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Stability evaluation of streambanks in Vermont

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Abstract

Streambank stability characterization has become a topic of major interest with rising concerns regarding phosphorous accumulation in surface water bodies. The ability to make rapid reliable determination of a streambank's stability is necessary as the ability to make informed land management decisions becomes paramount. Since the summer of 2006 two steam reaches in the Lake Champlain basin of northern Vermont have been characterized and monitored through the use of surveying, geotechnical investigations, and instrumentation. Temporal survey results were used to observe mass failures, sediment removal and deposition along the streambanks throughout the course of this study. Laboratory testing was used to determine soils, strengths, root strengths and soil erosion characteristics. Stability of the streambanks was analyzed using classical soil mechanics incorporating variable stream and ground water tables and results from the laboratory testing; revealing year round instability of many of the streambanks observed. This study showed the validity of the Iowa borehole shear test and the necessity to incorporate matric suction additions to soils strength.

1. Introduction

A large portion of potable water is contained in the small percentage of fresh surface water found on the surface of the earth. Today these fresh water sources are under constant threat from accelerated pollution due to anthropocentric causes. Pollution found in surface waters is caused by toxic and conventional pollutants (Kreger 2004). Although toxins pose immediate and serious health threats represent only a small portion of the total pollution found in our waterway. Conventional pollution represents the larger portion of pollution found today, being defined as nutrients and biological contaminants that enter waterways through natural means. Of pollutants phosphorus (P) has become recognized as the largest in US lakes and water ways(Agency 2009). Anthropocentric activity has been shown to increase conventional pollution levels; particularly in levels of nitrogen and phosphorous (Carpenter 1998). Responsible for these higher loading rates of conventional pollutants are commonly sewage treatment effluent, stormwater runoff, and non-point sources associated with agricultural and land management practices.

Large inorganic sources today are both point and non-point agricultural sources of particular importance due to increasing fertilizer application rates (Holtan 1988). Due to phosphorus nature to tightly bind to soil particles(Barros 2005); P transport is closely linked with that alluvial sediments(cite). Particulate-bound phosphorous (PP) levels quantified at the outflow of a watershed is a combination of PP that has entered into the waterways recently via point and non point sources, remobilized PP from alluvial deposits through stream bed migration and erosion processes. Streambank erosion including scour and mass failure is estimated to account for 30-80% of sediment loading into lakes and waterways (1999; Evans 2006; Fox 2007).

Much is known about sediment source processes, transport, and deposition. However modeling of these processes has applicability only to study regions where sources of sediments may be easily identified. Several direct measurement procedures have been used in efforts to quantify bank erosion. Lawler (1999) made use of longitudinal surveys and pins to quantify sediment loading though bank erosion. Longitudinal surveys allowed the measurement of bank top retreats while the use of pins allowed measurement of toe erosion of laterally migrating streambanks. As such direct techniques have been found valuable in determining sediment loads from sources in small watersheds that can usually be readily recognized(cite). Labor intensity of direct measurement techniques makes large scale use infeasible. Due to the labor of purely quantitative methods and waning confidence in qualitative methods it is necessary to develop a semi-quantitative approach to determine bank stability. Such an approach would make use of bulk samples and inexpensive testing procedures to determine soil types which would allow for determination of material properties of soil through the use of empirical calculations and index properties.

The following study was preformed as a initial efforts on developing a method for determining streambank stabilities in the alluvial river beds of Vermont. A series of surveys were conducted to monitor geometric changes in river cut banks. Subsurface investigations were also preformed in efforts to characterized local soil deposits type, and strength. Data gathered from these studies were used to analyze the changing stability of river cut banks using classical soil mechanics.

2. Factors Affecting Streambank Stability

2.1 Geotechnical Material strength

The most visible process that causes streambank changes is that of mass failure. These failures can be analyzed by slope stability methods. Several methods of analyzing slope stability are applicable to assessing the stability of streambanks for a single point in time (Langedoen et al., 2001). Bishop (1955) developed a technique to assess slope stability on a circular failure surface. Although failure surfaces are often circular non-circular failure surfaces occur more commonly in streambanks where common failure geometries are slab, rotational, wedge, and cantilever with a dependence on the amount of bank undercutting prior to failure(Simon and Collision 2002). To account for non-circular failure surfaces an analysis technique was developed that divides a predefined failure surface into a number of finite slices, where the driving and resisting forces along each section of the slip surface for each slice are quantified and summed (Morgenstern and Price 1965). Values from this method are then looked at in terms of a factor of safety; defined as the ratio of resisting forces to the driving forces. To quantify the resisting forces of the soil the Mohr-Coulomb failure criterion is used (equation 1).

$$\tau = c^{'} + (\sigma_n - \mu_w)tan\varphi' \quad (1)$$

Where c' is the effective cohesion of the soil; σ_n is the confining pressure on the slip surface; μ_w is pore water pressure, and φ' is the soils effective angle of internal friction. If the driving forces exceed the resisting forces for the given failure surface a factor of safety of less than one is seen indicating slope instability and failure. These methods tend to provide accurate analysis for instantaneous slope stability of a given geometry. However, this system is used to predict the stability of saturated slopes where only non-negative pore water pressures are allowed to develop.

2.2 Vegetation effects

Slope stability of streambanks is inherently more difficult to calculate than that of an earthen structure. Another aspect that needs to be considered when evaluating slope stability is the effects of vegetation found on streambanks. Vegetation has two primary effects on the stability of streambanks the surcharge imposed by vegetation on the bank and the shear strength addition caused by the root structure in the soil (De Wiel and Darby 2007). The surcharge can be added into a slope stability analysis quite easily by the addition of a pressure to the bank top. However the strength addition caused by roots is more difficult to incorporate. It has long been assumed that the presence of roots in soils increases the shear strength of soils, and therefore the stability of streambanks(Collision 2002). The soil root system creates a composite material where roots add tensile strength to the material while the soil provides compressive strength. The added strength of the soil-root composite material is dependent upon the fractional volume of live roots, tensile strength, and orientation of the roots. (Oshashi and Gray,1983; Wu *et al.*, 1979). The directional orientation of a root system through a failure surface has been shown to change the strain at which maximum shear strength is encountered (Wu, McKinnell III et al. 1979; Ennos 1990). Furthermore the spatial distribution of roots, generally quantified as a root-area ratio, is non uniform. With grass and sages that are densely grouped the root area ratio may be assumed as linear function with a maximum root area ratio at the surface and degrading to zero at the species maximum depth of rooting. With shrubs and trees the distribution is more sporadic and the root area ratios decrease in both the horizontal and vertical directions from the stem/trunk forming a cone shape root area ratio distribution that extend far beneath the surface (De Wiel and Darby 2007).

Equations developed by (Waldron 1977) estimate strength addition to soils due to the tensile properties of roots. Strength additions to a soil may be added as an increase in the cohesion of the soil as modeled by the Mohr-Coulomb failure criterion (equation 2).

$$\tau = c_r + c'_s + (\sigma_n - \mu_w)tan\varphi' \tag{2}$$

Where τ is the soil's shear strength; c_r is apparent cohesion due to roots; c'_s is the soil's effective cohesion; σ_n is the effective confining pressure; μ_w is the pore-water pressure and φ' is the effective internal shear angle of the soil. Waldron (1977) developed a generalized equation to describe additional cohesion provided by roots (equation 3.). This equation was later simplified after determining that for most variations of φ and θ a coefficient of 1.2 can be used with less than 10% additional error(Wu, McKinnell III et al. 1979). As determination of the root area ratio of soils lends to significant variability in estimated shear strength additions due to root systems this simplification may be used with negligible effect on the results.

$$c'_r = (sin\theta + cos\theta tan\varphi)(T_r R_a) \quad (3)$$

Where T_r is the tensile strength of roots; R_a is the root area ratio defined as the ratio of root volume to total volume of a given soil sample; and θ is the angle of roots passing through the failure plane.

2.3 Soil Pore-water Pressures

Pore-water pressures have been long known to affect the stability of an engineered slope, with positive pore-water hydrostatic forces creating positive pressures below the phreatic surface, with saturated conditions accounted for in the Mohr-Coulomb failure criterion (equation 1.). Due to the transient nature of stream water levels streambanks are often comprised of regions of saturated and unsaturated soils. Above the phreatic surface of the water negative poor water pressures develop which could affect the strength of the materials significantly and therefore the stability of the slope. The effects of negative pore-water pressures (matric suction) most easily are observed by the step bank angles that occur at eroding stream sites. Matric suction often develops during the lowering of water levels that causing disjointing of negatively pressurized water from the capillary fringe, and may be increased through effects of dehydration and evapotransportation(cite). Predictions of the magnitude of matric suction can be made through use of the soil water characteristic curve (figure 2.1.), which relates the value of matric suction to the degree of saturation of the soil.

For soils at 85% saturation or higher, at or below the air entry pressure, low values of matric suction and uniform pore-water distribution provide for easy analysis. Soil strengths in this zone can be analyzed using classical soil mechanics by using proper sign convention to account the negative water pressures (Fredlund and Rahardjo 1987). For values of matric suction greater than the air entry pressure the soil desaturates and pressures become heterogeneous (Mohamed, Ali et al. 2006). Changes in water content and pressures at the granular scale in the unsaturated zone were cause for a revision of the Mohr-Coulomb failure criterion by Fredlund and Rahardijo(1987) to the form of equation 4 to account for these inconsistencies.

$$\tau = c^{'} + (\sigma - \mu_a)tan\varphi' + \psi tan\varphi^{b} \qquad (4)$$

Where τ is the unsaturated shear strength of the soil; c' is the effective cohesion of the soil; σ is the stress applied to the soil; μ_a is the barometric pressure; φ' is the effective friction angle of the soil; ψ is the matric suction pressure, and φ^b is the angle of strength change with variations in matric suction.

2.4 Hydraulic effects

Another difficulty with predicting the long term stability of a stream bank is its constantly changing shape. Most bank failures occur due to undercutting, with the failure mode geometry strongly related to the degree of undercutting of the bank (Simon and Andrew 2001). Unlike more noticeable changes in the stream this undercutting is not caused by mass failure, but by scour erosion. The rate at which this erosion takes place is illustrated by the following equation.

$$E_r = k_d(\tau_e - \tau_c) \quad (5)$$

Where E_r is the erosion rate, k_d is the erosion coefficient, τ_e is the effective shear stress, and τ_c is the soils critical shear stress (Hanson, 199#). Scour occurs when effective shear stress exerted on the exposed portion of soil is greater than the critical shear stress of the soil causing soil detachment (Hanson 199#). Empirical correlations for soils with a mean particle diameter greater than 0.3mm the critical shear stress (pa) is equal to the mean grain size D_{50}(mm). For soils with D_{50}<0.3mm this equation is not useful for an accurate determination of critical shear stress (Briaud J.L. 2001). In soils consisting of finer grains delicate intricacies make predication of the erosion characteristics very difficult. Erosion of soils of this size is often affected by difficult to quantify electro-chemical interactions that may create multi partial erosion characteristics (Mazurek, Rajaratnam et al. 2001). In order to obtain critical shear stress and erosion rate values direct experimental testing is needed.

3. Site Selection and Instrumentation

3.1 Site Selection

Several requirements were needed for a stream in this study. Due to the time necessary for site characterization a multitude of site visits would need to be preformed. To accommodate this only stream reaches within 50km (35miles) of Burlington , VT would be considered for this study. Due to the nature of geotechnical investigations permission from landowners was also necessary, which was conditionally obtained for an initial non-intrusive site visit. Access was also a consideration as remote areas would prevent use of heavy drilling equipment. Reaches from a previous study conducted by Hession et al. (year) meeting this criteria were individually visited. Of the reaches that were investigated permission was granted to install instrumentation at a number of these sites; however permission was not given for any heavy drilling equipment.

Of the reaches where instrumentation privileges were granted further selection was conducted. Reaches with prevalent failing streambanks and streambanks that were marginally stable were further considered. The height of the falling banks was also taken into consideration with an optimal range of 2-5m (6.6-16.4ft) in height to provide sufficient survey resolution and subsurface investigation using hand auguring equipment. The soil deposits of the area had to be primarily composed of sand, silt, or clay. This provision was to insure that hand operated drilling equipment and insitu strength tests could be readily conducted for a more accurate quantification of the subsurface soils. Two sites were selected meeting these requirements. The first site selected was a lower reach of the Winooski River located in Burlington, VT. The second site is located in the lower reaches of Lewis Creek located in Ferrisburg, VT adjacent to Lake Champlain (see figure 3.1.).

Seven cross sections were selected at each reach for temporal monitoring. In order to observe changes in the streambank geometry a series of topographic surveys were used. Although time consuming this method was selected for use instead of the less labor intensive pin method developed by Lawler (year). This method allowed for changes in the entire cross section to be noted which would be particularly important in characterizing slope failure geometries The population of cross sections observed was composed of streambanks that had recently failed or were marginal stable. To facilitate multiple repeated surveys of the same cross sections pairs of iron rods were placed perpendicular to the channel flow allowing for repeated surveys of the same stream cross section. Survey sets were conducted using a Topcon GS 236 total station during the summer of 2007 and 2008 with additional surveys during the fall of 2008. In one instance a large bank failure prevented repetitive surveys of a cross section at the Lewis Creek site as both survey pins were removed.

3.2 Site Instrumentation

At each stream reach one cross section was selected for instrumentation, to monitor water level and bank activity information. Along the instrumented cross section 2-3 monitoring wells were put in place. Wells at the Winooski and Lewis Creek instrumented sites were places to depths of 4.3-4.9m (14-16ft) and 3.7-4.3m (12- 14 ft) respectively; allowing monitoring water levels at or above base flows. Water levels at each well were measured by use of a set of Solonist level loggers, data logging pressure transducers. Data collected with these would then be adjusted to account for barometric conditions, monitored by a barologger, to determine stream and ground water levels. A cross section of a typical instrumented site can be seen in figure 3.2.

In addition to the monitoring ground water elevations it is also desirable to know the stream water elevation for modeling purposes. To do this a pressure transducer was located with direct exposure to the stream water. Due to the variation in the streams monitored different approaches were taken to expose the logger directly to the stream. Due to depth and flow conditions at the Winooski River reach directly exposing the logger to the stream water was not possible. To determine the water levels in the stream without a media a non-vertical monitoring well with a zenith angle of 40˚ used. Placements of this well, in a steep vegetated bank adjacent to the stream, allowed exposure to the lower well sections directly to the stream. Shallower angles of the streambanks located at the Lewis Creek reach prevented stream water monitoring using a similar fashion.   To expose the pressure transducer to the streams water a section of well piping was capped at either end and then pinned to the streambed by use of 61cm (24in) long by 13cm (0.5in) diameter rebar staples. The logger was then tethered to the pins using a 0.48cm (0.19in) stainless steel cable and cable crimps to prevent loss if damage was sustained to the well tube.

If a bank failure were to occur during the course of this study it was preferable that the time of the failure could be recorded in order to determine what conditions caused the bank to be unstable. To do this a series of roller ball tilt switches were deployed along the instrumented cross section. A four channel third party Hobo Data loggers encased in a weatherproof box was then linked to the tilt switches imbedded into the streambank. A series of tilt switches were placed horizontally in the bank at an off position and a 2hour logging interval (see figure 3.2.). If a switch moved out of plane by more than 15˚ the switch would close and the logger's impulse signal, 2.5 volts, would be registered, indicating a failure.

4. Site Investigations

4.1 Soil Composition

Typically two boreholes were augured at each cross section to a depth below the bottom of the stream bed or the maximum depth possible at the site. Differences in soil characteristics such as color, texture, inclusions, and odor were noted for each borehole to determine if and where stratified soils were found. When significant differences were seen in the soils characteristic a bulk sample was collected for grain size analysis. Shelby tube samples were also collected at these and used for soil strength characterization using direct shear testing. Although the sampling was done using some impact which compromised the quality of the recovered samples.

The collected bulk samples collected were brought to the University of Vermont's geotechnical facility, where standard sieve analysis (ASTM 136-96) was preformed. For some soils standard sieve analysis alone was not sufficient for soil classification. On a select number of these samples further testing including Atterberg limits and hydrometer analysis. Banks the Winooski River reach are primarily composed of silty sandy soils with little or no cohesion and a gradual coarsening with depth. Soil strengths in this reach are highly variable with no relationships found between friction angle and elevation or depth from surface. An overview of soils found at each reach can be seen in table 4.1. In general the Lewis Creek sediments had a coarser composition than those of the Winooski reach with overlaying soils comprised primarily of sandy soils with some silt. Elevational stratification is found at the Lewis Creek site with a 0.3-0.45m (1-1.5ft) cobble layer found below the sandy layer and on top of a marine clay layer(see figure 4.1).

4.2 Unit Weights

Strength and unit weight properties were determined using index properties suggested by the US Naval Facilities Engineering Command Headquarters and Coduto(2001)(navfac cite), for each characterized soil type. Index properties found the unit weight to be 16.7kN/m^3 (106.3pcf) for silty sand and 16.6kN/m^3 (103.5pcf) showing quite good agreement with field testing results determined through use of the sand cone method (table 4.1.).

4.3 Soil Strengths

To quantify the shear strength properties of soils two separate testing procedures. Insitu tests were performed by use of the Iowa Borehole Shear Test (BST) apparatus. From each borehole at least two sets of shear tests were conducted where changes in soil texture or color was observed. Prior to beginning a BST set a thin tube Shelby tube sample 20.3cm (8in) long by 6.4cm (2.5in) in diameter was taken to smooth borehole walls increasing the accuracy of the test. BSTs were conducted with a range of consolidation pressures from 15kPa (2.2psi) to 120kPa (17.5psi) using 15kPa (2.2psi) increments with a consolidation period after each test allowing excess pore water pressure to dissipate. The values of maximum shear stress were then plotted against the tests normal pressure allowing for determination of the internal friction angle and the cohesion of the soil. As use of the BST is still uncommon it was necessary to compare results collected with this equipment against the more commonly preformed direct shear test (DST). Shelby tube samples that taken prior to running the BSTs were sectioned into 2.54cm (1in) tall samples and used in a series of DSTs in accordance with ASTM D3080-04 using a Geo Comp ShearTrac II automated shear device. Values obtained from both of these tests showed that the friction angle values obtained using the BST were consistently higher than those determined using the DST. The variation between these results was attributed to sample disturbance during collection and preparation of the DST samples. As such friction angles obtained by use of the BST were used for analysis purposes.

Strength testing of the soils found at the Winooski River site reviled highly variable with friction angle values ranging from 23-43˚ with an average of 33˚ and no correlation to the elevation of sampling (see figure 4.2.). Friction angles across the Lewis Creek reach were less variable with angles ranging from 32-41˚ and an average friction angle of 39.5˚(see figure 4.3.). Index properties for the primary soil types at each reach had reasonable agreement with the values measured. Internal shear angles of 27-37˚ was determined for silt and 28-38˚for sand. This showed good agreement with the values found at the Winooski with averaged for both indexed values and measured at 33˚. Friction angle values measured at the Lewis Creek reach were found to be outside of range of index values.

4.4 Tensile Reinforcement

To quantify the effect of roots on the shear strength for soils two different approaches were taken in this study. The first was to use the modified tensile reinforcement equation(3) developed by Wu et al(1979). To use this equation two values needed to be determined the tensile strength of the roots present at the sites, and the root area ratios in the impregnated soil. Along both sites the dominant species was golden rod (S. Canadensis). Root balls of the plants were taken at each of the study reaches and root samples less than 1 mm in diameter were harvested for tensile testing. To maintain the tensile strength that would be displayed by roots in the field the samples remained in a moist environment to prevent dehydration. The ends of the sample root sections were anchored into epoxy molds where epoxy was poured and allowed to fully harden. The samples and the epoxy anchors were then transferred to a tensile testing machine where they were loaded until the root material ruptured. The average rupture force for each root was then normalized by the cross sectional area of each root sample and averaged to determine the tensile strength for the roots found in the samples.

The second method of determining the addition cohesion due to roots was by use of a series of DSTs. Samples were collected from depths of up to 0.46m (1.5ft) with zenith angles of

0,45, and 90˚denoted as β (Figure 4.4). Three samples from each angle were run as consolidated drained tests with normal pressures of 13.8-48.3kPa (2-7psi). A shear stroke of 13mm (0.5in) was selected to allow for full mobilization of the roots shear strength addition. The maximum shear strength for each sample was then used to determine c' and φ' for each value of β. The change in cohesion between these samples and similar samples without roots was taken to be the additional cohesion due to tensile root reinforcement.

The samples run in the DSTs were then processed to remove the roots for calculation of the root area ratio. Samples were washed through number 16 (1.18mm, 0.0464in) and 50 (0.30mm, 0.0118in) sieves to allow soil and roots to be separated. The roots then received a second washing on the number 50 sieve where they were collected. These samples were then allowed to dry at 22˚C (72˚F) until dry. The low drying temperature and the relatively short time insured that the roots did not fully dehydrate which may change the measured root volume. Root samples were observed under a magnifying glass to determine if any residual moisture remained on the surface of the roots. Samples were then placed in a 50ml (1.7oz) graduated cylinder with 15ml (0.51oz) water, and submerged using a calibrated rod of a known diameter. The volume addition of the rod was subtracted from the final additional volume in the graduated cylinder allowing for the volume of the roots to be determined. Using the original volume of the sample the additional cohesion due to root strength was easily calculated.

Cohesion due to root addition using the modified equation developed by Wu et al.(1979) found to be 165-765kPa (24-111psi) using the averaged measured root tensile strength of 50MPa (7200psi).This unrealistic value of reinforcement calculated by this method was attributed to the difficulty of measuring the root area ratio in small samples and discarded from use. The values of additional cohesion determined by use of the DSTs was found to be 6.3kPa

(0.92psi) with no noticeable dependence on the zenith angle of sampling (see figures 4.5. and 4.6.).

4.5 Matric Suction

A series of tensiometer measurements were conducted during the summers of 2007 and 2008. Tensiometer were placed at various depths, 6-3 in (0.15-0.81m), below the ground surface and allowed to equilibrate for 5hrs. Throughout the course of this sampling only one negative pore-water pressure value was measured. This indicates one of two things either the moisture level of the ground was high enough to have minimal suction values or proper installation of the tensiometers was not conducted.

4.6 Erosion Characteristics (test not yet conducted)

Characterization of the soils erosion properties “were” carried out by the use of a series of Jet Erosion Test, in accordance with ASTM standard D 5852-00. Samples measuring 0.25m (10in) in diameter by 0.20m (8in) were collected using a sharp edged polyvinyl sampling tube from the base material at each stream reach and transported to the University of Vermont’s geotechnical testing facility. Prior to running the test the soil sample was extruded and leveled to provide an initially uniform surface. Scour depth was measured at 10 min increments for the duration of the test. Each test was run until the equilibrium scour depth was reached. The soil was then advanced upward in the tube and retested. Results from the test were then used to determine the coefficient of erosion and the critical shear velocity. Average erosion rates were determined to be ### and ### for the Winooski River and Lewis Creek reaches respectively. While the critical shear stresses were found to be ### and ### for the Winooski River and Lewis Creek reaches respectively.

5. Analysis

5.1 Lateral Retreat

The point of maximum lateral retreat was noted for each cross section along both of the reaches and then averaged. Across the seven sites at the Winooski average maximum lateral retreat rate was measured to be 0.061m/month (0.2ft/month). The average rate at the Lewis Creek site was found to be similar at 0.046m/month (0.15ft/month). Although the mechanism that caused these rates is unknown removal on steeper banks were from areas not commonly exposed to water indicating a mass failure due to instability. Shallower banks with material removal were generally from the toe of the slope and in a vertical fashion indicating scour erosion as a cause. Additionally two cross sections of the Lewis Creek reach, located at the start of a cut bank showed signs of aggregation towards the base of the slop with no indication of a failure that would have deposited these materials.

5.2 Slope Stability

Several models were used to look to evaluate the stability of the streambank cross sections, the widely available toe erosion and slope stability spread sheet package, GeoStudio’s stability modeling program SLOPE/W, and efforts were made on merging several algorithms into a single model. Initial modeling efforts were focused on the toe erosion and slope stability spread sheet package which is capable of modifying the bank geometry of a slope if the erosion characteristic and the size of the hydraulic event is known. Unfortunately the coupled slope stability model in this could not generate curved slip surfaces similar to those seen in the field. The most versatile of these modeling programs was the GeoStudio's SLOPE/W program. In this modeling package the phreatic surface determined from the pressure transducers could easily be entered. SLOPE/W was capable of creating and analyzing multiple slip surfaces within a user's specified range of possible slip surfaces. Due to the ability of this program to generate multiple failure surfaces it was used for the bulk of stability analysis preformed. Using information gathered from the pressure transducers 10 points in time were selected for analysis at each cross section.

As in the case of the Winooski River site a full cross sectional survey was not possible which would allow for easy computation of the changes in stream elevation across all of the sites. To compensate for this difficulty several water elevation surveys were conducted during base flow and one flow event exceeding 80% percentile flow for the stream. Linear interpolation and extrapolation was then used to offset the stream to the appropriate elevation at a given cross section to be modeled.

To date slope stability analysis has been performed on three sets of data collected for analysis of 90 streambank situations. Streambank geometries of the Lewis Creek Reach were modeled and then analyzed for stability using both the soils measured and index properties. Due to the highly variable strength characteristic found at the Winooski River reach slope stability analysis has only been performed using the determined index properties. Further analysis will be performed on the Winooski cross sections using section specific measured properties. Table5.1.

shows an overview of the results for the differences in satiability for the Lewis Creek site using index and measured soil properties

Using measured soil properties two sites in the study were found to have a factor of safety over one during at least one time step. With highest factor of safety observed for any given sight was 1.04 at the instrumented site on the Lewis Creek reach, indicating that this was the most stable site in the study. The remainder of the sites was found to have factors of safety to be less than one for all of the time steps analyzed. Using indexed properties found factors of safety consistently lower by approximately 20%. The factors of safety relating to water elevation for each analysis scheme can be seen in the following figures 5.1., 5.2., and 5.3.

6. Conclusions

1- Results from the BSTs and DST s showed that results obtained were comparable with each other for silty sands and sandy silts. Results from the BST obtained slightly higher values of the soils internal friction angles than those found by the DST which may affect the determination of a slopes stability.

2- The factors of safety observed at 11 of the 13 sites that were analyzed using classical soil mechanics were found to be less than one for all of the water levels that were observed, showing that using classical soil mechanics is inappropriate for the stream reaches and soil types analyzed.

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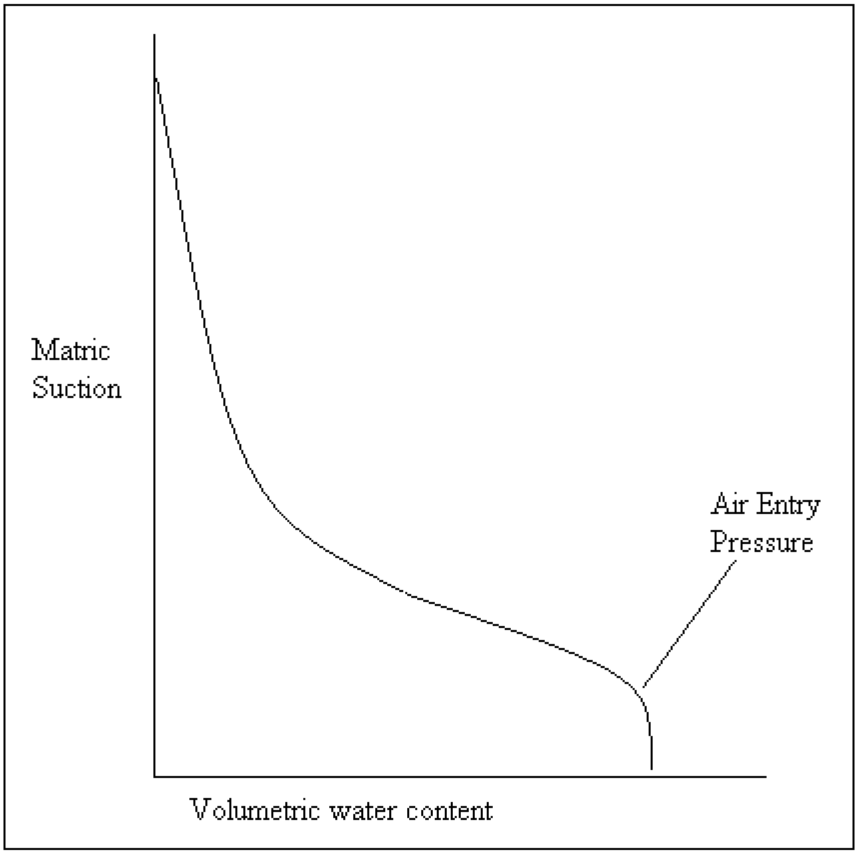
Matric
Suction
Air Entry
Pressure
Volumetric water content

Fig.2.1.

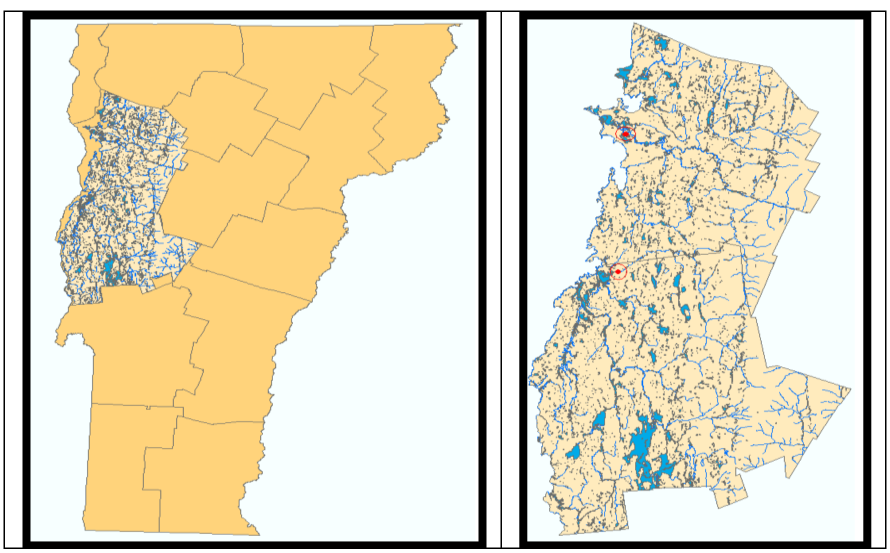

Fig.3.1.

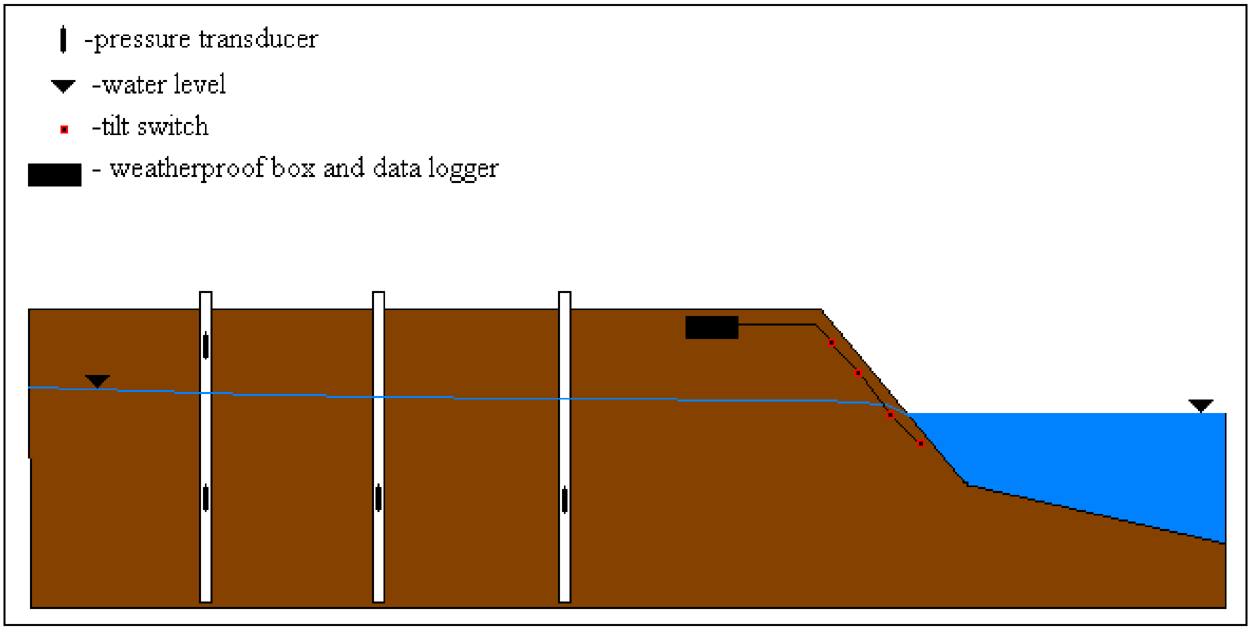
-pressure transducer
-water level
-tilt switch
- weatherproof box and data logger

Fig. 3.2

Fig. 4.1.

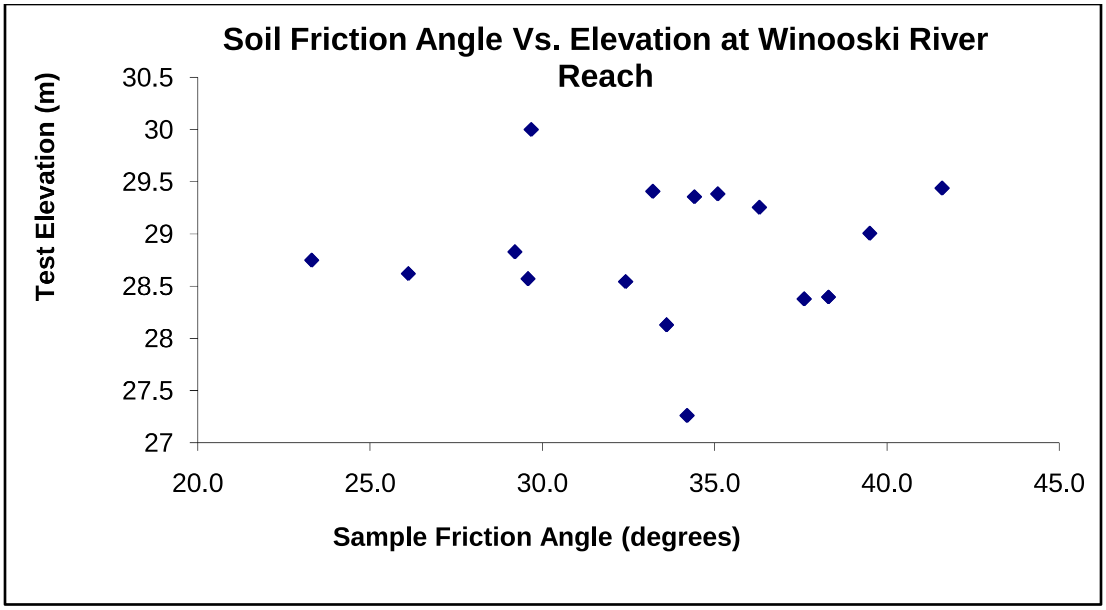
Soil Friction Angle Vs. Elevation at Winooski River Reach
Test Elevation (m)
30.5
30
29.5
29
28.5
28
27.5
27
20.0
25.0
30.0
35.0
40.0
45.0
Sample Friction Angle (degrees)

Fig. 4.2.

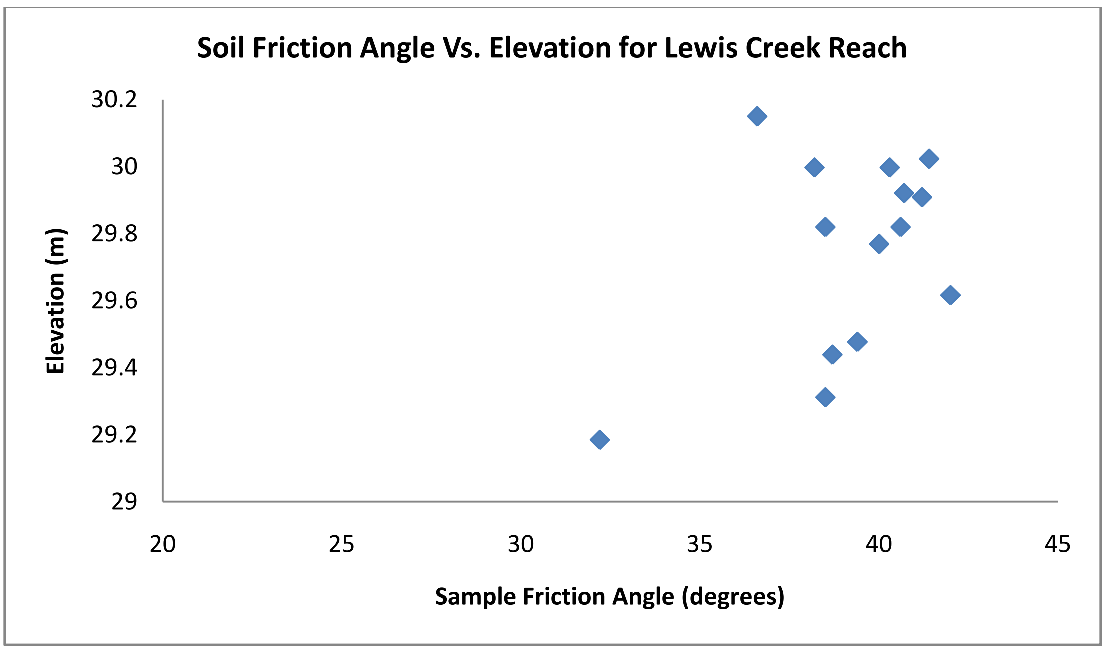
Soil Friction Angle Vs. Elevation for Lewis Creek Reach
Elevation (m)
30.2
30
29.8
29.6
29.4
29.2
29
20
25
30
35
40
45
Sample Friction Angle (degrees)

Fig. 4.3.

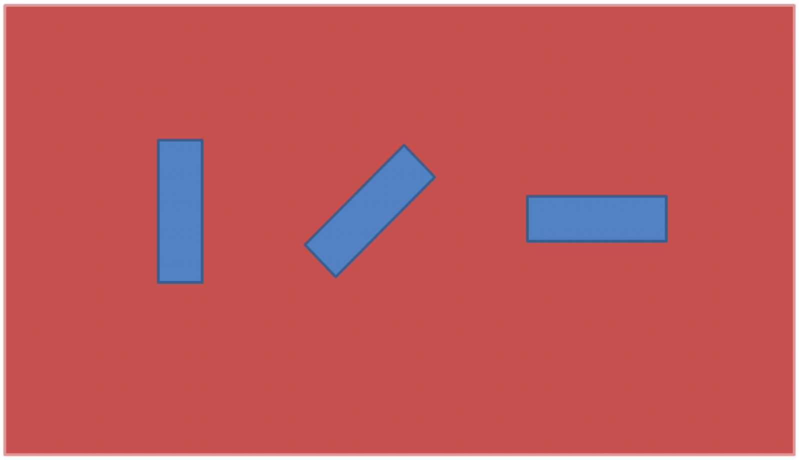

Fig. 4.4.

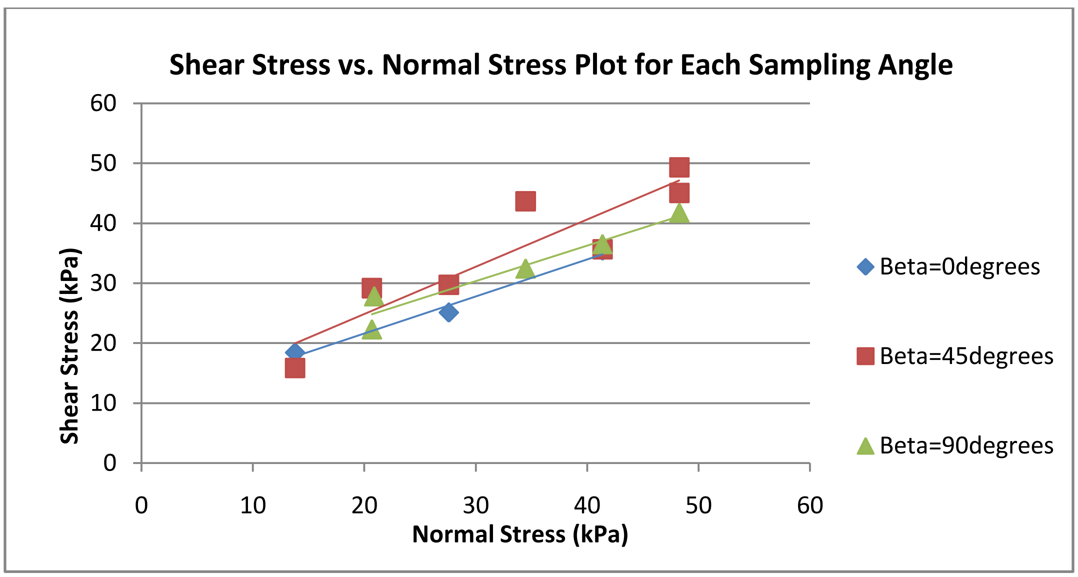
Shear Stress vs. Normal Stress Plot for Each Sampling Angle
Shear Stress (kPa)
60
50
40
30
20
10
0
0
10
20
30
40
50
60
Normal Stress (kPa)
Beta=0degrees
Beta=45degrees
Beta=90degrees

Fig 4.5.

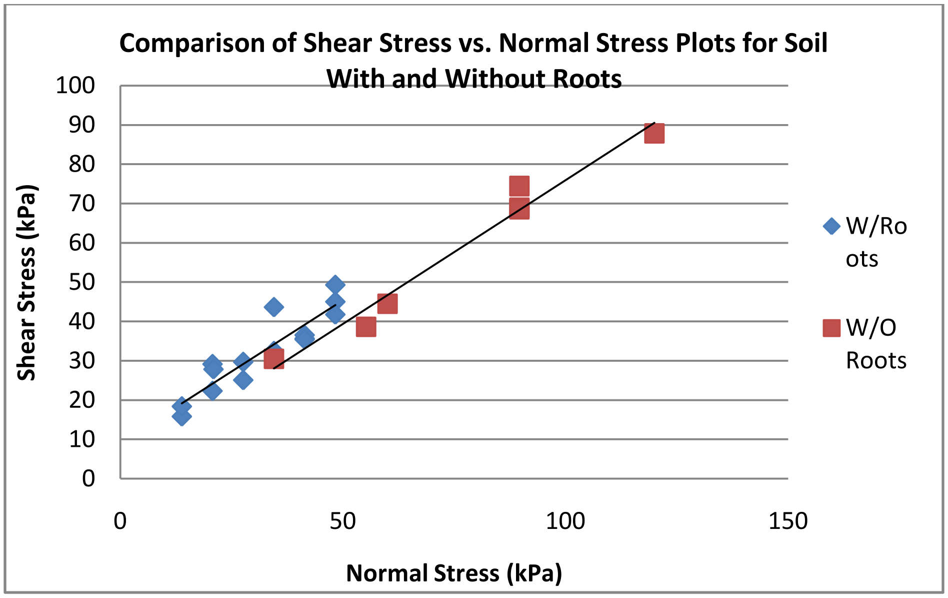
Comparison of Shear Stress vs. Normal Stress Plots for Soil With and Without Roots
Shear Stress (kPa)
Normal Stress (kPa)
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
W/Roots
W/O Roots

Fig 4.6.

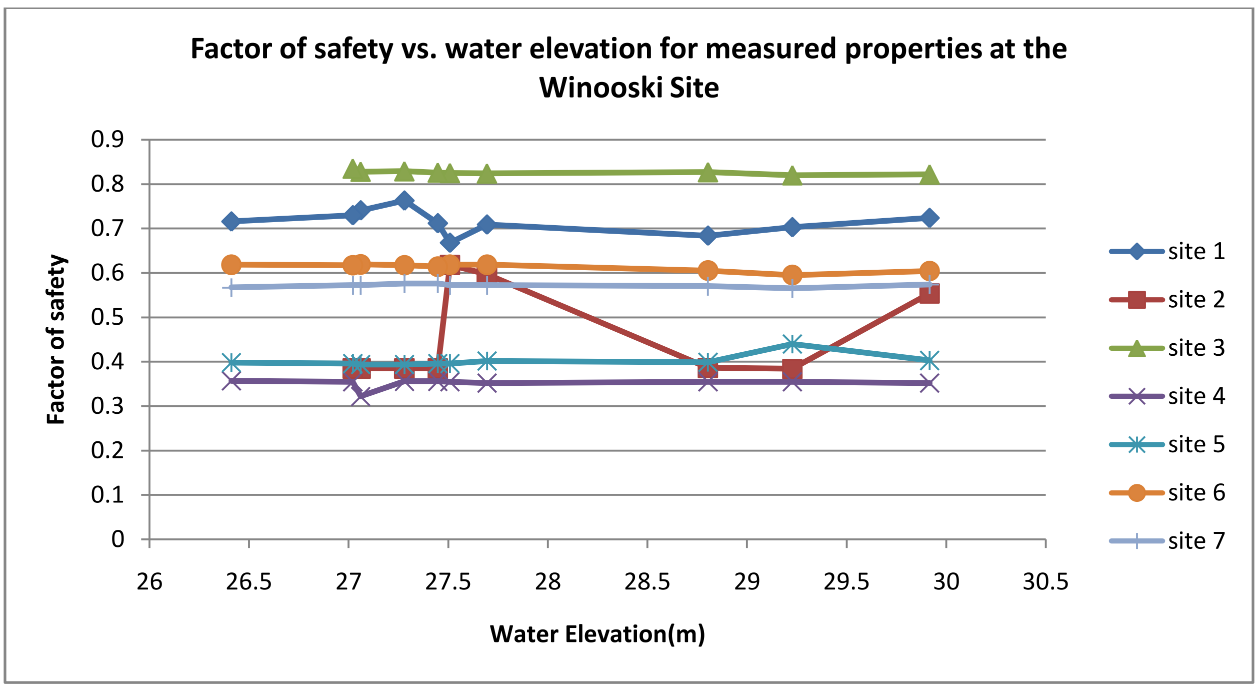
Factor of safety vs. water elevation for measured properties at the Winooski Site
Factor of safety
Water Elevation(m)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
26
26.5
27
27.5
28
28.5
29
29.5
30
30.5
site 1
site 2
site 3
site 4
site 5
site 6
site 7

Fig. 5.1.

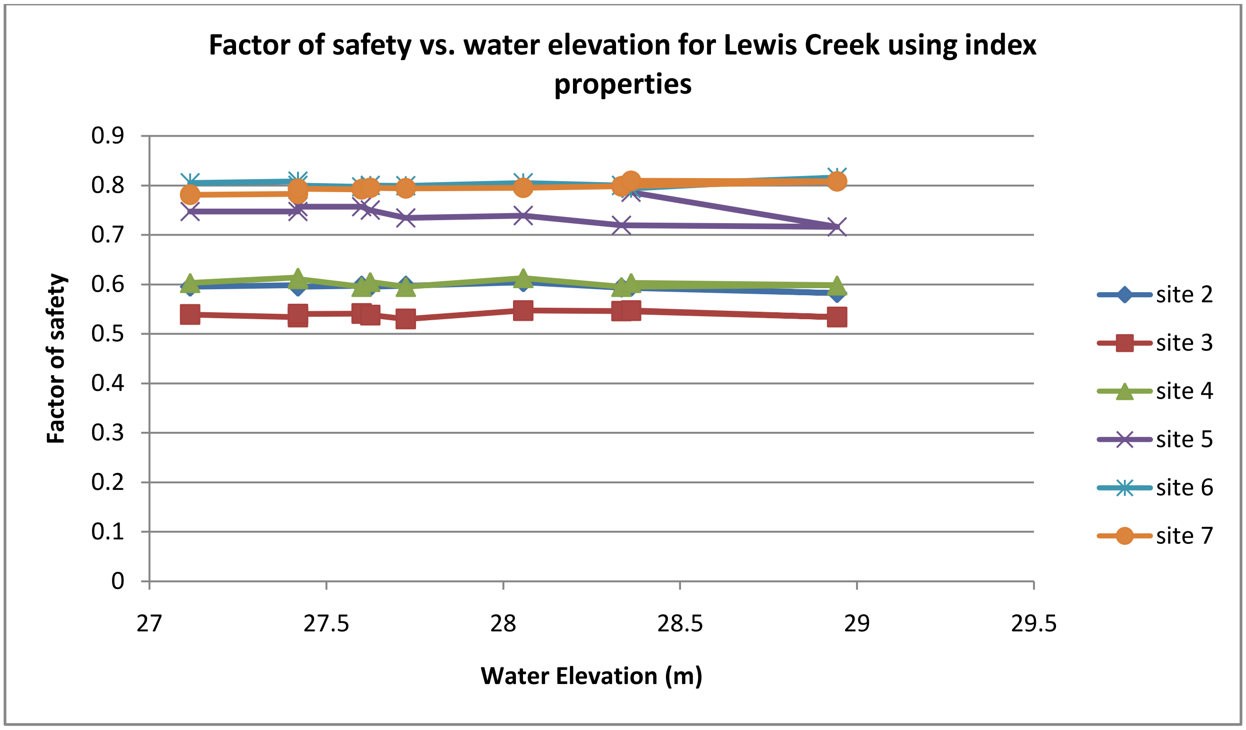
Factor of safety vs. water elevation for Lewis Creek using index properties
Factor of safety
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
27
27.5
28
28.5
29
29.5
Water Elevation (m)
site 2
site 3
site 4
site 5
site 6
site 7

Fig. 5.2.

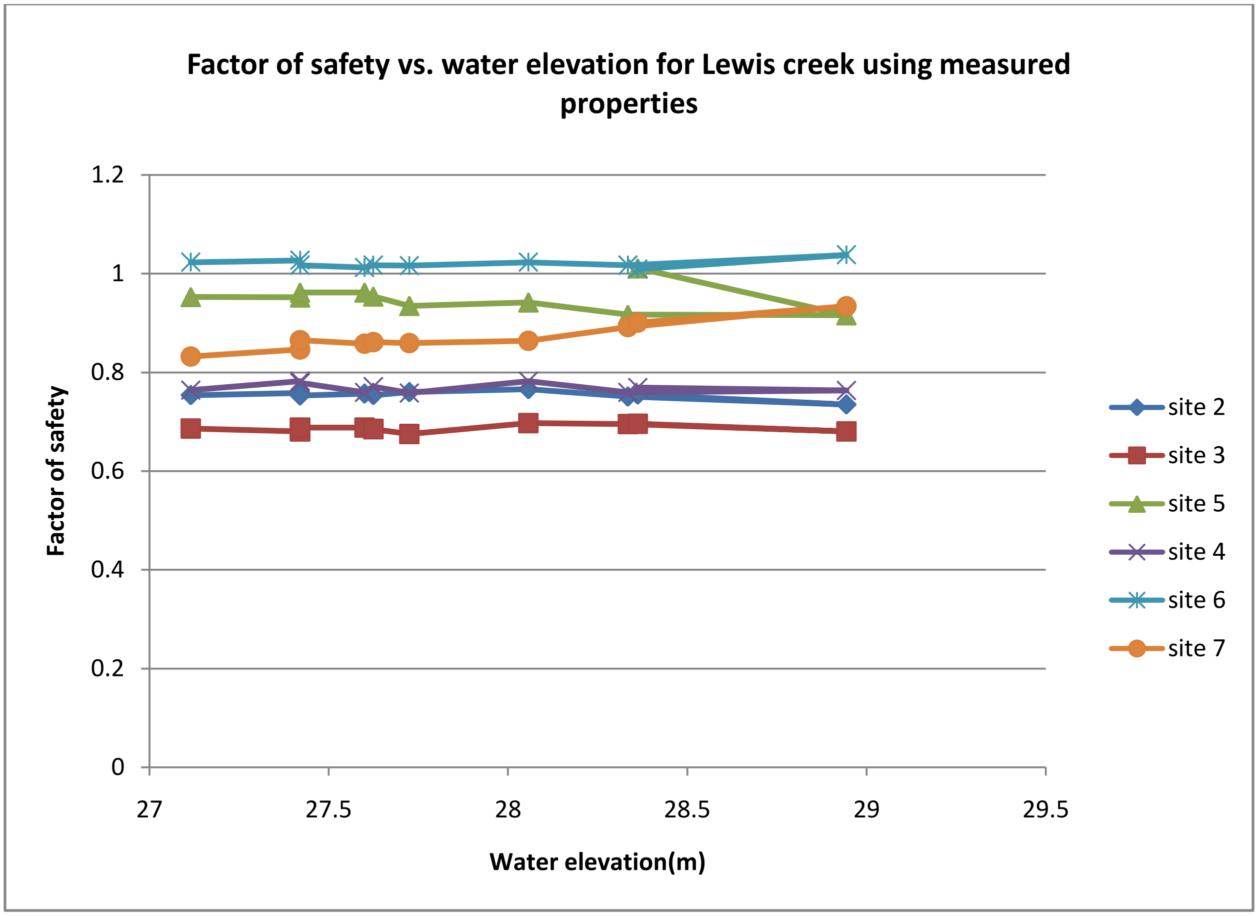

Fig 5.3.

Fig.2.1. Conceptual drawing of the soil water characteristic curve. Note air entry pressure is the transitional point between saturated and unsaturated conditions.

Fig.3.1. Location of the two studied reaches

Fig.3.2 Site instrumentation scheme used to collect water level and bank activity data

Fig.4.1. Typical stream bank selected for observation. This image of the Lewis Creek instrumented cross section displays stratification common to this reach.

Fig.4.2. Plot of soil friction angle across elevations encountered at the Winooski River reach.

Fig.4.3. Plot of soil friction angle across elevations encountered at the Lewis Creek Reach

Fig.4.4. Depiction of sample angles for root shear testing. Left to right angles of β= 0, 45, and 90 respectively.

Fig.4.5. Shear strength testing results for root impregnated samples collected at various angles. Tight clustering shows the limited affect of sample angle on shear strength.

Fig.4.6. Comparisons of soils shear strengths with roots present to those without. The vertical offset is representative of the additional cohesion provided by roots.

Fig.5.1. Determined factors of safety for the Winooski reach at each water elevation analyzed

Fig.5.2. Determined factors of safety for each of Lewis Creek sites at all 10 timesteps using indexed.

Fig.5.3. Determined factors of safety for each of the Lewis Creek sites for each time step using measured soil properties at each water elevation analyzed

Table 4.1. Site information and soil properties for both the Winooski River and Lewis Creek sites. Note Friction angles for Lewis Creek Site are only SM soils.

Site	Properties				
				Friction Angle (degrees)	
	USCS soil classes	Dry unit weight (kN/m^3)	Water Content	Range	Average
Winooski River	SM,ML,MH	17.1	0.3	22-42	33
Lewis Creek	SM,CH,	17.2	0.22	36-43	39.5

Table 5.1. Average factors of safety found at the Lewis Creek Site and percent difference between measured and index properties

	Bank ID					
	2	3	4	5	6	7
Measured Properties	0.75	0.69	0.77	0.95	1.02	0.87
Index Properties	0.6	0.54	0.6	0.75	0.8	0.79
Percent Difference	21%	22%	22%	22%	21%	9%