>0.28</td>
</tr>
<tr>
<td>UTR3</td>
<td>Yes</td>
<td>Block top parallel to flow</td>
<td>23.4</td>
<td>0.116</td>
</tr>
<tr>
<td>Trout 4</td>
<td>No</td>
<td>Bare bank toe</td>
<td>1.12</td>
<td>1.48</td>
</tr>
<tr>
<td>Trout 4</td>
<td>Yes</td>
<td>Block side parallel to flow</td>
<td>7.75</td>
<td>0.291</td>
</tr>
<tr>
<td>Trout 4</td>
<td>Yes</td>
<td>Block top parallel to flow</td>
<td>33.9</td>
<td>0.085</td>
</tr>
<tr>
<td>Trout 9</td>
<td>No</td>
<td>Bare bank toe</td>
<td>1.68</td>
<td>1.05</td>
</tr>
<tr>
<td>Trout 9</td>
<td>Yes</td>
<td>Block side parallel to flow</td>
<td>13.5</td>
<td>0.183</td>
</tr>
<tr>
<td>Trout 9</td>
<td>Yes</td>
<td>Block top parallel to flow</td>
<td>43.2</td>
<td>0.069</td>
</tr>
</tbody>
</table>

![Graph](image-url)

Figure 6. Relation between root biomass and critical shear stress of failed bank materials showing increased resistance with increasing root density.
Average boundary shear stresses ($\tau_o$) were calculated for the UTR3 site based on stage data obtained from the pressure transducer located at the site and channel slope surveyed in the field. The variation in average boundary shear stress over the period of field study are shown by the black trace plotted in Figure 7 and highlighted in blue. Without vegetation on the bank toe or permeated throughout failed blocks, it is clear that critical shear stresses would be exceeded for most of the period and would result in very high bank-erosion rates. Average boundary shear stresses in excess of the critical shear stress for toppled blocks occurred only during the peak of the snowmelt runoff in May 2009, and during several other peaks for the blocks whose roots were exposed to the flow (block parallel to flow; Figure 7). Another way of conceptualizing the role of vegetation on erosion resistance is in terms of the flow depth that would be required to initiate entrainment of the blocks under various vegetative conditions (Figure 7).

**Figure 7.** Variation in average boundary shear stress at UTR3 over the period of field study. The critical shear stresses of various block conditions are shown for comparison. Plotted critical shear stress values are from Table 1.
Figure 8. Flow depth at UTR3 required to attain an average boundary shear stress capable of entraining failed blocks under various vegetative conditions (See Table 1).

With bank surface or block material without vegetation, relatively low, and frequent flows are capable of undercutting the streambank ($\tau_c = 8.7$ Pa). In contrast, those blocks that toppled towards the channel and contain grasses require a much deeper flow (about 1.7 m-deep) to initiate particle-by-particle erosion of the block ($\tau_c = 23.4$ Pa) (Figure 8). Bankfull, average boundary shear stress is 29.5 Pa, indicating that continued particle-by-particle erosion of the larger blocks will be required before they can be entrained en masse by the flow.
In summary, these results seem intuitively correct but to test the validity and applicability of the critical shear stress measurements in terms of improving prediction of bank-erosion rates, a series of iterative modeling runs were conducted using the static version of BSTEM-5.4. Model runs were conducted assuming (1) only in situ material (without blocks or vegetation) on the bank toe, and (2) root-permeated blocks with a resistance of 23.4 Pa were at the bank toe. The flow events bounded by the dates of field work during the summers of 2008 and 2009 (shaded in blue in Figure 7) were used as input into the toe-erosion sub-model of BSTEM. The iterative process worked as follows:

1. Toe erosion was simulated for a given storm event;
2. The eroded geometry was exported into the geotechnical sub-model
3. Bank stability was simulated with the groundwater table set to the same elevation as the water in the channel;
4. If no failure occurred ($F_s > 1.0$) the bank-stability sub-model was run again with the surface water elevation set to the base flow following recession. This represents the drawdown, or most critical case.
5. If no failure occurred, the next flow event would be simulated. If failure occurred, the new geometry would be exported into the toe-erosion sub-model to simulate hydraulic erosion for the next storm event.
6. In this way, all events were simulated, erosion totals summed for all events and failure frequency noted.

Results of the iterative simulations are shown in Table 2 and Figure 9. Total erosion for the in situ case was 510 m$^3$ (over a 100 m length) compared to 197 m$^3$ over the same distance for the case with vegetated blocks; hydraulic erosion was 148 and 115 m$^3$, respectively. Although the differences in hydraulic erosion are not drastic, it is interesting to note the impact that this limited reduction in hydraulic erosion had on rates of geotechnical (mass failure) erosion. The in situ case showed 362 m$^3$ of erosion by mass failures (from three failure events) compared to 82 m$^3$ of erosion (from a single failure) for the case where the blocks offered some protection. Conceptually, these results are in complete agreement with those of the earlier study on the Upper Truckee River that showed the great importance of hydraulic erosion in aiding mass failure and lateral retreat of banks (Simon et al., 2009). Figure 10 is also provided to show the contributions of hydraulic and geotechnical erosion relative to total erosion for the two modeling scenarios.
Table 2. Iterative modeling results at UTR3 assuming a 100 meter-long reach length. Note:: DD= failure occurred at under drawdown conditions.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Maximum basal retreat (m)</th>
<th>Hydraulic erosion (m$^3$)</th>
<th>Top-bank retreat (m)</th>
<th># Failures</th>
<th>Geotechnical erosion (m$^3$)</th>
<th>Total erosion (m$^3$)</th>
<th>Toe contribution (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>In situ</td>
<td>4.0</td>
<td>148</td>
<td>2.8</td>
<td>3 (2:DD)</td>
<td>362</td>
<td>510</td>
<td>29.0</td>
</tr>
<tr>
<td>Blocks at toe</td>
<td>1.6</td>
<td>115</td>
<td>1.2</td>
<td>1 (1:DD)</td>
<td>82</td>
<td>197</td>
<td>58.3</td>
</tr>
</tbody>
</table>

Figure 9. Cumulative erosion at UTR3 under the two modeling scenarios. Note: Brown line represents bare, in situ bank-toe material; green line represents bank toes containing vegetated blocks.
Figure 10. Relative contributions of hydraulic (dark blue), geotechnical (beige) to total erosion (green) for the two modeling scenarios; bare, in situ material (top graph), and vegetated blocks (bottom graph). Cumulative erosion is expressed in m$^3$. 
Finally, comparison of the bank profiles during the summer of 2008 and 2009 show 1.4 m of bank retreat over the period (Figure 11) compared to 2.8 m for the *in situ* case and 1.2 m for the case with blocks. Results of these simulations support the original hypothesis as to why erosion rates were over-estimated by not accounting for the vegetated blocks at the bank. These features are typical of incised channels cutting through dry- and wet-meadow environments and clearly need to be accounted for in any mechanistic approach to predicting streambank stability and channel-migration rates. In combination with the data collected on the root reinforcement attributes of these meadow species, accurate modeling of bank-stability conditions can now be undertaken. As a prelude to using the empirical relation between root biomass and critical shear stress (owing to a low number of matching samples/tests), the increased critical shear stress of bank-surface due to the below ground biomass is accounted for by increasing the value obtained for bare soil by an order of magnitude. The effect of the above ground biomass is accounted for by evaluating the effective stress acting on the grains beneath the vegetation. This is discussed in greater detail in another section of this report.

**Figure 11.** Bank profiles of the modeled bank at UTR3 showing 1.4 m of top-bank retreat between the summers of 2008 and 2009. Simulated retreat rate for the case where blocks were accounted for was 1.2 m.
1.2 QUANTIFYING ROOT-REINFORCEMENT OF SPECIES FOUND IN THE UPPER TRUCKEE AND TROUT CREEK WATERSHEDS

In light of the increasing use of plants as an effective, economical and environmentally friendly solution for slope and streambank stabilization, the effects of vegetation on mass wasting processes have become of particular interest to watershed managers seeking to reduce sediment loadings in rivers. Vegetation exerts a number of controls on the geomorphic processes affecting slope and streambank stability. The manifestation of these controls are a number of hydraulic, hydrologic and mechanical effects, some of which have positive and some of which have negative impacts on soil stability. To evaluate whether a particular slope or streambank will be strengthened or weakened by vegetation of a given species and age, the balance between these effects must be considered. As part of this study, four riparian tree species typical of the Upper Truckee River and Trout Creek watersheds were selected for testing of root-reinforcement: lodgepole pine, Lemmon’s willow, Geyer’s willow and Alder. (*Pinus contorta, Salix Lemonii, Salix geyeriana, Alnus incana*). In addition, wet meadow and dry meadow grass assemblages were tested. Assemblages of grasses were tested instead of individual species as separating out the roots of individual grasses growing in each assemblage was difficult, and investigation of the entire assemblage was determined to be more representative of real life conditions, as opposed to a monoculture.

1.2.1 Methodology

Root tensile strengths were measured using a “Root Puller” device based on a design by Abernethy (1999). This is composed of a metal frame with a winch attached to a load cell connected to a data logger (Figure 12; Figure 13). The Root Puller was attached to the bank face, and different size roots were attached to the load cell using various size u-bolts. Cranking the winch applies a tensile stress to the root (measured as a load, in kg) that increases until tensile failure of the root occurs. The diameter of each root was recorded along with the logged history of tensile load. The maximum load applied to each root before breaking and root diameter were used to calculate the tensile strength of each root. Relations of root diameter and tensile strength were established for each species tested (Figure 14) to use as input to the root reinforcement model, RipRoot (Pollen and Simon, 2005; Pollen, 2007; Thomas and Pollen-Bankhead, 2010). Fifty roots of each species were tested for tensile strength.

The root systems of each species or meadow assemblage were examined and recorded using the wall profile method of Bohm (1979). In this case, the bank face acted as the profile wall, and visible roots were cut back to the face of the bank so that a grid 0.5 m by 0.5 m with 0.1 m increments could be placed against it. The use of the bank face as the wall profile means that the exposed roots may have been subject to weathering; only those roots that had not been washed away by flow events could be recorded. Root diameters were measured and recorded according to depth in the bank profile. Estimates of root-reinforcement provided by each species were calculated using the root architecture and tensile strength data that were collected, and the root-reinforcement algorithm, RipRoot.
**Figure 12.** Measuring the tensile strength of Geyer’s willow roots on a streambank of the Upper Truckee River, CA, using the Root Puller.

**Figure 13.** Close-up photographs showing the way the load cell and roots are connected during the root pulling process.
Results of root tensile strength testing along the Upper Truckee River and Trout Creek, showed that root strengths of the four tree species tested all fell within the range of root strengths previously measured for other riparian species across the USA (Figure 14). Root strengths of Geyer’s willow trees were, however, shown to be stronger per unit area than roots of corresponding diameters tested from lodgepole pine, alder or Lemmon’s willow trees at the study sites visited. This was especially true of roots with diameters greater than 1.8 mm (Figure 14). Wet meadow (measured on Trout Creek) and dry meadow (measured on the Upper Truckee) assemblages of annual grasses had tensile strength curves that were similar to perennial grass species tested at other locations across the USA; the smallest roots were stronger per unit area than many of the tree species studied, but larger roots, where present, were weaker per unit area than the woody roots of trees. A comparison of the tests carried out on wet and dry meadow grasses in this study indicated that the fine roots of each type of assemblage had very similar strengths, but the roots of the wet meadow assemblage were slightly stronger than those of the dry meadow assemblage. However, where thicker roots (>1.5 mm diameter) were seen, the roots of the dry meadow assemblage were slightly stronger.

Figure 14. Comparison of Tahoe riparian species root-tensile strength curves with those obtained for other riparian species across the USA.
Addition of Tahoe species to BSTEM Static and BSTEM Dynamic

The root-tensile strength curves obtained for the four Tahoe Basin tree species and for wet and dry meadow grass assemblages were added to the database of riparian species listed in the RipRoot algorithm in BSTEM Static and BSTEM Dynamic.

Figure 15. Screenshot showing dropdown box containing new species added to the RipRoot algorithm for the Tahoe basin.

For RipRoot to calculate the additional reinforcement provided by the roots of a given species or assemblage, the model must be given not only the tensile strength-diameter curves for each species (Figure 14), but also the number of roots of a given diameter crossing a potential failure plane within the streambank; these variables vary by species and plant age.

In-field measurements of root densities and diameters were recorded in the field for the tree and grass species studied. In the case of the tree species a limited range of tree ages were investigated, and so the average root-growth curve obtained for the other riparian species in the database was applied to Geyer’s willow, Lemmons willow, lodgepole pine and alder (Figure 16).
In the case of the wet and dry meadow assemblages, the root densities and diameter distributions measured in the field were used to represent values during the growing season, as measurements were made in September, before annual die-back of grass roots had begun. The average total values given in Table 3 of 330 and 460 roots per square meter of bank for dry and wet meadow grasses respectively are significantly lower than root densities measured for perennial grasses at other locations across the USA, such as Reed canarygrass and Alamo switchgrass. These perennial species can have root densities of as high as 5,000 roots per square meter, after several years of growth, and their rooting depths also tend to be greater than those recorded for these annual grasses.

Table 3. Average number of roots of different diameters measured for wet and dry meadow annual grass assemblages. All data are number of roots per square meter of streambank.

<table>
<thead>
<tr>
<th></th>
<th>&lt;1mm</th>
<th>1-2 mm</th>
<th>2-3mm</th>
<th>3-5mm</th>
<th>5-10mm</th>
<th>10-20mm</th>
<th>20-40mm</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Meadow</td>
<td>239</td>
<td>67</td>
<td>13</td>
<td>8</td>
<td>3</td>
<td>0</td>
<td>0</td>
<td>330</td>
</tr>
<tr>
<td>Wet Meadow</td>
<td>362</td>
<td>70</td>
<td>12</td>
<td>14</td>
<td>2</td>
<td>0</td>
<td>0</td>
<td>460</td>
</tr>
</tbody>
</table>
RipRoot also requires one final input to be able to calculate root-reinforcement for a given riparian plant assemblage: the age of the trees, or shrubs, and whether or not grasses are perennial or annual. A sigmoidal curve is used in RipRoot to determine where along the growth curve a shrub or tree of a given age is, which then determines the number of roots crossing the shear plane. Trees are assumed to reach a maturing phase at around 30 years of growth, whilst shrubs and perennial grasses are assumed to reach a maturing phase at around five years of growth (Figure 17). For annual grasses the user is prompted to select whether they are modeling a bank during the growing season when annual grass roots will be at their densest (100% of the field measurements taken), or during the dormant season, when a large percentage of annual grass roots have died back (30% of the field measured root densities) (Figure 18).
Figure 17. Cumulative growth curves used in RipRoot for shrubs, perennial grasses and trees.

\[ y = a (1 - e^{-bx})^c \]

Where:
\[ a = 1.097 \times 10^2 \]
\[ b = 1.491 \times 10^{-1} \]
\[ c = 4.365 \]

Figure 18. Growth pattern used in RipRoot for annual grasses to represent growth and die-back of roots over the seasons.

\[ y = a (1 - e^{-bx})^c \]

Where:
\[ a = 1.191 \times 10^2 \]
\[ b = 8.21 \times 10^{-1} \]
\[ c = 10.17 \]
1.2.3 Results of BSTEM runs with and without vegetation

Once tensile strength curves and root-growth curves over time had been entered into RipRoot for the species studied in the Tahoe Basin, the RipRoot algorithm was run to assess potential root-reinforcement over time, and in different bank materials (Table 4). The results showed that root-reinforcement increased over time as a tree matures, as determined by the growth curve applied in the RipRoot algorithm. The magnitude of root-reinforcement did however, vary by species according to variations in the root-tensile strength values measured in the field. Variations also occurred between material types, as the stronger the bank material, the more root breaking dominates over root pullout as a failure mechanism.

In rounded sand pullout dominated over root breaking as a failure mechanism, and root-reinforcement values were relatively low. Variations in root-reinforcement in sand for the tree species studied ranged from 0.01 to 1.67 kPa for lodgepole pine and from 0.01 to 2.27 for the remaining tree species studied. Little difference was seen between the species in the rounded sand scenarios as root pullout is independent of root strength. Greater differences were seen between the species in scenarios for moderate silt and soft clay banks where root breaking became a more important failure mechanism. Of the tree species studied, the Geyer’s willow roots were the strongest and therefore provided the highest modeled root-reinforcement values at each stage of growth. Root-reinforcement values for Geyer’s willows were estimated to reach 2.27, 9.35 and 11.2 kPa of reinforcement in sand, silt and clay respectively after 30 years of growth. When compared to typical values of cohesion for sands, silts and clays (Table 5) these values represent a percentage increase in bank cohesion of 568, 217 and 137 % respectively.
Table 4. Results of RipRoot algorithm, showing root-reinforcement values for common Lake Tahoe riparian species, growing in different streambank materials.

<table>
<thead>
<tr>
<th>Age</th>
<th>Bank Material Type</th>
<th>Root-reinforcement, in kPa</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Rounded Sand</td>
<td>Moderate Silt</td>
</tr>
<tr>
<td>Dry Meadow</td>
<td>Growing season</td>
<td>0.13</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>Dormant</td>
<td>0.03</td>
<td>0.23</td>
</tr>
<tr>
<td>Wet Meadow</td>
<td>Growing season</td>
<td>0.17</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>Dormant</td>
<td>0.04</td>
<td>0.31</td>
</tr>
<tr>
<td>Lodgepole pine</td>
<td>1 year</td>
<td>0.01</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>5 years</td>
<td>0.16</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>10 years</td>
<td>0.84</td>
<td>2.27</td>
</tr>
<tr>
<td></td>
<td>20 years</td>
<td>1.40</td>
<td>3.80</td>
</tr>
<tr>
<td></td>
<td>30 years</td>
<td>1.67</td>
<td>4.54</td>
</tr>
<tr>
<td>Lemmon’s willow</td>
<td>1 year</td>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>5 years</td>
<td>0.21</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>10 years</td>
<td>1.14</td>
<td>2.26</td>
</tr>
<tr>
<td></td>
<td>20 years</td>
<td>1.90</td>
<td>4.64</td>
</tr>
<tr>
<td></td>
<td>30 years</td>
<td>2.27</td>
<td>5.59</td>
</tr>
<tr>
<td>Geyer’s willow</td>
<td>1 year</td>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>5 years</td>
<td>0.21</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>10 years</td>
<td>1.14</td>
<td>4.67</td>
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<tr>
<td></td>
<td>20 years</td>
<td>1.90</td>
<td>7.81</td>
</tr>
<tr>
<td></td>
<td>30 years</td>
<td>2.27</td>
<td>9.35</td>
</tr>
<tr>
<td>Alder</td>
<td>1 year</td>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td>5 years</td>
<td>0.21</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>10 years</td>
<td>1.14</td>
<td>2.13</td>
</tr>
<tr>
<td></td>
<td>20 years</td>
<td>1.90</td>
<td>4.51</td>
</tr>
<tr>
<td></td>
<td>30 years</td>
<td>2.27</td>
<td>5.40</td>
</tr>
</tbody>
</table>

Table 5. Average cohesion values for bank material types, and percentage increase due to root reinforcement of a 30-year old stand of Geyer’s willow trees.

<table>
<thead>
<tr>
<th>Bank Material Type</th>
<th>Average cohesion, c’ in kPa</th>
<th>Percentage increase in bank cohesion from 30 year old Geyer’s willow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rounded sand</td>
<td>0.4</td>
<td>568 %</td>
</tr>
<tr>
<td>Moderate silt</td>
<td>4.3</td>
<td>217 %</td>
</tr>
<tr>
<td>Soft clay</td>
<td>8.2</td>
<td>137 %</td>
</tr>
</tbody>
</table>
2. DEVELOPMENT OF BSTEM-DYNAMIC
2.1 STREAMBANK-EROSION PROCESSES

Conceptual models of bank retreat and the delivery of bank sediments to the flow emphasize the importance of interactions between hydraulic forces acting at the bed and bank toe and gravitational forces acting on in situ bank materials (Carson and Kirkby, 1972; Thorne, 1982; Simon et al., 1991; Langendoen and Simon, 2008). Failure occurs when erosion of the bank toe and the channel bed adjacent to the bank increase the height and angle of the bank to the point that gravitational forces exceed the shear strength of the bank material. Streambank failure can occur by several mechanisms (Figure 1), including cantilever failures of undercut banks, toppling of vertically arranged slabs, rotational slumping, and wedge failures (Thorne et al., 1981). The type of failure reflects the degree of undercutting (if any) by fluvial scour or other mechanisms, and the nature of the bank materials. After failure, failed bank materials may be delivered directly to the flow and deposited as bed material, dispersed as wash load, or deposited along the toe of the bank as intact blocks, or as smaller, dispersed aggregates (Simon et al., 1991).

Figure 19 Types of streambank failures.
2.1.1 History and Structure of the Bank Stability and Toe-Erosion Model (BSTEM)

The original BSTEM model (Simon et al. 1999) allowed for five unique layers, accounted for pore-water pressures on both the saturated and unsaturated parts of the failure plane, and the confining pressure from streamflow. The enhanced ‘static’ BSTEM (Version 5.4) includes a sub-model to predict bank-toe erosion and undercutting by hydraulic shear. This is based on an excess shear-stress approach that is linked to the geotechnical algorithms. Complex geometries resulting from simulated bank-toe are used as the new input geometry for the geotechnical part of the bank-stability model. If a failure is simulated, that new bank geometry can be exported back into either sub-model to simulate conditions over time by running the sub-models iteratively with different flow and water-table conditions. In addition, the enhanced bank-stability sub-model selects between cantilever and planar-failure modes and allows for inclusion of the mechanical, reinforcing effects of riparian vegetation (Simon and Collison, 2002; Micheli and Kirchner, 2002; Pollen and Simon, 2005).

2.1.2 Hydraulic Erosion Sub-Model

Whether sediment is entrained by a moving fluid depends on both the properties of the fluid (i.e. its density, viscosity and velocity) and the physical properties of the sediment, such as its size, shape, density and arrangement (Knighton, 1998). A basic distinction exists between the entrainment of non-cohesive sediment (usually coarse silt, sand, gravel and boulders or cobbles) and cohesive sediments, because the entrainment of the latter is complicated by the presence of cohesion (Knighton, 1998). In both cases, most approaches to sediment transport have relied upon the concept of a critical value of a parameter such as either the depth-averaged or near-bed velocity (Hjulström, 1935), unit streampower (Bagnold, 1966; Yang, 1973) or bed shear stress (e.g. Meyer-Peter and Müller, 1948; Laursen, 1958; van Rijn, 1984a; 1984b; Parker, 1990; Wu et al., 2000 and many others). Herein, we employ the bed shear stress, $\tau_o$, as the independent variable.

Sediment Entrainment and the Shields Curve

Shields (1936) conducted laboratory flume studies examining incipient motion and bed-load transport of non-cohesive, nearly uniform grains. The dimensionless critical shear stress, which appears on the y-axis of the Shields diagram (Figure 2), is defined as:

$$\tau^*_c = \frac{\tau_c}{g(\rho_s - \rho)D_{50}}$$

where $\tau_c$ = critical shear stress (Pa), $g$ = acceleration due to gravity (9.807 m s$^{-2}$), $\rho_s$ = density of sediment (kg m$^{-3}$), $\rho$ = density of water (kg m$^{-3}$) and $D_{50}$ = median diameter of grains in the bed (m). The critical shear stress, $\tau_c$, can be determined from $\tau_c = c_f \rho U^2$, where $c_f$ = a non-dimensional bed roughness coefficient (~0.0044 for sand beds; Hanson and Cook, 1997) and $U$ = flow velocity (m s$^{-1}$). $\tau^*_c$ can be interpreted as the ratio of the average drag force per unit area to
the average gravitational force resisting motion per unit area. The critical roughness Reynolds number, which appears on the x-axis of the Shields diagram, is defined as:

$$Re^*_c = \sqrt{\frac{\tau^*_c(\rho_s - \rho)gD_{50}}{\rho} \frac{D_{50}}{\nu}}$$

where $\nu = \text{kinematic viscosity of water (m}^2\text{s}^{-1})$.

**Figure 20** Shields diagram for incipient motion (modified from Buffington, 1999). The y-axis is defined by equation 1 and the x-axis is defined by equation 2.

The roughness Reynolds number is defined using the shear velocity, $u_c = \sqrt{\tau_o/\rho}$, as the velocity scale and the particle diameter as the length scale. At the onset of motion, $\tau_o \approx \tau_c$ and equation 2 is obtained. $Re^*_c$ can be interpreted as being proportional to the ratio between the particle size and the thickness of the viscous sublayer, and therefore its value indicates the extent to which particles protrude into the turbulent boundary layer.

The Shields diagram clearly shows that finer particles, those with critical roughness Reynolds numbers less than 10, become progressively harder to erode. These roughness Reynolds numbers correspond to clay grade material that exhibit cohesive forces.

**Mechanisms of Cohesive Sediment Erosion**

Mechanistically, the detachment and erosion of cohesive (silt- and clay-sized) material by gravity and/or flowing water is controlled by a variety of physical, electrical, and chemical forces. Identification of all of these forces and the role they play in determining detachment, incipient motion, and erodibility, of cohesive materials is incomplete and still relatively poorly understood (Winterwerp and van Kesteren, 2004). Assessing the erosion resistance of cohesive materials by flowing water is complex due to difficulties in characterizing the strength of the electro-chemical bonds that define the resistance of cohesive materials. The many studies that
have been conducted on cohesive materials have observed that numerous soil properties influence erosion resistance including antecedent moisture, clay mineralogy and proportion, density, soil structure, organic content, as well as pore and water chemistry (Grissinger, 1982). For example, Arulanandan (1975) described how the erodibility of a soil decreases with increasing salt concentration of the eroding fluid, inducing weakening of inter-particle bonds. Kelly and Gularte (1981) showed that for cohesive sediments, increasing temperature increases erosion rates, particularly at low salinity, while at high salinity, there is less of an effect on erosion. Furthermore, studies of streambank stability in cohesive materials (Casagli et al., 1997; Rinaldi and Casagli, 1999; Simon et al., 2000) led to the idea that positive and negative pore-water pressures may play an important role in the entrainment and erosion of cohesive streambed particles or aggregates (Simon and Collison, 2001). Negative pore-water pressures increase the shear strength of unsaturated, cohesive materials by providing tension between particles.

Cohesive materials can be eroded in three contrasting ways (Mehta 1991; Figure 4): (1) surface erosion of bed aggregates; (2) mass erosion of the bed; and (3) entrainment of fluid mud. Partheniades (1965) showed that clay resistance to erosion seemed to be independent of the macroscopic shear strength of the bed, provided that the bed shear stresses did not exceed the macroscopic shear strength of the material. Once the bed shear stress exceeds some critical value, then following Ariathurai and Arulanandan (1978) the rate of erosion, $\varepsilon$, of cohesive materials can be predicted by:

$$\varepsilon = k_d \left( \frac{\tau_o}{\tau_c} - 1 \right)^a$$

$\varepsilon = 0$ (for $\tau_o \leq \tau_c$) \hspace{2cm} -3

where $\varepsilon =$ erosion rate (m s$^{-1}$), $k_d =$ erosion rate coefficient (m s$^{-1}$), $\tau_o =$ bed shear stress (Pa), $\tau_c =$ critical shear stress (Pa), and $a =$ exponent assumed to equal 1.0. Equation 3 may also be written as (Partheniades, 1965):

$$\varepsilon = \frac{k_d}{\tau_c} \left( \tau_o - \tau_c \right) = k \left( \tau_o - \tau_c \right)$$

$\varepsilon = 0$ (for $\tau_o \leq \tau_c$) \hspace{2cm} -4

where $k =$ erodibility coefficient (m$^3$N$^{-1}$s$^{-1}$), representing the volume of material eroded per unit force and per unit time.

A submerged jet-test device has been developed by Hanson (1990) to conduct soil erodibility tests in situ. This device has been developed based on knowledge of the hydraulic characteristics of a submerged jet and the characteristics of soil material erodibility. Utilizing this device, Hanson and Simon (2001) developed the following relation between $\tau_c$ and $k$ for cohesive silts, silt-clays and clays:

$$k = 2 \times 10^{-7} \tau_c^{-0.5}$$

-5
This relation is very similar to observed trends reported by Arulanandan et al. (1980) in laboratory flume testing of streambed material samples from across the United States. Jet-testing conducted on bank toes prior to 2002 suggested that although the exponent is the same, the coefficient was instead $1 \times 10^{-7}$. These relations have been recently updated based on hundreds of tests on streambanks across the United States (Simon et al., 2010):

$$k = 1.62 \times 10^{-6} \tau_c^{-0.838}$$

**Figure 21** Three modes of cohesive sediment erosion: a) surface erosion of bed aggregates; b) mass erosion of the bed; c) entrainment of fluid mud (from Mehta, 1991, fig.1, p.41).

**Predicting the Distribution of Near-Bank Shear Stress**

The magnitude of bank-face, bank-toe and bed erosion and the extent of bank steepening by hydraulic forces are calculated using an algorithm that computes the hydraulic force acting on the near-bank zone during a particular flow event. In the present approach, the boundary shear stress exerted by the flow on each node, $i$, is estimated by dividing the flow area at a cross-section into segments that are affected only by the roughness of the bank or the bed and then further subdividing to determine the flow area affected by the roughness on each node (e.g. Einstein, 1942; Figure 22). The procedure is as follows:

1. Extend a bisector through the base of the bank toe to the water surface at an angle that is the average of the two nodes closest to the base of the bank toe (Figure 22; label 1);
2. Determine the mid-points between nodes on the bank face (Figure 22; label 2);
3. Compute the absolute vertical distance between the mid-points on the bank face and bank toe and compute the total absolute vertical distance encompassed by the mid-points of the bank face and bank toe nodes. Split the water surface between the water-bank intersect...
and the intersect of the line drawn in step 1 into segments with lengths that are proportional to the ratio between the absolute vertical distance between each mid-point and the total absolute vertical distance (Figure 5; label 3); and

4. For each node, \( i \), the hydraulic radius of a segment, \( R_i \), is the area of the flow segment formed (delineated by dashed lines in Figure 5), \( A_i \), divided by the wetted perimeter of the segment. The boundary shear stress active at the node \( i \) may then be estimated as:

\[
\tau_{oi} = \rho g R_i S
\]

where \( S \) is the energy (~bed or water surface) slope (m m\(^{-1}\)).

Flow resistance in an open channel is a result of viscous and pressure drag over its wetted perimeter. For a vegetated channel, this drag may be conceptually divided into three components: 1) the sum of viscous drag on the ground surface and pressure drag on particles or aggregates small enough to be individually moved by the flow (grain roughness); 2) pressure drag associated with large non-vegetal boundary roughness (form roughness); and 3) drag on vegetal elements (vegetal roughness) (Temple et al., 1987). As energy lost to the flow represents work done by a force acting on the moving water, the total boundary shear stress may also be divided into three components:
\[ \tau_o = \tau_{og} + \tau_{of} + \tau_{ov} \]  

where the subscripts \( g, f \) and \( v \) signify the grain, form and vegetal components of the boundary shear stress, respectively.

If it is assumed that these components may be expressed in terms of a Manning’s coefficient for each, and Manning’s equation is assumed to apply for each component, equation 8 can be rewritten as (Temple, 1980):

\[ n^2 = n_g^2 + n_f^2 + n_v^2 \]  

where \( n \) = Manning’s roughness coefficient (s m\(^{-1/3}\)). Grain roughness is estimated for each node on the bank profile using the equation of Strickler (Chow, 1959):

\[ n_g = 0.045 \left( \frac{D_{50}}{16} \right) \]  

Combining equations 8 and 9, the effective boundary shear stress, the component of the boundary shear stress acting on the boundary in the absence of form and vegetal roughness, may be computed as:

\[ \tau_g = \tau_o \left( \frac{n_g^2}{n^2} \right) \]  

The rate of erosion of bank-face, bank-toe and bed materials can then be calculated using equations 4 and 11 (Hanson, 1990). During the dynamic simulations described herein, the horizontal erosion distance during a timestep is computed by integrating the erosion rate within the timestep by the timestep size:

\[ E = \varepsilon \Delta t \]  

where \( E \) = erosion distance (m), and \( \Delta t \) = timestep (s).

### 2.1.3 Geotechnical/Mass Failure Sub-Model

The model simulates failure type 1b, planar failure in short, steep banks, and 1c, shear failure in banks that have been undercut by preferential erosion of an erodible basal layer (Figure 1). These are shear-type failures that occur when the driving force (stress) exceeds the resisting force (strength). The model combines two limit-equilibrium methods that estimate the Factor of Safety \( (F_s) \) of multi-layer streambanks. \( F_s \) is the ratio between the resisting and driving forces acting on a potential failure block. A value of unity indicates that the driving forces are equal to the resisting forces and that failure is imminent \( (F_s = 1) \). Instability exists under any condition where the driving forces exceed the resisting forces \( (F_s < 1) \), conditional stability is indicated by \( F_s \) values between 1 and 1.3, with stable bank conditions having a \( F_s \) value of >1.3.
Quantifying the Resisting and Driving Geotechnical Forces

Soil shear strength varies with the moisture content of the bank and the elevation of the saturated zone in the bank mass. In the part of the streambank above the “normal” level of the groundwater table, bank materials are unsaturated, pores are filled with both water and air, and pore-water pressure is negative. The difference \( (\mu_a - \mu_w) \) between the air pressure, \( \mu_a \), and the water pressure in the pores, \( \mu_w \), represents matric suction. The increase in shear strength due to an increase in matric suction \( (\mu_a - \mu_w) \) is described by the angle \( \phi' \). \( \phi' \) varies for all soils and with moisture content for a given soil (Fredlund and Rahardjo, 1993), but generally takes a value between 10° and 20°, with a maximum of the effective soil friction angle, \( \phi' \), under saturated conditions (Fredlund and Rahardjo, 1993). The effect of matric suction on shear strength is reflected in the apparent cohesion \( c_a \) term, which incorporates both electro-chemical bonding within the soil matrix (described by the effective cohesion, \( c' \)) and cohesion due to surface tension on the air-water interface of the unsaturated soil:

\[
c_a = c' + (\mu_a - \mu_w) \tan \phi'
\]

where \( c_a \) = apparent cohesion (kPa), \( c' \) = effective cohesion (kPa), \( \mu_a \) = pore-air pressure (kPa), \( \mu_w \) = pore-water pressure, \( (\mu_a - \mu_w) \) = matric suction (kPa) and \( \phi' \) is the angle describing the increase in shear strength due to an increase in matric suction (degrees).

As can be seen from equation 1, negative pore-water pressures (positive matric suction) in the unsaturated zone provide for cohesion greater than the effective cohesion, and thus, greater shearing resistance. This is often manifested in steeper bank slopes than would be indicated by \( \phi' \). Conversely, the wetter the bank and the higher the water table, the weaker the bank mass becomes and the more prone it is to failure. Accounting for the effects of friction, the shear strength of a soil, \( \tau_s \), may thus be described by the Mohr-Coulomb shear strength criterion for unsaturated soils (Fredlund et al., 1978):

\[
\tau_s = \frac{1}{F_s} \left[c' + (\mu_a - \mu_w) \tan \phi' + (\sigma - \mu_a) \tan \phi' \right]
\]

where \( F_s \) = Factor of Safety, the ratio between the resisting and driving forces acting on a potential failure block, \( \sigma \) = normal stress on the shear plane (kPa) and \( \phi' \) = effective angle of internal friction (degrees).

Whilst it is assumed that the pore-air pressure is atmospheric (i.e. \( \mu_a = 0 \)), positive and negative pore-water pressures are calculated for the mid-point of each layer based on hydrostatic pressure above and below the water table so that:

\[
\mu_w = \gamma_w h
\]

where \( \mu_w \) = pore-water pressure (kPa), \( \gamma_w \) = unit weight of water (9.807 kN m\(^{-3}\)) and \( h \) = head of water above the mid-point of the layer (m).
The geotechnical driving forces are controlled by bank height and slope, the unit weight of the soil and the mass of water within it, and the surcharge imposed by any objects on the bank top.

### 2.1.4 Root Reinforcement Sub-Model (Riproot)

Soil is generally strong in compression, but weak in tension. The fibrous roots of trees and herbaceous species are strong in tension but weak in compression. Root-permeated soil, therefore, makes up a composite material that has enhanced strength (Thorne, 1990). Numerous authors have quantified this reinforcement using a mixture of field and laboratory experiments. Endo and Tsuruta (1969) used *in situ* shear boxes to measure the strength difference between soil and soil with roots. Gray and Leiser (1982) and Wu (1984) used laboratory-grown plants and quantified root strength in large shear boxes.

Many studies have found an inverse power relationship between ultimate tensile stress, $T_r$, and root diameter, $d$ (examples include but are not limited to: Waldron and Dakessian, 1981; Riestenberg and Sovonick-Dunford, 1983; Coppin and Richards, 1990; Gray and Sotir, 1996; Abernethy and Rutherfurd, 2001; Simon and Collison, 2002; Pollen and Simon, 2005; Fan and Su, 2008):

$$T_r = e(1000d)^f$$

(16)

where $e = \text{multiplier (MPa m}^{-f})$, and $f = \text{exponent (dimensionless) in the root tensile stress-diameter function, respectively. Note that $f$ is always negative. Root tensile strength (in kN) can therefore be evaluated as the product of the root area, $A_r (\pi d^2/4)$, and the ultimate tensile stress, $T_r$:}$

$$T_r A_r = \frac{e\pi(1000)^{1+f}d^{2+f}}{4}$$

(17)

Smaller roots are stronger per unit area (higher ultimate tensile stress), but the larger cross-sectional area of larger diameter roots means that the peak load they can withstand before breaking is higher than that of small roots.

Wu et al. (1979, after Waldron, 1977) developed a widely-used equation that estimates the increase in soil strength ($c_r$) as a function of root tensile strength, areal density and root distortion during shear:

$$c_r = \frac{1}{A} \sum_{i=1}^{i=I} (A_i T_i) [\sin(90-\zeta) + \cos(90-\zeta)\tan\phi']$$

(18)

where $c_r = \text{cohesion due to roots (kPa)}$, $T_i = \text{tensile strength of roots (kPa)}$, $A_i = \text{area of roots in the plane of the shear surface (m}^2)$, $A = \text{area of the shear surface (m}^2)$, $I = \text{total number of roots crossing the shear plane, the subscript } i = i^{th} \text{ root, and}$
\[
\zeta = \tan^{-1}\left(\frac{1}{\tan \theta + \cot \chi}\right)
\]

(19)

where \( \theta \) = angle of shear distortion (degrees), and \( \chi \) = initial orientation angle of fiber relative to the failure plane (degrees).

Pollen et al. (2004) and Pollen and Simon (2005) found that models based on equation 18 tend to overestimate root reinforcement because it is assumed that the full tensile strength of each root is mobilized during soil shearing and that the roots all break simultaneously. This overestimation was largely corrected by Pollen and Simon (2005) by developing a fiber-bundle model (RipRoot) to account for progressive breaking during mass failure.

Fiber-bundle models (FBMs) have been widely used in the materials industry to aid in the understanding of composite materials (starting with the work of Daniels, 1945). They are easy to parameterize and incorporate the most important aspects of soil-root interactions, using a dynamic approach to remove the assumption that all of the roots in the soil matrix break simultaneously. When a load is applied to the bundle of fibers it is apportioned equally between all intact fibers (Daniels, 1945). The maximum load that can be supported by the bundle corresponds not to the weakest or strongest fiber, but to one of the fibers in the middle.

FBMs work by apportioning the total load applied to a bundle of \( N \) parallel fibers (roots) and then monitoring whether the load applied to the \( n \)th fiber exceeds its strength. The governing equation of a fiber-bundle model can therefore be written as:

\[
\text{Load to Break } n^{\text{th}} \text{ Fiber} = \frac{\text{Total Applied Load}}{\text{Number of Intact Fibers, } N}
\]

(20)

The term “break” does not differentiate between failure modes. Once the load has increased sufficiently for a fiber to break, the load that was carried by the broken fiber is redistributed equally amongst the remaining \((N-1)\) intact roots, each of which then bears a larger load, and is hence more likely to break. If this redistribution causes further roots to break, additional redistribution of load occurs until no more breakages occur (in this type of model this is known as an avalanche effect). Another increment of load is then added to the system, and the process is repeated until either all of the fibers have broken, or the maximum driving force acting on the matrix is supported by the fibers contained within it.

RipRoot was validated by comparing results of root-permeated and non-root-permeated direct-shear tests. These tests revealed that, relative to results obtained with the perpendicular model of Wu et al. (1979), accuracy was improved by an order of magnitude, but some error still existed (Pollen and Simon, 2005). One explanation for the remaining error in root-reinforcement estimates lies in the fact that observations of incised streambanks suggest that when a root-reinforced soil shears, two mechanisms of root failure occur: root rupture and root pullout. The anchorage of individual leek roots was studied by Ennos (1990), who developed a function for pullout forces based on the strength of the bonds between the roots and soil:

\[
F_p = \pi d \tau_s L_r
\]

(21)
where $F_p =$ pullout force for an individual root (N), and $L_r =$ root length (m), which can be estimated in the absence of field data using $L_r = 123.1 d^{0.7}$ (Pollen, 2007).

The pullout force was not accounted for in the original version of RipRoot (Pollen and Simon, 2005) and so the role played by spatio-temporal variations in soil shear strength was neglected. Pollen (2007) tested the appropriateness of equation 21 through field measurements of the forces required to pull out roots. Pullout forces were then compared with breaking forces obtained from tensile strength testing and the RipRoot model was modified to account for both breaking and pullout. Thomas and Pollen-Bankhead (2010) improved equation 21 by employing Rankine’s active earth pressure theory to compute $\tau_s$. See Terzaghi and Peck (1967) for a description of Rankine’s active earth pressure theory.

A second explanation is that, following the work of Wu et al. (1979), it has commonly been assumed that the $\sin(90 - \zeta) + \cos(90 - \zeta) \tan \phi'$ term in equation 18 takes an approximately constant value of 1.2. Sensitivity analysis indicates that this assumption is flawed as this term varies from -1 when $\zeta = 60^\circ$ to a maximum as $\zeta \rightarrow \phi$ (Thomas and Pollen-Bankhead, 2010). A series of Monte Carlo simulations was undertaken, assuming that $\theta$ was uniformly distributed between $0^\circ$ and $90^\circ$ and assuming that $\chi$ was uniformly distributed between $\pm 90^\circ$ from the vertical, approximating a heartroot network. Friction angle was varied from $0^\circ$ to $44^\circ$ and failure plane angle was varied from $10^\circ$ to $90^\circ$. For this assumed distribution, the $\sin(90 - \zeta) + \cos(90 - \zeta) \tan \phi'$ term was found to be independent of failure plane angle. In addition, for a given friction angle, the distribution of values was highly skewed, with the median and 84th percentile being approximately equal but the 4th percentile being much smaller (Thomas and Pollen-Bankhead, 2010). We found during tests that it was possible to predict the median value of the $\sin(90 - \zeta) + \cos(90 - \zeta) \tan \phi'$ term using a cubic polynomial involving only the friction angle and this has been implemented herein.

The combination of the fiber bundle approach, in which roots break progressively during failure, the incorporation of pullout forces that vary as a function of the shear strength of the soil surrounding each root, and the variability in root orientation caused by local factors (e.g., water and nutrient availability, substrate and topographic variability) ensure that predictions of $c_r$ cannot be readily extrapolated from one areal density to another nor from site to site.

### 2.1.5 Assessing the Potential for Geotechnical Failure

The methods employed within BSTEM are horizontal layers (Simon et al., 2000) and cantilever failures (Thorne and Tovey, 1981). Both methods account for the strength of multiple soil layers, the effect of pore-water pressure (both positive and negative (matric suction)), and the confining pressure due to streamflow.

**Horizontal Layer Method**

The Horizontal Layer method is a further development of the wedge failure type developed by Simon and Curini (1998) and Simon et al. (2000), which in turn is a refinement of the models developed by Osman and Thorne (1988) and Simon et al. (1991). The Factor of Safety ($F_s$) is given by:
\[
F_s = \frac{\sum_{i=1}^{I} \left[ (c_i' + c_r)L_i + (\mu_a - \mu_w)L_i \tan \phi_i^b + [W_i \cos \beta - \mu_{al}L_i + P_i \cos(\alpha - \beta)] \tan \phi_i \right]}{\sum_{i=1}^{I} (W_i \sin \beta - P_i \sin(\alpha - \beta))}
\]

where \(c_i'\) = effective cohesion of \(i^{th}\) layer (kPa), \(L_i\) = length of the failure plane incorporated within the \(i^{th}\) layer (m), \(W_i\) = weight of the \(i^{th}\) layer per unit length of stream channel (kN m\(^{-1}\)), \(P_i\) = hydrostatic-confining force due to the external water level (kN m\(^{-1}\)) acting on the \(i^{th}\) layer, \(\beta\) = failure-plane angle (degrees from horizontal), \(\alpha\) = local bank angle (degrees from horizontal), and \(I\) = number of layers. The hydrostatic confining force, \(P_i\), is calculated from the area of the confining pressure (\(\gamma_w h\)) by:

\[
P_i = \frac{\gamma_w h^2}{2}
\]

where \(h\) = head of water in the channel (m). The loss of the hydrostatic-confining force is the primary reason bank failures often occur after the peak flow and on the recessional limb of hydrographs.

**Cantilever Shear Failure Algorithm**

The cantilever shear failure algorithm results from inserting \(\beta = 90^\circ\) into equation 22 and simplifying. \(F_s\) is given by:

\[
F_s = \frac{\sum_{i=1}^{I} \left[ (c_i' + c_r)L_i + (\mu_a - \mu_w)L_i \tan \phi_i^b + [P_i \cos \alpha - \mu_{al}L_i] \tan \phi_i \right]}{\sum_{i=1}^{I} (W_i + P_i \cos \alpha)}
\]

Put simply, the \(F_s\) is the ratio of the shear strength of the soil to the weight of the cantilever. The inclusion of \(\alpha\)-terms in equation 24 ensures that if the bank is partially or totally submerged the weights of the layers affected by water are correctly reduced irrespective of the geometry of the basal surface of the overhang.

**Locating the failure plane that minimizes \(F_s\)**

A minimum can be either global (truly the lowest function value) or local (the lowest in a finite neighborhood and not on the boundary of that neighborhood). Finding the global minimum is, in general, a very difficult problem (Press et al., 1992). Herein, we adopt one of the standard heuristics: at a user-defined number of failure base elevations, we isolate the failure plane angle that produces the minimum factor of safety. Once all the potential failure base locations have been searched, we select the minimum of all the local minima. This reduces our problem to a series of one-dimensional minimization problems. We follow the recommendation of Press et al. (1992): “For one-dimensional minimization (minimize a function of one variable) without calculation of the derivative, bracket the minimum… and then use Brent’s method…” If your
function has a discontinuous second (or lower) derivative, then the parabolic interpolations of Brent’s method are of no advantage, and you might wish to use the simplest form of golden section search.” For more details of the routine and its implementation, the interested reader is referred to Press et al. (1992) §10.2.

2.1.6 Modeling Movement of the Groundwater Table

It is apparent from equations 13, 14, and 15 that the elevation of the groundwater table is an important parameter controlling soil shear strength. For the purposes of this study, a simplified one-dimensional (1-D) groundwater model, based on the 1-D Richards Equation, was developed to simulate the motion of the groundwater table. This model assumes that the dominant pressure gradient within a streambank is the difference between the groundwater table elevation and the in-channel water surface elevation (i.e., it neglects the influence of infiltrating precipitation) (e.g. Langendoen, 2010). Assuming that water infiltrates either into or out of the bank along a horizontal plane of unit length and computing distance-weighted mean soil properties between these two elevations, the simplified equation can be written as:

\[
\frac{\partial h}{\partial t} - K_r K_{sat} |h - z|^2 = 0
\]

where \( h \) = groundwater elevation (m), \( z \) is the water surface elevation (m), \( t \) = time (s), and \( K_r K_{sat} \) = relative permeability \( \times \) saturated hydraulic conductivity. \( K_r \) is evaluated as \( K_r = \Theta^{1/2} \left[ 1 - \left( 1 - \Theta^{1/n} \right)^2 \right] \), where \( \Theta \) = soil saturation and, following van Genuchten (1980), \( \Theta \) is evaluated as:

\[
\Theta = \Theta_r + \frac{\Theta_s - \Theta_r}{1 + \left( \frac{|z - h|}{\alpha} \right)^{1-n}}
\]

where the subscripts \( r \) and \( s \) denote the residual moisture content and saturated moisture content (= porosity), and \( \alpha \) and \( n \) are curve-fitting parameters defined by van Genuchten (1980). Note that if \( h \geq z \), \( K_r = 1 \).
2.1.7 Data Requirements

The data required to operate the model are all related to quantifying the driving and resisting forces that control the hydraulic and geotechnical processes that operate on a streambank. Input parameters can all be obtained directly from field surveying and testing. If this is not possible, the model provides default values by material type for many parameters. It has been our experience that all of the data can be collected at a site by a crew of four within one day. Required data fall into three broad categories: (1) bank geometry and stratigraphy, (2) hydraulic data, and (3) geotechnical data. A summary of the required input parameters is provided in Table 6. The default geotechnical values that are included in the model are provided in Table 7.
Table 6. Required input parameters for BSTEM.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Purpose</th>
<th>Source</th>
<th>Parameter</th>
<th>Purpose</th>
<th>Source</th>
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<tr>
<td><strong>Driving Forces</strong></td>
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<td><strong>Resisting Forces</strong></td>
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</tr>
<tr>
<td><strong>Hydraulic Processes: Bank Surface</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Channel slope ($S$)</td>
<td>Boundary shear stress ($\tau_o$)</td>
<td>Field survey or design plan</td>
<td>Particle diameter ($D$) (cohesionless)</td>
<td>Critical shear stress ($\tau_c$)</td>
<td>Sample; Model defaults</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>Critical shear stress ($\tau_c$) (cohesive)</td>
<td></td>
<td>Jet test; Model defaults</td>
</tr>
<tr>
<td>Water surface elevation</td>
<td>Boundary shear stress ($\tau_o$)</td>
<td>Field survey, gage information, design plan</td>
<td>Particle diameter ($D$) (cohesionless)</td>
<td>Erodibility coefficient ($k$)</td>
<td>Sample; Model defaults</td>
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<tr>
<td></td>
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<td></td>
<td>Critical shear stress ($\tau_c$) (cohesive)</td>
<td></td>
<td>Jet test; Model defaults</td>
</tr>
<tr>
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<td>Boundary shear stress ($\tau_o$)</td>
<td>Gage information, Chow (1959), Barnes (1967)</td>
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<tr>
<td><strong>Geotechnical Processes: Bank Mass</strong></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit weight of sediment ($\gamma_s$)</td>
<td>Weight ($W$), Normal force ($\sigma$)</td>
<td>Sample; Model defaults</td>
<td>Unit weight of sediment ($\gamma_s$)</td>
<td>Weight ($W$), Normal force ($\sigma$)</td>
<td>Sample; Model defaults</td>
</tr>
<tr>
<td>Bank height ($H$)</td>
<td>Weight ($W$), Normal force ($\sigma$)</td>
<td>Field survey or design plan</td>
<td>Bank height ($H$)</td>
<td>Field survey or design plan</td>
<td></td>
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<td></td>
<td></td>
<td>Bank angle ($\alpha$)</td>
<td>Field survey or design plan</td>
<td></td>
</tr>
<tr>
<td>Initial groundwater elevation</td>
<td>Weight ($W$), Normal force ($\sigma$)</td>
<td>Field observation; Model defaults</td>
<td>Initial groundwater elevation</td>
<td>Weight ($W$), Normal force ($\sigma$), Shear strength ($\tau_c$)</td>
<td>Field observation; Model defaults</td>
</tr>
<tr>
<td>Water surface elevation</td>
<td>Hydrostatic confining force</td>
<td>Field survey, gage information, design plan</td>
<td>Water surface elevation</td>
<td>Hydrostatic confining force</td>
<td>Field survey, gage information, design plan</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Effective cohesion ($c'$)</td>
<td></td>
<td>Borehole, direct, or triaxial shear test; Model defaults</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Effective friction angle ($\phi'$)</td>
<td></td>
<td>Model defaults</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Angle - increase in $\tau_c$ for increase in matric suction ($\phi'$)</td>
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<td></td>
</tr>
</tbody>
</table>


| Driving Forces | Parameter | Purpose | Source | | Resisting Forces | Parameter | Purpose | Source |
|----------------|-----------|---------|--------|----------------|-----------|---------|--------|
| Vadose Zone Processes: Bank Mass | Initial groundwater elevation | Compute head driving groundwater motion | Field observation; Model defaults | Saturated Hydraulic Conductivity \( k_{sat} \) | Falling head permeameter test; Model defaults |
| | Water surface elevation | Field survey, gage information, design plan | | Porosity \( \Theta_s \) | Sample; Model defaults |
| | | | | Residual water content \( \Theta_r \) | Sample; Model defaults |
| | | | | van Genuchten \( \alpha \) | Falling head permeameter test; Pedotransfer function; Model defaults |
| | | | | van Genuchten \( n \) | Falling head permeameter test; Pedotransfer function; Model defaults |
| | | | | | | | |
| Mass balance accounting | Chemical concentration in each layer and toe [optional] | | Sample |
| | Grain size distribution of each layer and toe | | Sample; Model defaults |
| | Unit weight of sediment of each layer and toe \( \gamma_s \) | | Sample; Model defaults |
| | | | | | | | |
| Model accuracy and stability parameters | Number of failure base elevations used to find the minimum factor of safety | Increase accuracy of location of failure base; reduce simulation time |
| | Number of mass failures permitted to occur in each time step | Increase accuracy of time of failure; reduce simulation time |
| | Threshold factor of safety at which mass failure occurs | Add safety margin when classifying banks as stable. Values between 1.0 and 1.5 are permitted. |
| | Number of sub-time steps executed by the groundwater sub-component per model time step | Increase accuracy and stability of groundwater sub-model calculations; reduce simulation time |
| | Number of sub-time steps executed by the hydraulic scour sub-component per model time step | Increase accuracy and stability of hydraulic scour sub-model calculations; reduce simulation time |
Table 7. Default values in BSTEM (bold) for geotechnical properties. Data derived from more than 800 in situ direct-shear tests with the Iowa Borehole Shear Tester except where indicated.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Statistic</th>
<th>(c') (kPa)</th>
<th>(\phi') (degrees)</th>
<th>(\gamma_{\text{sat}}) (kN/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel (uniform)*</td>
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<td>36.0</td>
<td>20.0</td>
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<tr>
<td>Sand and Gravel*</td>
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<tr>
<td>Sand</td>
<td>75th percentile</td>
<td>1.0</td>
<td>32.3</td>
<td>19.1</td>
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<tr>
<td></td>
<td>Median</td>
<td><strong>0.4</strong></td>
<td><strong>30.3</strong></td>
<td><strong>18.5</strong></td>
</tr>
<tr>
<td></td>
<td>25th percentile</td>
<td>0.0</td>
<td>25.7</td>
<td>17.9</td>
</tr>
<tr>
<td>Loam</td>
<td>75th percentile</td>
<td>8.3</td>
<td>29.9</td>
<td>19.2</td>
</tr>
<tr>
<td></td>
<td>Median</td>
<td><strong>4.3</strong></td>
<td><strong>26.6</strong></td>
<td><strong>18.0</strong></td>
</tr>
<tr>
<td></td>
<td>25th percentile</td>
<td>2.2</td>
<td>16.7</td>
<td>17.4</td>
</tr>
<tr>
<td>Clay</td>
<td>75th percentile</td>
<td>12.6</td>
<td>26.4</td>
<td>18.3</td>
</tr>
<tr>
<td></td>
<td>Median</td>
<td><strong>8.2</strong></td>
<td><strong>21.1</strong></td>
<td><strong>17.7</strong></td>
</tr>
<tr>
<td></td>
<td>25th percentile</td>
<td>3.7</td>
<td>11.4</td>
<td>16.9</td>
</tr>
</tbody>
</table>

* Data from Hoek and Bray (1977)
2.2 VALIDATION OF THE NEAR-BANK GROUNDWATER MODEL

2.2.1 Methodology

To validate the near-bank groundwater model added to BSTEM-Dynamic, model runs were conducted for one site on Trout Creek (TROUT1), and one site on the Upper Truckee River (UTR3). Groundwater data were obtained from nests of tensiometers installed at the Trout1 and UTR3 monitoring sites from 10/1/2007 to 8/25/2009. Tensiometers were installed at two depths (0.3 and 1.0 m) near one and three meters back from the bank edge (Figure 23). The tensiometer data (measured in kPa; Figure 24) was converted to pressure head to give the groundwater elevation during the monitored period at each site. Monitored data collected between 10/1/2007 and 10/30/2008 was selected to model in BSTEM so that monitored and modeled groundwater values could be compared. Flow data for this period was obtained from surface water transducers also collecting data at the two sites during this period.

Figure 23. Monitoring site at Trout 1, showing (left) tensiometer locations and logger housing, and (right) the pressure transducer set up to measure water surface elevation.
2.2.2 Results

During the monitored period no bank failures were recorded at either of the sites, but some hydraulic scour did occur. The BSTEM runs for the monitored time period also showed no bank failure events, but some hydraulic scour in the bank toe region at each site (figure 25; Figure 26), therefore confirming the dominant processes occurring during this time period. The groundwater model outputs from BSTEM at the two tested sites, are shown in Figure 27 Figure 28, with the monitored data. At the site on Trout Creek the groundwater model performed very well against monitored data until the onset of snowmelt in the spring of 2008. The discrepancy in monitored and modeled groundwater values during this period results from the fact that the near-bank groundwater model is linked to within-channel stage, and cannot account for infiltration of surface water entering the bank from precipitation or snowmelt. By late May 2008, the groundwater at the Trout Creek site had once again become more linked to flow stage, than infiltration from snowmelt, and the model performed well for the remainder of the monitoring period. During the summer months, however, the modeled groundwater elevation did remain approximately 10 cm higher than the monitored groundwater elevation; this may have been a result of the model’s inability to account for evapotranspiration as an additional process capable of removing water from the bank and lowering the water table.

Similar responses were seen in the results for the UTR3 site, with the modeled and monitored groundwater generally matching very well until spring snowmelt in 2008. In early summer the effect of evapotranspiration had less of an effect on the monitored groundwater level at this site than at the Trout 1 site, most likely because the Upper Truckee River streambank is taller than the bank at the Trout Creek site. Removal of surface moisture by evapotranspiration therefore had less of an effect on groundwater deeper within the bank, early in the summer. This resulted in the monitored and modeled groundwater matching closely at this site after snowmelt had finished. Later in the summer months of 2008 there was approximately a 10 cm discrepancy in water table heights, with monitored values being lower than modeled values. This may have occurred because plants had to remove water from greater soil depths as the summer months progressed, with the effect of evapotranspiration therefore not being seen until later in the summer at this site.
Figure 24. Tensiometer and precipitation data from Trout1 (above) and UTR3 (below) monitoring sites.
Figure 25. Before and after profile from groundwater validation run at the Trout 1 site.

Figure 26. Before and after profile from groundwater validation run at the UTR3 site.
**Figure 27.** Comparison of groundwater output from BSTEM compared to monitored groundwater data over the same time period at the Trout 1 site.

**Figure 28.** Comparison of groundwater output from BSTEM compared to monitored groundwater data over the same time period at the UTR3 site.
3. EFFECTIVENESS OF EROSION CONTROL STRATEGIES
3.1 BANK-STABILITY MODELING FOR STREAM RESTORATION

Bank-stability modeling is an important, if not critical component of stream-restoration or erosion-control activities that pertain to excess sediment loads or potential risk of adjacent lands and infrastructure. There are at least three restoration objectives that can benefit greatly from the use of a mechanistic tool to reliably predict sediment loadings and widening rates from streambank erosion. These include:

1. Determining bank-stability conditions under a range of hydraulic and geotechnical conditions and erosion-control strategies. This includes designing
   a. Sustainable bank-stabilization measures, and
   b. Determining unstable bank conditions to assure continued delivery of sediment to the channel (in cases where there is insufficient supply; i.e. Wyzga et al., this volume)
2. Quantifying bank widening rates and sediment loads emanating from streambanks, and
3. Determining potential reductions in widening rates and sediment-loads under a range of mitigation techniques.

Any restoration objective that requires reduction of sediment loads from streambanks must focus on mitigation measures that directly affect the processes that control streambank stability, namely hydraulic erosion and geotechnical instability (Table 8). Protection from hydraulic processes must either reduce the available boundary shear stress, and/or increase the shear resistance to particle detachment, thereby reducing the likelihood and magnitude of bank-toe steepening. Protection from geotechnical instability must focus on increasing soil shear strength and/or decreasing the driving (gravitational) forces to reduce the likelihood of mass failure of the upper bank.

Implementation of any design plan requires the analysis of the hydraulic and geotechnical processes likely to exist at the site, particularly during worst-case conditions. For hydraulic processes, these occur at peak flows when boundary shear stresses are greatest. For geotechnical processes, these generally occur during a wet period and following recession of peak stage when pore-water pressures in the bank are at a maximum and the confining pressure provided by the flow on the bank has been lost. This is referred to as the “drawdown” condition.

Table 8. Potential alternative means to control the two primary processes that control streambank stability.

<table>
<thead>
<tr>
<th>Hydraulic protection</th>
<th>Geotechnical protection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Increase critical shear stress: Bank-toe and face armoring with rock, large wood, live vegetation;</td>
<td>Increase bank shear strength: pole and post plantings, bank top vegetation, brush layers, drainage</td>
</tr>
<tr>
<td>Decrease applied shear stress: redirect flows, reduce channel slope (re-meandering), increase bottom width, live vegetation (increased roughness)</td>
<td>Decrease driving, gravitational forces: reduce bank height, terraces, flatten bank slope; buttress bank toe</td>
</tr>
</tbody>
</table>
3.1.1 Methodology

Initial BSTEM runs were carried out to investigate bank erosion volumes at three Upper Truckee sites (UTR3, UTR54 and UTR Hole 6) and two Trout Creek sites (TROUT1 and TROUT4), under existing bank geometry and vegetation conditions. The model was run for a period of 13 months of flow data. The 99th percentile flow year (1995) was calculated from mean daily values obtained from USGS gage data (TROUT1: Pioneer Trail USGS gage 10336775; TROUT4: Martin Ave USGS gage 10336780; UTR3 and UTR54 USGS gage 10336610; UTR Hole 6 USGS gage 103366092). The month of January 1997 was also added to the flow series as a significant rain on snow event occurred during this month (Figure 31). This thirteen month data series therefore represented the upper limit of potential erosion volumes over an annual hydrograph.

Site specific bank surveys were input to BSTEM along with bank material properties (effective cohesion, friction angle, bulk unit weight, critical shear stress) measured in the field at each site. The resistance of the bank toe and bank face materials to erosion by hydraulic scour was measured at each site using the jet-test device previously discussed in the section on failed blocks. Effective cohesion and friction angle values for each layer in each streambank modeled, were collected using a bore-hole shear tester.

![Field photos showing (left) the BST device and (right) jet test devices being used on Trout Creek.](image)

**Figure 29.** Field photos showing (left) the BST device and (right) jet test devices being used on Trout Creek.
The borehole shear tester (Lohnes and Handy 1968; Lutenegger and Hallberg 1981; Little et al. 1982; Figure 30) was used to measure drained, direct-shear geotechnical parameters (apparent cohesion and angle of internal friction). Advantages of the instrument include:

1. The test is performed *in situ* and testing is, therefore, performed on undisturbed material;
2. Cohesion and friction angle are evaluated separately with the cohesion value representing apparent cohesion ($c_a$). Effective cohesion ($c'$) is then obtained by adjusting $c_a$ according to measured pore-water pressure and $\phi'$;
3. A number of separate trials are run at the same sample depth to produce single values of cohesion and friction angle based on a standard Mohr-Coulomb failure envelope;
4. Data and results obtained from the instrument are plotted and calculated on site, allowing for repetition if results are unreasonable; and
5. Tests can be carried out at various depths in the bank to locate weak strata (Thorne et al. 1981). under existing bank conditions.

A 7 cm-diameter hole was bored to slightly deeper than the desired depth in the streambank. The shear head was placed in the borehole to the desired depth and expanded out under a known initial pressure (generally about 40 kPa) to the walls of the borehole. Depending on the properties of the material, an initial consolidation time of either 60 or 90 minutes was used. An axial stress was then applied and measured on the shearing gauge until failure beyond the walls of the borehole occurred. The axial stress was released, the normal pressure was raised in increments of about 10 kPa, an additional 30 minutes of consolidation was provided and the axial stress was applied again. In this way a series of data points were obtained providing the shear stress required to fail the material for each associated normal stress that was applied to the walls of the borehole. A linear regression between shear stress (y-axis) and normal stress (x-axis) then provided apparent cohesion (y-intercept) and friction angle (slope of the regression line).

![Schematic representation of borehole shear tester (BST) used to determine cohesive and frictional strengths of *in situ* bank materials. Modified from Thorne et al. (1981).](image-url)
Figure 31. Flow stage data examples for the UTR3 site and the Trout1 site.
Apparent cohesion includes both the inherent (effective) cohesion due to the soil skeleton and any additional cohesion provided by matric suction. To obtain values of effective cohesion, coincident measurements of pore-water pressures were required for each BST test. Positive or negative (matric suction or tension) pore-water pressure was measured with a miniature, digital tensiometer. A 5 mm hole was bored or drilled into the top of the core, the tensiometer was immediately placed in the hole and readings were recorded every 15 seconds for a minimum of 6 minutes. If the readings had not stabilized during the first 6 minutes, additional readings were taken until the readings equilibrated. When the time to reach equilibrium was especially long (e.g. in tight clays), a power regression function was applied to obtain the asymptotic, equilibrium value of pore-water pressure.

Root-reinforcement values for dry meadow grasses were applied to sites along the Upper Truckee, and values for wet meadow grasses applied to the sites along Trout Creek. Vegetated blocks lining the bank toe on the outer bend of the UTR3 site were also accounted for by increasing the critical shear stress of the toe material by an order of magnitude to account for the protection afforded by these rooted blocks of failed material. Within the model output, volumes of erosion resulting from geotechnical failures and from hydraulic erosion were separated out so dominant processes could be assessed (Table 9).

On the following pages, the input geometry, bank material properties and bank layering are provided for each of the five sites modeled (Figure 32 to Figure 36).
Figure 32. Bank geometry, layer depths and bank material properties for Trout 1 site.
Figure 33. Bank geometry, layer depths and bank material properties for Trout 4 site.
Figure 34. Bank geometry, layer depths and bank material properties for UTR Hole 6 site.
Figure 35. Bank geometry, layer depths and bank material properties for UTR 54 site.
Figure 36. Bank geometry, layer depths and bank material properties for UTR3 site.
3.1.2 Model Runs to Explore Current Loading Rates And Potential Mitigation Techniques

Under existing conditions, volumes of geotechnical erosion ranged from 0.584 to 4.834 m$^3$ per m of bank at the Upper Truckee sites modeled, and were 0.005 m$^3$ per m of bank at both Trout Creek sites. Hydraulic erosion at the Upper Truckee sites ranged from 0.166 to 1.12 m$^3$ per m of bank and from 0.23 to 0.445 m$^3$ per m of bank at the Trout Creek sites. The results presented in Table 9, Figure 37 and Figure 38 indicated that under current bank conditions, during a high flow year, volumes of erosion resulting form geotechnical failures dominated on the Upper Truckee River, and volumes of erosion from hydraulic erosion dominated on Trout Creek. It should be noted, however, that at the Upper Truckee sites, hydraulic erosion was usually a necessary precursor for geotechnical failures to occur. Overall erosion volumes for the 13 month period modeled were 0.75, 2.64 and 5.95 m$^3$ per m of bank for the UTR hole 6, UTR54 and UTR3 sites respectively. Total eroded volumes at the Trout Creek sites were lower than the Upper Truckee sites at 0.45 and 0.24 m$^3$ per m of bank erosion for Trout1 and Trout4 respectively. The differences between the two rivers were a result of both the general difference in bank heights (approximately 1.5 to 2m banks on the Upper Truckee, versus approximately 0.5 to 1m on Trout Creek), and the cohesion of the material at the sites. At the Trout Creek sites bank cohesion values were higher than those measured at the Upper Truckee sites, so the banks on Trout Creek withstood more undercutting by hydraulic erosion before geotechnical failures occurred.

BSTEM runs were then carried out using the same flow period, but with various mitigation strategies added to assess the ability of each for reducing bank erosion. Mitigation strategies modeled included adding riprap to the bank toe, adding riprap to 1m, regardless of the location of the toe, engineered log jams (ELJs), addition of 3, 10 and 20 year-old Geyer’s willows, and bank grading to a 2:1 slope.

The addition of riprap to the bank toe had a dramatic impact on volumes of hydraulic erosion at two of the three Upper Truckee sites (UTR Hole 6 and UTR3). Corresponding decreases in volumes of geotechnical erosion were also recorded during these simulations as a reduction in hydraulic erosion led to less oversteepening and destabilization of the banks. At UTR54 the addition of riprap to the bank toe region, led to just a small reduction in hydraulic erosion volume, and virtually no difference in geotechnical erosion volume. At this site, protection of the toe alone was insufficient to prevent oversteepening and destabilization of the bank, with geotechnical failures still resulting. At the Trout Creek sites, addition of riprap to the toe region reduced undercutting slightly at Trout1 and had no effect on hydraulic erosion at Trout4.

Addition of riprap to a height of 1m above the base of the bank toe further reduced hydraulic erosion at four of the five sites. At UTR Hole 6, no further reduction in hydraulic erosion was recorded when additional riprap was added to the model simulation, because the bank layer above the toe was fairly resistant to erosion with or without the additional riprap being present. At UTR3 and UTR54 addition of riprap to 1m reduced hydraulic erosion to negligible amounts, with no corresponding geotechnical erosion. At Trout1 riprap to a height of 1m further reduced hydraulic erosion whilst the volume of erosion from geotechnical erosion remained small but unchanged from the model run with riprap only at the bank toe. In this case any geotechnical erosion occurred from a location on the bank face above the riprap, and reductions in hydraulic erosion did not act to stabilize this part of the bank. At Trout4, addition of riprap to 1m prevented
any hydraulic erosion from happening, but similar to at Trout1, a very small amount of geotechnical erosion still occurred further up the bank.

The results from the two sets of model runs with riprap suggest that rock can be a very useful strategy for mitigating sediment loadings from unstable streambanks. This is because although hydraulic erosion generally makes up less of the total eroded volume than geotechnical erosion does, this hydraulic erosion is often required as a precursor to geotechnical erosion. The height of riprap needed to reduce sediment loadings sufficiently depends on the load reduction required and the geometry and materials at each specific site. For example, placing 1m of rock at the base of the bank at UTR Hole 6 has no greater impact than just placing rock on the 0.7m high toe. In contrast, at Trout4, no reduction in hydraulic erosion was seen until a full meter of riprap was added in the BSTEM simulations.

The next set of runs included ELJs. The height and width of the ELJs modeled depended on the bank height at each site. For the Upper Truckee sites an ELJ 1m high by 2m wide was modeled. At the Trout Creek sites, which had lower banks and generally narrower cross sections, 0.5m high by 1m wide ELJs were modeled. At the UTR sites the ELJs prevented all hydraulic erosion from occurring, thereby also preventing any geotechnical erosion from occurring. At Trout1, the ELJ scenario was more effective at reducing hydraulic erosion than riprap to a height of 1m, even though the ELJ was only 0.5m high. This was because the shape of the ELJ changed the distribution of hydraulic shear stresses, and therefore the amount of hydraulic erosion in this model run. At Trout4, the ELJ was less effective than the run with riprap to 1m because the distribution of shear stresses, although altered by the ELJ, still produced some erosion above the 0.5m high ELJ. The effectiveness of an ELJ therefore depends on the height and width of the structure, and the way that the structure changes near-bank shear stresses in the bank zone. In these runs ELJs were only considered as 1-D, non-porous structures, but in reality they are somewhat porous, 3-D structures, and may be placed at an angle to further impact the local hydraulics.

To test the effect of woody riparian species growing in addition to the wet and dry meadow grass assemblages, BSTEM simulations including different ages of Geyer’s willow trees were run. Geyer’s willow was selected out of the species tested in the Tahoe Basin, because over time it was estimated to provide the greatest amount of root-reinforcement of the woody species tested in this study. The results provided here therefore provide the upper potential limits of load reduction likely to be achieved through plantings of any combination of the species tested herein. At the UTR Hole 6 site, the addition of Geyer’s willow trees ranging from 3 to 20 years old, resulted in less hydraulic erosion of the toe compared to the run for existing conditions, but more hydraulic erosion than either of the runs including riprap. Compared to the existing conditions run, the critical shear stress of the toe and bank materials were the same, but because the bank had more cohesion due to the root-reinforcement, less geotechnical failures occurred, and the distribution of shear stresses at the bank face did not change in the same way as during the run for existing conditions. Compared to the runs with riprap, however, the toe was able to erode, resulting in higher volumes of hydraulic erosion in the runs with vegetation. At this site the increasing root-reinforcement over time did not have any further effect on total erosion volumes over the modeled time period.
At UTR 54, the total volumes of erosion (both geotechnical and hydraulic) declined from the existing conditions, as the age of the willow trees, and therefore their root-reinforcement increased from 3 years to 20 years. In particular, the increased bank cohesion due to root-reinforcement greatly reduced the volume of geotechnical erosion from 2.19 to 0.76 m$^3$ per m of bank, over the simulated period.

At UTR 3, the presence of even 3-year-old willows reduced the total volume of erosion from 5.95 to 4.71 m$^3$ per m of bank over the 13-month modeling period. This volume decreased had decreased even more significantly by the time the willows had reached 10-years of age, to just 0.234 m$^3$ of erosion per m of bank. An interesting situation arises when the simulations for 10 and 20-year old willows are compared. Although the volume of geotechnical erosion decreases again slightly for 20-year compared to 10-year willows, the amount of hydraulic erosion increases slightly. This is a result of greater hydraulic undercutting being able to occur in the 20-year willow simulation (due to greater root-reinforcement and therefore bank cohesion) before a geotechnical failure occurred, compared to the 10-year willow simulation. A similar result was also seen at the Trout 1 site.

The results for runs involving addition of Geyer’s willow trees to wet and dry meadow grass assemblages therefore showed that in general as the trees matured, the additional root-reinforcement reduced geotechnical erosion by increasing the resisting forces acting on the bank. In some cases the model results also showed that the presence of more mature vegetation can lead to the development of undercut banks that would not necessarily form if established woody vegetation was absent, as they would fail earlier. The results also suggested that planting of additional vegetation may not provide a quick-fix for bank stability, as the root networks can take a few years to grow and develop to sufficient depths to cross potential shear surfaces within the banks.

The final set of mitigation runs involved grading the bank at each site back to a 2:1 slope (26.56 degrees). Two sets of runs were carried out at the 2:1 slope angle, the first using the same manning’s n value as were used in all the other runs for that site (ie. Bank graded back but roughness elements such as new vegetative plantings, willow posts etc, added to bank), and the second using no roughness (ie. Bank graded back and left bare and smooth). For four of the five sites, by grading the bank to a 2:1 slope, and providing the bank with roughness similar to the existing conditions, volumes of erosion over the modeled period were reduced. At one site however, (Trout4) the volume of erosion actually increased after the bank had been graded back as the distribution of near-bank shear stresses was altered in such a way as to apply higher forces to weaker bank layers, compared to in the model run for existing conditions. In the runs conducted using no grain roughness, so much erosion occurred that for all but one site (UTR54) the model run could not complete. This result indicates that if bank grading is used as a mitigation strategy without also replanting, adding riprap to the toe, or using other protection measures such as geotextiles, excessive erosion could still occur. Grading the banks back may therefore be a useful mitigation strategy for reducing eroded bank volumes and thus sediment loadings, but only if used in conjunction with other techniques.
Table 9. Volumes of erosion from BSTEM runs at three sites along the Upper Truckee River and two sites along Trout Creek.

*All values are m³ per m of bank

<table>
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<tr>
<th>Manning’s n value used</th>
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<th>0.035</th>
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</thead>
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<td>0.005</td>
<td>0.004</td>
<td>0.005</td>
</tr>
<tr>
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<td>0.000</td>
<td>0.000</td>
<td>0.004</td>
<td>0.005</td>
</tr>
<tr>
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<td>3.911</td>
<td>0.003</td>
<td>0.005</td>
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<td>0.005</td>
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<td>0.000</td>
<td>1.063</td>
<td>0.020</td>
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</tr>
<tr>
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<tr>
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<td>0.445</td>
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<td>0.378</td>
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<td>Existing conditions</td>
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<td>0.000</td>
<td>0.002</td>
<td>0.088</td>
<td>0.005</td>
</tr>
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<td>2.643</td>
<td>4.706</td>
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<td>0.465</td>
<td>0.236</td>
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<td>0.590</td>
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<td>1.396</td>
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<td>&gt;&gt;</td>
<td>0.539</td>
<td>&gt;&gt;</td>
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</tr>
</tbody>
</table>

>> = too much erosion for model to complete run
**Figure 37.** Geotechnical erosion volumes under existing conditions and various mitigation scenarios. Values are in m$^3$ per m of streambank, over one flow year.

**Figure 38.** Hydraulic erosion volumes under existing conditions and various mitigation scenarios. Values are in m$^3$ per m of streambank, over one flow year.
Figure 39. Total erosion volumes under existing conditions and various mitigation scenarios. Values are in m$^3$ per m of streambank, over one flow year.

Figure 40. Before and after profiles for existing and mitigated scenarios at UTR 54.
Figure 41. Before and after profiles for existing and mitigated scenarios at UTR Hole 6.

Figure 42. Before and after profiles for existing and mitigated scenarios at UTR 3.
Figure 43. Before and after profiles for existing and mitigated scenarios at TROUT 1.

Figure 44. Before and after profiles for existing and mitigated scenarios at TROUT 4.
3.2 INVESTIGATING THE USE OF ENGINEERED LOG JAMS FOR REDUCING STREAMBANK EROSION

As part of the analysis of potential mitigation measures that could be effective for reducing streambank erosion in the Lake Tahoe Basin, a series of numerical simulations were conducted with BSTEM Ver. 5.4 to evaluate the conditions under which engineered log jams (ELJs) might prove effective. ELJs have become a popular mode of protecting streambanks. They act somewhat like bendway weirs by deflecting flow away from the bank towards the center of the channel. With hydraulic erosion of the bank toe shown to be a critical component of streambank erosion and lateral retreat, these types of structures would tend to reduce undercutting of the bank and subsequent mass failures. The analysis was conducted to evaluate under what conditions (bank height, material type and channel slope) ELJs could be potentially used alone, or need to be used in combination with other stabilization measures such as reducing bank slopes.

3.2.1 Methodology

A matrix of numerical experiments was established using a range of bank heights (3 - 8 m), channel slopes (0.0005 – 0.01 m/m) and material types (clay, silt, and sand). Bank angles were all initially set at 80°. The critical shear stress applied for the ELJs was about 200 Pa (Abbe and Brooks, 2011). Geotechnical values for cohesion, friction angle and saturated unit weight for each of the material types were obtained from the median values of distributions published in Simon et al., (2011) representing more than 800 geotechnical tests of streambank materials conducted across the United States (Table 7).

For each of the simulation trials the toe-erosion sub-model was initially run assuming a bankfull flow of 24 hours (Figure 45). The eroded geometry was then exported into the bank-stability sub-model to check for stability with the groundwater elevation set to the elevation of the water surface (Figure 46). If failure did not occur, the simulation was conducted again with the water-surface set to the top of the ELJ structure (2 m above the bed). This represented the drawdown or the most-critical case for stability. If failure did occur, the geometry was updated and bank stability was simulated again. If the factor of safety at this point was determined to be greater than 1.0, the bank slope was, therefore, assumed to indicative of an angle that could be used for design in combination with the ELJ.
**Figure 45.** Simulation of hydraulic erosion for a 4 m-high silt bank with a slope of 0.005 m/m. This eroded geometry is then exported into the bank-stability sub-model to evaluate geotechnical stability.

**Figure 46.** Simulation of bank stability for a 4 m-high silt bank with a slope of 0.005 m/m after hydraulic erosion by a bankfull flow. The resulting factor of safety was 1.39, indicating stability. Under drawdown conditions, this bank was unstable, failing along the red line shown in the figure.
3.2.2 Results

As one might expect, ELJs are most effective as a stand alone measure for clay banks where cohesive strengths are greater than for silt or sand. Clay banks of up 6 m, over the range of channel slopes tested are effective at maintaining bank stability without bank re-shaping (Table 10). This assertion does include, however, cases where one failure did occur. However, the resulting flatter bank slope can then be maintained because of the hydraulic protection provided by the ELJ. In contrast, results for sand banks show more than one failure under each scenario, even after the bank slope has been reduced ‘naturally’ by the mass failure (Table 10). Results for the silt banks are intermediate and indicate that bank re-shaping at angles between 26 and 57 degrees may have to be included in mitigation schemes, particularly for higher stream banks. Results of all of these experiments are tabulated in Table 10.
Table 10. Summary table of ELJ results for different bank heights and material types.

<table>
<thead>
<tr>
<th>Material</th>
<th>Gradient</th>
<th>Bank Height (m)</th>
<th>ELJ Height (m)</th>
<th>Hydraulic Erosion (Pa/m²)</th>
<th>Factor of Safety / Drawdown</th>
<th>After Failure/Stable Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>0.01</td>
<td>3</td>
<td>2</td>
<td>81.0/0.219</td>
<td>3.86/1.55</td>
<td>-</td>
</tr>
<tr>
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<td>0.01</td>
<td>4</td>
<td>2</td>
<td>166/0.94</td>
<td>1.91/0.71</td>
<td>2.02/50</td>
</tr>
<tr>
<td>Clay</td>
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<td>6</td>
<td>2</td>
<td>333/2.55</td>
<td>0.34/0.37</td>
<td>1.03/51</td>
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<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>3</td>
<td>2</td>
<td>83.9/0.312</td>
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<td>1.93/57</td>
</tr>
<tr>
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<td>2</td>
<td>174/1.78</td>
<td>1.06/0.33</td>
<td>0.87/26</td>
</tr>
<tr>
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<td>2</td>
<td>333/3.45</td>
<td>0.72/-</td>
<td>0.48/-</td>
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<tr>
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<td></td>
<td></td>
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<tr>
<td>Clay</td>
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<td>3</td>
<td>2</td>
<td>40.5/0.093</td>
<td>5.06/2.05</td>
<td>-</td>
</tr>
<tr>
<td>Clay</td>
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<td>2</td>
<td>83.2/0.419</td>
<td>2.47/0.92</td>
<td>2.46/44</td>
</tr>
<tr>
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<td>2</td>
<td>166/1.17</td>
<td>1.41/0.45</td>
<td>1.10/43</td>
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<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silt</td>
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<td>3</td>
<td>2</td>
<td>40.5/0.146</td>
<td>2.63/0.93</td>
<td>2.20/55</td>
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<td>2</td>
<td>83.2/0.625</td>
<td>1.39/0.43</td>
<td>1.17/47</td>
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<tr>
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<td>2</td>
<td>166/1.69</td>
<td>0.94/-</td>
<td>2.93/-</td>
</tr>
<tr>
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<td>3</td>
<td>2</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Clay</td>
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<td>2</td>
<td>8.1/0.005</td>
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<td>16.6/0.048</td>
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<td>1.77/0.55</td>
<td>1.19/41</td>
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<td>33.3/0.29</td>
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<td>0.95/-</td>
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<td>0.98</td>
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<td>2</td>
<td>4.05/0.003</td>
<td>0.97</td>
<td>6.47/38.1</td>
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</table>

After 2nd failure (BS-3)
Unstable throughout
4. SUMMARY AND CONCLUSIONS

To evaluate potential reduction in fine-sediment loadings emanating from streambanks, it was necessary to analyze the discrete processes that control streambank erosion under existing and mitigated conditions. This was accomplished by quantifying the controlling driving and resisting forces that affect bank steepening by hydraulic erosion and mass-bank stability, controlled by gravity. These processes include hydraulic erosion of bank-toe and bank-surface sediments, mass failure of upper-bank materials and the effects of vegetation, if present. Many of these processes have been modeled previously in the Lake Tahoe Basin (Simon et al., 2006) using the static version of the Bank-Stability and Toe-Erosion Model (BSTEM) developed by the USDA-ARS, National Sedimentation Laboratory (Simon et al., 1999; 2000). This work identified several shortcomings of the analysis and simulation tool that have been addressed in this project through further research on bank processes, the role of the above- and below-ground biomass of riparian vegetation, and development of a dynamic version of BSTEM that accepts time-series stage data.

In this report we have summarized the research findings and enhancements made to BSTEM to address the three main study objectives:

1. Research on bank processes, including the effect of failed blocks on bank erosion rates, and effects of root-reinforcement on streambank stability.
2. Development of BSTEM-Dynamic, to be able to utilize continuous flow-stage data, development of a near-bank groundwater model, and incorporation of common Tahoe Basin riparian species to the RipRoot sub-model within BSTEM.
3. Investigation of the effectiveness of erosion-control strategies, including BSTEM-Dynamic runs involving multiple mitigation methods, and an analysis of the effect of engineered log jams (ELJs) on bank stability.

Previous research on streambank erosion rates along the Upper Truckee River identified the importance of failed blocks that come to rest at the toe of the bank on erosion rates (Simon et al., 2009). In that study, streambank erosion rates were over-estimated because simulations of hydraulic erosion over a series of flow events were resisted by in situ material only. Because these blocks are generally permeated with a dense network of grass roots, it was hypothesized that they would be more resistant to hydraulic erosion by subsequent flows. Research was conducted at one site on the Upper Truckee River (UTR3) and two sites on Trout Creek to quantify the erosion resistance of root-permeated failure blocks. Results of these simulations support the original hypothesis as to why erosion rates were over-estimated by not accounting for the vegetated blocks at the bank. These features are typical of incised channels cutting through dry- and wet-meadow environments and clearly need to be accounted for in any mechanistic approach to predicting streambank stability and channel-migration rates. In combination with the data collected on the root-reinforcement attributes of these meadow species, accurate modeling of bank-stability conditions can now be undertaken. As a prelude to using the empirical relation between root biomass and critical shear stress (owing to a low number of matching samples/tests), the increased critical shear stress of bank-surface due to the below ground biomass is accounted for by increasing the value obtained for bare soil by an order of magnitude.
As part of this study, four riparian tree species typical of the Upper Truckee River and Trout Creek watersheds were selected for testing of root-reinforcement: lodgepole pine, Lemmon’s willow, Geyer’s willow and Alder. (*Pinus contorta, Salix Lemmonii, Salix geyeriana, Alnus incana*). In addition, wet meadow and dry meadow grass assemblages were tested. Estimated values for root-reinforcement provided by these species were shown to vary by plant age and by the bank material the roots are growing in. In rounded sand root pullout dominated over root breaking as a failure mechanism, and root-reinforcement values were relatively low. Variations in root-reinforcement in sand for the tree species studied ranged from 0.01 to 1.67 kPa for lodgepole pine and from 0.01 to 2.27 for the remaining tree species studied. Little difference was seen between the species in the rounded sand scenarios as root pullout is independent of root strength. Greater differences were seen between the species in scenarios for moderate silt and soft clay banks where root breaking became a more important failure mechanism. Of the tree species studied, the Geyer’s willow roots were the strongest and therefore provided the highest modeled root-reinforcement values at each stage of growth. Root-reinforcement values for Geyer’s willows were estimated to reach 2.27, 9.35 and 11.2 kPa of reinforcement in sand, silt and clay respectively after 30 years of growth. Root-reinforcement can therefore provide significant increases to soil strength, but the amount of reinforcement is spatially variable according to species distributions, and soil types, and temporally variably according to rates of plant growth and root system development.

The improved understanding of the bank processes investigated for Objective 1 led to developments in BSTEM required for Objective 2. To account for failed blocks at the toe containing roots, critical shear stresses are increased by an order of magnitude, resulting in slower, more realistic rates of erosion. This order of magnitude increase can be justified from the results of mini-jet tests carried out on failed blocks at both Upper Truckee and Trout Creek sites. Additionally, BSTEM was improved by adding the Tahoe Basin riparian species to the database in the RipRoot submodel. Significant changes were also made to the original BSTEM-Static model to create BSTEM-Dynamic, capable of handling continuous hydrograph data, and incorporating a near-bank groundwater model. This groundwater model was tested against measured tensiometers and groundwater data from one site on Trout Creek and one site on the Upper Truckee, over a year long period.

At the site on Trout Creek the groundwater model performed very well against monitored data until the onset of snowmelt in the spring of 2008. The discrepancy in monitored and modeled groundwater values during this period results from the fact that the near-bank groundwater model is linked to within-channel stage, and cannot account for infiltration of surface water entering the bank from precipitation or snowmelt. During the summer months, the modeled groundwater elevation remained approximately 10 cm higher than the monitored groundwater elevation; this may have been a result of the model’s inability to account for evapotranspiration as an additional process capable of removing water from the bank and lowering the water table. Similar responses were seen in the results for the UTR3 site, with the modeled and monitored groundwater generally matching very well until spring snowmelt in 2008. In early summer the effect of evapotranspiration had less of an effect on the monitored groundwater level at this site than at the Trout 1 site, most likely because the Upper Truckee River streambank is taller than the bank at the Trout Creek site. Removal of surface moisture by evapotranspiration therefore had less of an effect on groundwater deeper within the bank, early in the summer. This resulted
in the monitored and modeled groundwater matching closely at this site after snowmelt had finished. Later in the summer months of 2008 there was approximately a 10 cm discrepancy in water table heights, with monitored values being lower than modeled values. This may have occurred because plants had to remove water from greater soil depths as the summer months progressed, with the effect of evapotranspiration therefore not being seen until later in the summer at this site.

The final part of this project (Objective 3) required testing of the new BSTEM-Dynamic. Additionally, runs were carried out for existing conditions at several Upper Truckee and Trout Creek sites, with further runs to test the potential for mitigated scenarios to reduce current bank erosion loadings. Under existing conditions, during a high flow year, volumes of erosion resulting from geotechnical failures dominated on the Upper Truckee River, and volumes of erosion from hydraulic erosion dominated on Trout Creek. It should be noted, however, that at the Upper Truckee sites, hydraulic erosion was usually a necessary precursor for geotechnical failures to occur. Overall erosion volumes for the 13 month period modeled were 0.75, 2.64 and 5.95 m$^3$ per m of bank for the UTR Hole 6, UTR54 and UTR3 sites respectively. Total eroded volumes at the Trout Creek sites were lower than the Upper Truckee sites at 0.45 and 0.24 m$^3$ per m of bank for Trout1 and Trout4 respectively. The differences between the two rivers were a result of both the general difference in bank heights (approximately 1.5 to 2m banks on the Upper Truckee, versus approximately 0.5 to 1m on Trout Creek), and the cohesion of the material at the sites. At the Trout Creek sites bank cohesion values were higher than those measured at the Upper Truckee sites, so the banks on Trout Creek withstood more undercutting by hydraulic erosion before geotechnical failures occurred.

The results from the two sets of model runs with riprap suggest that rock can be a very useful strategy for mitigating sediment loadings from unstable streambanks. This is because although hydraulic erosion generally makes up less of the total eroded volume than geotechnical erosion does, this hydraulic erosion is often required as a precursor to geotechnical erosion. The height of riprap needed to reduce sediment loadings sufficiently depended on the load reduction required and the geometry and materials at each specific site. For example, placing 1m of rock at the base of the bank at UTR Hole 6 had no greater impact than just placing rock on the 0.7m high toe. In contrast, at Trout4, no reduction in hydraulic erosion was seen until a full meter of riprap was added in the BSTEM simulations. The effectiveness of an ELJ was similar to addition of riprap to a bank, in that its presence greatly reduced hydraulic erosion of the bank toe and face. The effectiveness of an ELJ does, however, depend on the height and width of the structure, and the way that the structure changes near-bank shear stresses in the bank zone.

The results for runs involving addition of Geyer’s willow trees to wet and dry meadow grass assemblages therefore showed that in general as the trees matured, the additional root-reinforcement reduced geotechnical erosion by increasing the resisting forces acting on the bank. The results also suggested that planting of additional vegetation may not provide a quick-fix for bank stability, as the root networks can take a few years to grow and develop to sufficient depths to cross potential shear surfaces within the banks. BSTEM results also indicated that if bank grading was used as a mitigation strategy in BSTEM without also replanting, adding riprap to the toe, or using other protection measures such as geotextiles, excessive erosion could still occur. Grading the banks back may therefore be a useful mitigation strategy for reducing eroded bank
volumes and thus sediment loadings, but only if used in conjunction with other techniques that maintain channel roughness elements to absorb some of the excess shear stresses acting within the modified reach.

In summary, the most effective mitigation strategies were those that were able to reduce or prevent hydraulic erosion from the bank toe and bank face. By reducing the volume of erosion from these areas of the bank, the geotechnical stability of the banks tended to remain higher as the banks did not become oversteepened or undercut. Model runs involving the growth of riparian trees also indicated that root-reinforcement can be a significant factor for maintaining geotechnical stability, but only once they have matured enough for the roots to cross potential failures planes within the banks. Vegetation was also seen to be very important in terms of channel roughness as vegetation growing on the banks and in the toe region act to absorb some of the flow energy, thereby reducing the energy available to scour the substrate. Removal of vegetation would, therefore, decrease channel roughness and likely increase erosion rates throughout these channel systems. Further planting and growth of riparian species would, conversely, help to reduce rates of bank erosion.
REFERENCES


