Report on Streambank Stability Assessment Techniques

Vermont Geological Survey Technical Report VGTR2007-1

George Springston
Research Associate,
Department of Earth and Environmental Sciences
Norwich University
158 Harmon Drive
Northfield, VT 05663

(802) 485-2734
gsprings@norwich.edu

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On the cover: Varved lacustrine silt exposed in an eroding bank of the Third Branch of the White River, Randolph, Vermont. Photo by George Springston.

Executive Summary

This report describes the processes that operate to break down stream banks, provides a set of procedures for gathering field data, and describes several ways of modeling the stability of slopes and the erosion of stream bank materials by flowing water.

Stream bank failures in Vermont can range from a few cubic meters of alluvium sloughing off of a low bank to, in the worst cases, thousands of cubic meters of material sliding off of a bank 30 meters or more in height, leading to stream channel disruption and massive damage. At least a few exceptional landslides have run out horizontally over distances three or more times their height.

Bank failure is the result of a combination of three sets of processes: Weathering, fluvial erosion, and slope failure. Weathering processes such as the leaching of soluble minerals and freeze-thaw action soften the materials. Fluvial erosion due to the shearing stress of flowing water against the bank removes particles or aggregates of particles and sweeps them downstream. A variety of slope failure types are found along Vermont Rivers, with the most common types being summarized below.

Banks composed of glacio-lacustrine or glacio-fluvial sand, silt, silty clay, or clay commonly fail as rotational slides that may cut deep into the bank. The result is often a complex rotational slide and flow, with the surfaces of blocks near the back of the slide tilted backward away from the river and at the base a more or less disaggregated mass of slide material that has flowed out toward the river.

In marked contrast to the rotational failures in stratified deposits, failures in dense till are often surprisingly shallow. The difference appears to lie in the lack of extensive shear surfaces in most tills. Although the material is very inhomogeneous, containing an incredible range of grain sizes from clay-size particles to boulders the size of a house, there are commonly no extensive surfaces that can serve as easy shear surfaces. The result is that a steep, eroding bank of fresh, dense till does not fail as a whole. Instead it tends to spall off shallow slabs a fraction of a meter thick. If there is weathered till above or to the sides, such material will fail as some combination of translational slides and flows, depending on how well the surface is tied together by vegetation. Translational slides commonly appear to require near-total saturation with ground water before failure is initiated.

Low banks less than 10 to 15 feet high that are composed predominantly of non-cohesive modern alluvium or older stream terrace deposits tend to fail as wedge or cantilever failures. Some of these slope failures may leave vegetated blocks on the bank, providing some temporary "armoring", at least at fairly low stream flows. However this arrangement of blocks is quite temporary--the next high stream flows will erode or sweep away the blocks and the erosion process will resume

Water is an important factor to consider in all types of slope failures. The influence can range from promoting weathering, to increasing the bulk density of the mass, to

decreasing the effective shear stress within the soil or on a discontinuity. Stream erosion acting at the base of a slope can oversteepen the slope, leading directly to failure. This is probably the single most important factor in destabilizing stream banks. Water that fills the pores of the soil can be under pressure and can reduce the effective stress sufficiently for a once-stable bank to collapse. Conversely, negative pore pressure in unsaturated soils may provide a certain amount of cohesion, thus increasing the strength of the soil (as long as it remains unsaturated). In summary, there is probably no more important a set of factors to be considered in slope stability studies than those related to surface and ground water.

Data sheets are provided for detailed field descriptions of the surficial materials and for slope failure characteristics. These sheets are intended to help the observer record the relevant observations regarding the geometry of the stream bank, the surficial materials underlying the slope, and the surface and ground water characteristics in order to have the necessary information for modeling both slope stability and fluvial erosion. As it is assumed that Phase I and II Geomorphic Assessments have already been completed, the focus here is on geologic and geotechnical characteristics rather than fluvial geomorphic characteristics.
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Because of the variety of surficial geologic materials underlying Vermont stream banks, no one geotechnical model is appropriate for all sites. All of the models used in this study are "limit equilibrium" models that calculate the balance between driving forces and resisting forces in order to generate a factor of safety. If this factor is significantly greater than 1.0, the slope is stable. If significantly below 1.0, it is unstable. A computerized infinite slope model is used for the shallow, translational slides, both chart-based and computerized slip circle methods can be used for deeper rotational slides in glacio-lacustrine and glacio-fluvial deposits, and computerized wedge and cantilever failure analysis models can be used for low banks composed predominantly of non-cohesive materials.

For low banks composed predominantly of non-cohesive materials the Bank Stability and Toe Erosion Model of the U.S.D.A. Agricultural Research Service is a good first step for calculating both stability and erosion on low banks. The program incorporates three types of slope failure: The two wedge failure types and one of the cantilever types. In addition to the slope stability modeling, it can be used to model the erosion of the toe and bank by fluvial shear. The stability and erosion parts of the model can be run repeatedly to simulate a sequence of fluvial erosion and slope failure events.

The models described in this report can provide useful information on slope stability and erosion by stream flow. However, this will only really become practical if additional data is obtained on Vermont stream bank materials.

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1. Introduction

Rivers are continually changing their channels, eating into one bank and adding their sediment to the other, so that frequently where there is a great bend, you see a high and steep bank or hill on one side which the river washes, and a broad meadow on the other. As the river eats into the hill, especially in freshets, it undermines the rocks, large and small, and they slide down alone or with the sand and soil to the water's edge. The river continues to eat into the hill, carrying away all the lighter parts, the sand and soil, to add to its meadows or islands somewhere, but leaves the rocks where they rested . . . Thus in the course of ages, the river wriggles in its bed until it feels comfortable.

Henry David Thoreau, Journal, March 24, 1855

Purpose

An understanding of stream bank erosion processes is critical to efforts to protect, restore, and manage stream channels and riparian corridors. Stream bank erosion is one of the many possible responses of a stream to variations in discharge, stream gradient, sediment volume, and sediment size. Although a natural process, the extent and rate of stream bank erosion can be accelerated by human alterations within a watershed. A greater understanding of the bank erosion processes operating at a site will help in making watershed management decisions that promote rather than detract from the overall stability of the river system.

The objective of this report is to provide a practical methodology for evaluating the erodibility and stability of individual sections of stream banks in Vermont. The report is intended to supplement the detailed stream assessment protocols developed by the Vermont Agency of Natural Resources (2003a, b, and c). It is assumed that a Phase 1 Watershed Assessment and a Phase 2 Rapid Stream Assessment (Vermont Agency of Natural Resources, 2003a and b) will already have been completed. The bank stability assessment will probably be undertaken in conjunction with components of the detailed Phase 3 Survey Assessment (Vermont Agency of Natural Resources (2003c).

This report is not intended to be used in slope stability analyses involving structures and facilities such as buildings, retaining walls, roads, railroads, etc. Appropriate geotechnical engineering information and advice should be obtained for projects involving such human constructions.

Structure of the Report

After a brief review of the causes of slope failures and a discussion of terminology, Chapter 2 is a review of the literature on stream bank stability in general and on field investigations in Vermont and surrounding areas. Chapter 3 provides an overview of slope failure classifications and of the relevant soil mechanics topics. Chapter 4 provides details on stream bank weathering and erosion processes--the processes that result in removal of material from the failed bank. Chapter 5 describes field assessment techniques. Chapter 6 describes the proposed bank stability analysis methods. These are intended to allow managers to evaluate the effects of processes such as down-cutting or lateral channel migration. Several example sites are described. A summary follows in Chapter 7 and recommendations for further research are given in Chapter 8. Field data sheets are provided in Appendices A and B, an introduction to relevant topics in soil mechanics is in Appendix C, typical geotechnical properties of Vermont soils are summarized in Appendix D, and a field procedure for geotechnical classification of soils is provided in Appendix E.

Note on Terminology

Perhaps because stream bank erosion studies have been undertaken by investigators trained in a variety of specialties, inconsistencies in terminology have arisen that may cause some confusion unless addressed.

The definitions of "landslide", "slope failure", "bank failure", and "mass failure" all overlap. A "landslide" is defined as "the movement of a mass of rock, debris, or earth down a slope" (Cruden, 1991) and even more broadly as "...a wide variety of mass-movement landforms and processes involving the downslope transport under gravitational influence, of soil and rock material *en masse*" (Neuendorf and others, 2005). Standard terminology for the many sub-types of landslide will be discussed in a later section. "Slope failure", as the term is used today, is essentially a synonym for "landslide", although Terzaghi and Peck (1948, p. 182) used it in the more restricted sense of a landslide in which the failure surface intersects the ground surface above the level of the toe. This usage is commonly encountered in geotechnical literature. The term "bank failure" is defined by Lawler and others (1997, p. 148) as "[c]ollapse of all or part of the bank *en masse*, in response to geotechnical instability processes." As used by Lawler and others (1997, p. 155), the term appears to be generally synonymous with "mass failure", although the Vermont stream assessment protocols restrict this term to a low eroding stream bank while reserving the term "mass failure" to a high eroding stream bank (Vermont Agency of Natural Resources, 2003b).

Given the prior and longstanding use of the term "landslide", the fact that all of the features that are currently termed "mass failure" fit accurately within the definition of "landslide", and the fact that the term "mass failure" is no more specific than "landslide", it is recommended that the term "mass failure" be abandoned.

The term "soil" as used in this report refers to the inorganic or organic materials that overlay hard bedrock. These are the natural, more or less unconsolidated surficial materials typically described as gravel, sand, silt, clay, till or diamict, peat, and muck. It seems reasonable that artificial fill that is composed of such material also be included in the definition. In soils mechanics terms, these soils may range from very soft to extremely stiff or from very loose to very dense. The term "soil" will be used to encompass the modifiers "debris" and "earth" used in the landslide classification described below. This definition is largely in the spirit of that used by Terzaghi and Peck (1948, p. 4), who defined soil as "...a natural aggregate of mineral grains that can be separated by such gentle mechanical means as agitation in water". Their definition had to be quite sweeping in order to include both the residual soils derived from weathered bedrock and some very diggable rocks, both of which are viewed as "soil" by engineers but would be classed as "rock" by geologists. Both of these definitions diverge widely from those used by soil scientists, who have traditionally focused on material that is suitable for plant growth and have rarely considered soil to extend deeper than a few feet. Here, soil will be the unconsolidated surficial material, regardless how thick it may be.

Acknowledgements

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Most of all, thanks to my dear wife, Rose Paul, for putting up with this long-standing obsession with mud and water, and cheering me on throughout.

2. Previous Work on Stream Bank Stability Evaluation

General Research on Stream Bank Processes

The processes controlling bank stability and erosion were not placed on a quantitative footing until well into the 20th century. Throughout the 19th century geologists and naturalists had observed and described the patterns of erosion and deposition along streams and engineers had been developing the principles of slope stability analysis to cope with slope failures along the new canals and railroads, but the understanding of stream bank processes remained largely descriptive. By the 1940's and 50's, an emphasis on the quantitative analysis of geomorphic processes had begun to unravel the conditions under which sediment was transported and deposited and the processes that contribute to or detract from the stability of a natural stream bank. A few examples include the work of Lane (1955) on channel stability, Horton (1945) on drainage basin evolution, and Wolman (1959) on bank erosion. For more detail on this period see Morisawa (1985). Wolman and others (1964) summarized the new knowledge of process-based bank failure and erosion mechanisms in their classic *Fluvial Processes in Geomorphology*, a work that remains useful to this day. More recent research is summarized in Lawler and others (1997) and Knighton (1998).

Following Lawler and others (1997), three sets of processes need to be considered: Weathering, fluvial erosion, and bank or slope failure ("mass failure" in their terminology). "Weathering" refers to the softening of freshly exposed material due to processes such as leaching of soluble minerals, pre-wetting, desiccation, and freeze-thaw action. "Fluvial erosion" is due to excess fluvial (hydraulic) shear stress applied to the stream bank.

It is of critical importance that the processes operating on a particular stream bank be placed in the proper context. For example, a particular bank may not fail unless scour at the base increases the effective height of the bank to a critical value. If the reach is aggrading, such an increase in height will be unlikely to occur. Therefore, it is often necessary to look at the characteristics of the stream reach, and indeed the watershed as a whole. The beginnings of this approach can be found in the summary of fluvial geomorphology by Leopold and others (1964). These concepts are greatly elaborated by Rosgen (1996) and in the stream assessment protocols developed by the Vermont Agency of Natural Resources (2003a, b, and c).

Field Studies in Vermont and Surrounding Regions

Although the naturalists and geologists working in Vermont in the nineteenth century certainly examined some eroding stream banks, such investigations appear to have focused solely on the underlying materials and not on the mechanisms or styles of bank failure. Catastrophic bank erosion at New Haven during the flood of 1830 is mentioned in Zadock Thompson's Natural History of Vermont (1853) but few details are given. The *Report on the Geology of Vermont* (Hitchcock and others, 1861) contains numerous valley cross sections and descriptions of terraces and underlying deposits and some references to shifting of stream and river channels (apparently both meandering and avulsions), but there are no detailed descriptions of styles of stream bank failure.

On August 20, 1901, heavy antecedent rainfall and rains of at least 2 to 3.42 inches resulted in three large and 8 small debris flows or avalanches on the east side of Mt. Greylock in western Massachusetts (Cleland, 1902). These landslides led to heavy deposition on alluvial fans in the valleys below. A description in the same article of a landslide at Briggsville is more obscure, but it sounds as if it was deep-seated rotational sliding in lacustrine "clay".

An abstract published in 1916 gives a general description of landslides formed within laminated clay and silt deposits formed within glacial Lake Bascom in the lowlands of the Hoosic River in southwestern Vermont and adjacent parts of New York and Massachusetts (Taylor, 1916). Some were described as being ancient while others were active due to ongoing toe erosion by streams.

A tremendous stream bank failure occurred on the east bank of the Winooski River on June 1, 1914 and was described by Jacobs (1916). The bank was reported to have been 50 feet in height and composed of "...about fifty feet of nearly pure yellow sand and underlaid by ten to fifteen feet of the same sand but containing small boulders and pebbles, the whole resting on a bed of stiff blue clay of unknown depth" (Jacobs, 1916, p. 221). The bank failure took the form of a deep, rotational slide that extended for almost a thousand feet with a toe that extended halfway across the 300 to 400 foot wide river.

Newland (1916) gives excellent descriptions of several varieties of landslides in surficial deposits in the Hudson valley of New York, many of which are associated with stream bank erosion. He discusses surface creep phenomena, earth slumps and flows in clays and silts, earth or debris slides (mostly rotational rather than translational), and retrogressive flows in apparently sensitive clays. He clearly recognized the importance of pre-existing weaknesses in the materials. The report is still worth reading for the physical descriptions of the landslides, particularly the August 2, 1915 landslide at the Knickerbocker Portland Cement Company plant on Claverack Creek outside of Hudson, New York. A low-angle, rapid earth flow (retrogressive?) wrecked a power house, killed five workers, and completely blocked the creek, the toe of the slide raising the creek bed 25 feet into the air. The site appears to have been on the outside of an eroding bend of the creek. The slope prior to failure was about 30 to 40 feet above the creek and was composed of several feet of silt and sand over varved clay. At a depth of 75 to 100 feet the clay is underlain by morainal gravels. Causes appear to be artificial loading, stream bank erosion, and heavy rainfall. Based on the characteristics of the slide, this clay may have been quite sensitive and lost all shear strength once failure began. An analysis of the failure mechanism in terms of excess pore pressure is given by Terzaghi (1950) and is summarized in Chapter 3.

Numerous landslides along Vermont streams were examined by Antevs (1922, 1928) as part of an extensive regional study of annual layering (varves) in glacial lake deposits. His publications include small scale location maps and detailed stratigraphic descriptions, but no details on the geometry of the landslides. Although his study focused on banks dominated by lacustrine deposits, a few did include one or more layers of till. This work remains useful as the first to give some idea of the distribution of unstable stream banks in the larger valley bottoms.

The photographs and accounts of the 1927 flood are full of evidence of widespread bank erosion, the most spectacular examples being at two sites: Gaysville in the Town of Stockbridge in the White River valley and at Cavendish in the Black River valley. Popular accounts of these and many other sites are given in Luther Johnson's *Vermont in Floodtime* (1928) and a brief report on the Cavendish site was made in *Science* by Jacobs (1927). Numerous excellent photos of these sites can be found in special issues of *The Vermonter* from 1927 and 1928 (Figure 2.1).

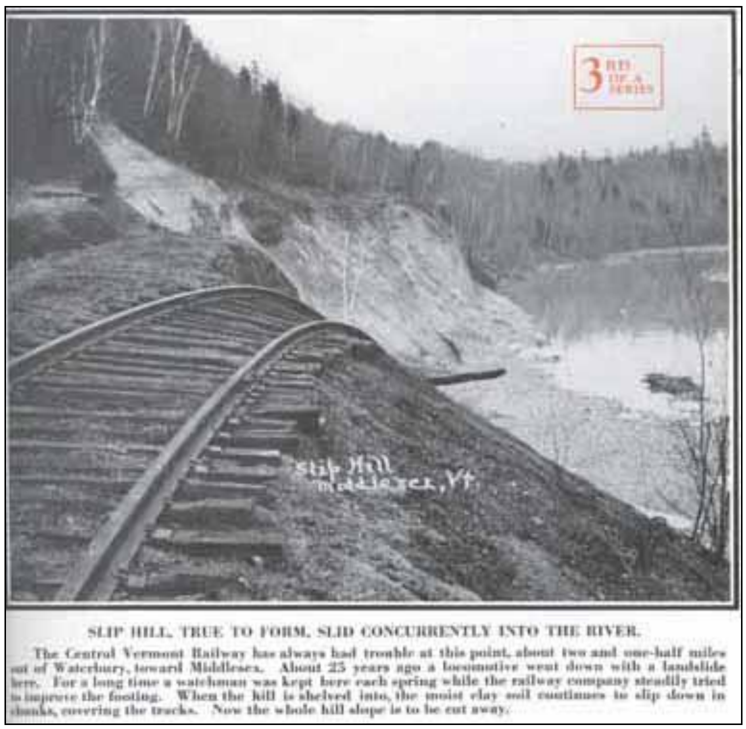

Figure 2.1. Cover of a 1928 issue of *The Vermonter* showing catastrophic bank erosion in Middlesex due to the 1927 flood.

Many eroding banks were doubtless examined as part of a statewide effort in the 1950s and 60s to map the surficial geologic deposits of Vermont (summarized in Stewart and MacClintock, 1969 and Doll, 1970). However, as with the nineteenth century investigations, little mention was made of eroding banks and landslides, although they did note that slope failures are abundant on the stream banks of the Connecticut River valley where these are underlain by varved lacustrine deposits (Stewart and MacClintock, 1969, p. 102). Based on the descriptions of till fabric locations in Stewart and MacClintock (1969) many of the sites were clearly at eroded stream banks, some of which still appear to be actively eroding.

A large landslide occurred in marine and lacustrine clay on the outside (south) bank of a meander bend on Otter Creek in Weybridge on February 15, 1956. A general description is given by Stewart (1973) and Solomon (1975) undertook a detailed analysis. The slide was a complex, rapid, retrogressive, rotational slide. It had a head scarp about 430 meters long and extended 75 to 140 meters back from the river with a volume of around 46,450 cubic meters. The stream bank at the time of failure appears to have been not more than around 9 to 10 meters high. A section of Vermont Route 23 was destroyed, a passing car was engulfed (evidently the driver escaped without injury) and a house was threatened and had to be moved. The bed of Otter Creek was heaved up and at least some impoundment of water occurred upstream of the slide. Solomon interprets the upper 3 to 6 meters of the deposit as marine or estuarine (Champlain Sea) and the lower part as lacustrine (glacial Lake Vermont). Solomon speculated that the material may actually be "sensitive" as defined below in Chapter 3, but his liquidity index data do not support this interpretation (the lack of a thick Champlain Sea section also makes this less likely). His geotechnical data is discussed further in Appendix C. The triggers of the slide appear to have been high pore pressure due to snow melt and oversteepening due to toe erosion. According to Solomon there was earlier landslide activity at the site, including significant slumping in 1952. The site is still on the outside of an active meander bend and landslide activity has continued (Jeff Munroe, Middlebury College Department of Geology, personal communication, 2004).

The extent of the Weybridge slide should be noted carefully: The erosion of a stream bank that was only a few meters in height resulted in a bank failure that rapidly extended up to 140 meters back from the edge of the river. Whether or not this was a "sensitive" material in a strict geotechnical definition, this is a far deeper "reach" than would normally be expected from the failure of a bank of this height. Analyses of stream banks in similar materials will need to be made quite carefully.

Slavin (1977) studied bank erosion processes on the Browns River in northwestern Vermont. He examined six streamside sites in the Essex Center quadrangle, all of which were in alluvial silts, sands, and gravels. Layers of coarse-grained alluvium at the test sites had lower shear strength and were more erodible than overlying finer, more cohesive materials, and were thus more susceptible to erosion by either undercutting (direct entrainment of particles by fluvial shear) or by subaqueous flows out into the river. Bank collapse was mostly as forward-rotating slab failures. Where these slabs were vegetated and remained on the outer parts of the bank after collapse, they then served to temporarily armor the bank. Not until the slab was eroded or torn away would erosion again proceed. He also attempted to use a pump and sprinkler to saturate the soil at one of the sites in order to examine the role of increased pore pressure in reducing the stability of the banks. However, the overall hydraulic conductivity of the bank at his chosen site was extremely high due to fractures and animal burrows and the water flowed out into the river as fast as it was pumped onto the bank. In his conclusions he stressed that bank erosion at these sites did not appear to be a simple function of stream flow (a process-response model). Instead, he concluded, following the idea of Schumm (1973), that one or more geomorphic thresholds needed to be exceeded before bank erosion would occur. That is, besides high streamflow, one or more antecedent factors or conditions, such as removal of any armoring slabs, had to be in place before a given section of bank would erode.

A geo-archaeological study in the Missisquoi River valley in northwestern Vermont by Brackenridge and others (1988) provides useful insights into how the dominant channel alteration processes can change over time in response to factors such as climate, changing base level, and changing land use. The authors looked at the long-term vertical incision history and lateral migration rates in alluvial deposits as revealed by detailed trenching and dating of deposits and surfaces by a combination of radiocarbon analysis and interpretation of artifacts. Isostatic rebound of the crust raised the site above the level of the Champlain Sea bed at some time between 12,000 and 10,000 14C yr B.P., initiating rapid vertical incision. After initially high rates of vertical channel incision of >1 meter/100 yr prior to 8,000 14C yr B.P., the incision rate dropped to around 0.01 m/100 yr and lateral migration became the dominant process. The overall lateral migration rate since that time has been about 1 m/100 yr but it has been highly variable, with long stretches of little migration being punctuated with periods of higher migration rate. Post settlement deposits show an increase in lateral migration rate. Note, however, that there does not appear to be any indication of an episode of aggradation associated with the post-settlement deposits and that lateral migration has remained the dominant process. The authors argue that the infrequent large floods with recurrence intervals on the order of hundreds or thousands of years may play an important role in destabilizing the channel and initiating periods of rapid channel adjustment. Detailed geo-archeological studies such as this can provide important data for determining whether or not bed aggradation or degradation are likely to occur at a site.

A field trip guide by Bierman and others (1999) illustrates bank erosion processes at several sites in northwestern Vermont. At Stop 1, on Town Line Brook in Winooski and Colchester, a brook is incising into fluvial sand and gravel and Champlain Sea silt with sandy interbeds. Incision in the gully bottom leads to episodic landsliding, which has been documented at a pin line to vary from several cm/yr to > 1 meter per year. The authors discuss slope failure being driven by failure of the silt by washing out of sand interbeds or failure due to high pore pressure, both processes that seem likely to operate at such a site. However, the headcutting that this author has observed in the base of the gully is also adding to the instability by increasing the height of the slope. Stop 2 at Mill Brook near West Bolton illustrates the dramatic effect on steep-gradient stream channels caused by a flash flood and the resulting debris jams. Stop 4 is an actively eroding gully and the resulting alluvial fan in Stowe. The gully erosion is occurring through piping rather than the more common surface erosion. Stop 5 is a debris flow in Smugglers Notch near the Cambridge/Stowe town line. Although most debris flow channels are not perennial stream courses, some of the larger debris flow paths may correspond to stream channels, and the process of debris flow transport is of prime importance in bringing coarse sediment into the first and second order channels. The final stop is at the site of the dramatic 1999 Jeffersonville landslide and is discussed in some detail in the following paragraph.

Between April 11 and July 4, 1999, three landslides on a 50 meter high bank on the east side of the Brewster River carried at least 27,000 cubic meters of material down to and across the river toward the outskirts of the Village of Jeffersonville (unpublished memo from Vermont Geological Survey, 1999; Bierman and others, 1999; Nichols and others, 2004). The slope is composed of lacustrine varved silt and clay, overlain by lacustrine varved fine sand and clay, which is topped by a few meters of fluvial sand and gravel. The upper fluvial materials were probably deposited by an earlier incarnation of the Brewster River soon after drainage of the pro-glacial lake but prior to downcutting by the river through the underlying lacustrine materials. The maximum runout of the slides was greater than 150 meters, with the second slide extending so far that mud was splashed onto two houses. The slope retreat at the top of the bank threatened to topple a house, which was subsequently demolished. The main cause of the slope instability was certainly toe erosion by the river, a process that has been ongoing at the site for many decades, as evidenced by an earlier landslide nearby in the 1950s and records of a landslide scar at the site in the 1920s. Bierman and others (1999) and Nichols and others (2001) discuss ideas about the final triggering of the slide. The determination of the cause is complicated by the fact that the failures occurred during a dry season (heavy rains last occurred in the summer of 1998). The extensive runout at the Jeffersonville slide (over 3 times the height of the slope) is very important to keep in mind when evaluating the stability of high banks.

A U.S. Geological Survey study of slope stability issues in Vermont, undertaken in cooperation with the Vermont Geological Survey, resulted in several publications that contain useful information on bank stability. Much of this work is summarized in Baskerville and others (1993) and Baskerville and Ohlmacher (2001). Of particular note is the cluster of at least four debris avalanches that occurred on Dorset Mountain on August 10, 1976. Such events, although comparatively rare in Vermont, have the power to cause tremendous damage. Where they have occurred in stream valleys, the signs may be discernible for many decades thereafter. Note that the Dorset slides extended up to 4.2 kilometers from their source areas. Similar debris avalanches or debris flows also swept down the valleys of Mill Brook in Fayston in 1827 and Slide Brook in Fayston in 1897 (Baskerville and others, 1993).

Several studies of debris flows and/or debris avalanches in the mountainous terrain of surrounding states have been undertaken in recent decades, including Flaccus (1958), Kull and Magilligan (1994), and Milender (2004) in New Hampshire, Bogucki (1977) in the Adirondacks, and Dethier and others (1992) on Mount Greylock in Massachusetts.

As part of a pilot study of fluvial geomorphology in the Great Brook watershed in Plainfield, bank materials were rated according to their apparent resistance to erosion or erodibility (Barg and Springston, 2001; Springston and Barg, 2001). Using geotechnical terminology, the noncohesive materials were ranked according to increasing relative density and the cohesive materials were organized in order of increasing plasticity, stiffness, and stickiness, with all of the cohesive materials being less erodible than the non-cohesive ones. As is the case with the Vermont Bank Erosion Hazard Index described in the next section, this classification does not address the correct factors for putting bank erodibility on a quantitative basis. See Chapter 4 for discussions of the concepts of critical shear stress and erodibility.

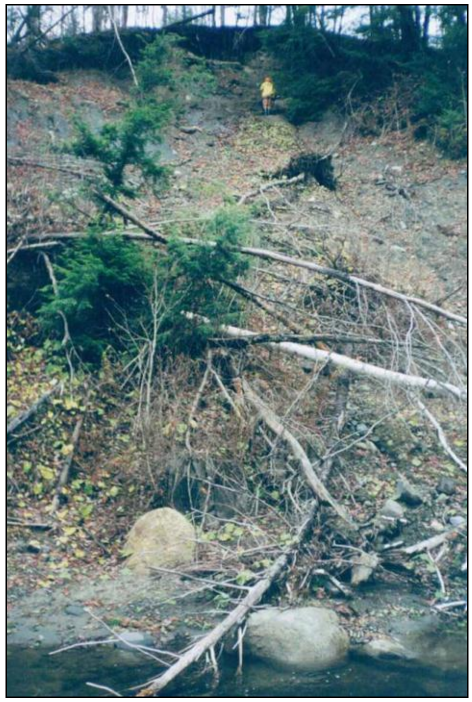

Figure 2.2. Large landslide in dense till on Great Brook, Plainfield. Site GB-179. Photo by Lori Barg.

Bank Stability as a Component of Stream Reach Assessment Methods

A variety of methods have been developed in recent decades to systematize data collection on stream channel geomorphology and processes. Many of these methods include bank erodibility and stability assessment components. One of the earliest, the "Stream Reach Inventory and Channel Stability Evaluation" of Pfankuch (1978) has seen widespread use and although it has been rendered obsolete as a quantitative assessment tool by successors, it remains useful as a concise overview of the factors that influence stability of a channel reach or a specific portion of a stream bank. The more recent assessment methodologies of Rosgen (1996) and the Rapid Geomorphic Assessment of the Center for Watershed Protection and others (1999) include more detailed assessments of bank conditions that begin to produce data on the erodibility of the outer material of the bank, but they still do not produce quantitative data on the deep geotechnical stability of the bank.

The Vermont Bank Erosion Hazard Index (Agency of Natural Resources (2003b, Section 3.5 and Appendix N) was developed as part of a comprehensive geomorphic assessment methodology for Vermont streams. It is an attempt to quantify the likelihood of further erosion based on some of the observable bank characteristics, including the bank height, the height of bankfull flow of the stream, rooting depth, a weighted index of root density, and a weighted index of bank angle. Although these factors certainly are related to the issue of bank stability, this attempt at a simple index confounds factors related to stream erosion due to excess hydraulic shear stress with factors influencing the deep stability of the bank as a whole and the index does not appear to yield accurate predictions of bank erosion. The procedures outlined in this report are intended to replace this index.

Progress has been made in recent years in developing process-based models that can correctly calculate the direct erosion and entrainment of sediment particles from the bank by an excess fluvial shear stress and then calculate the stability of the bank after an increment of such erosion, taking into account changing stream flow and soil pore water conditions. Although no single model has been developed that can be applied to all of the bank materials and slope failure types encountered in Vermont, useful steps in this direction have been taken by Rinaldi and others (2004), Simon and Collison (2002), Simon and others (2001), and Simon and others (2003).

Rinaldi and others provide a particularly attractive methodology that combines modeling of hydraulic shear stress and the resulting entrainment of particles from the bank with sophisticated computer modeling of soil pore water conditions and slope stability. Animations of the results of this analysis can be viewed at <http://www.dicea.unifi.it/massimo.rinaldi/_private/simulations_it.htm>. The downside to this approach is that it is extremely data-intensive, requiring detailed geotechnical data and continuous monitoring of stream flow and soil pore water conditions.

The Bank Stability and Toe Erosion Model (Version 4.2) of Simon and others (2001) allows incorporation of geotechnical data and generalized stream flow and soil pore water conditions, but the models are somewhat simplified, assuming horizontal piezometric surfaces, and a choice of a planar failure or a cantilever failure. Rotational failures cannot currently be modeled with this program. If this method turns out to be applicable to some of the bank conditions in Vermont, then the relatively simple spreadsheet calculations could become an effective tool for bank assessment. Examples of the output from this model are given in Chapter 6.

3. Bank Stability Principles

If a landslide comes as a surprise to the eyewitnesses,
it would be more accurate to say that the observers
failed to detect the phenomena which preceded the slide.
Karl Terzaghi, 1950

General Statement

This chapter will describe the types of slope failures common on eroding stream banks in Vermont. Because of the complex influence of vegetation on bank stability and bank erosion, the topics are discussed together here, even though parts are perhaps more properly discussed in Chapter 4. For users unfamiliar with soil mechanics, discussions of shear strength of soils, the effective stress concept, apparent cohesion in unsaturated soils, and the effects of fractures and other anisotropies in soil are provided in Appendix C. Bank stability modeling will be described in Chapter 6.

The evaluation of the stability of natural and artificial slopes has been the subject of a vast array of engineering and geological literature. However, the broad principles necessary for understanding the stability of stream banks are well-summarized in a few recent references. A very simple overview of the principles of stability evaluations of natural slopes (including stream banks) is given in Goudie (1990, Chapter 4). Turner and Schuster (1996) provide a very complete overview of landslide analysis, including detailed summaries of landslide types, field investigation methods, and strength and stability analysis. The handbook by Hall and others (1994) includes detailed procedures for slope stability evaluations in U.S. National Forests. The chapters on "Strength and Behavior of Soils" and "Ground Water Fundamentals" provide a good introduction to the soil mechanics necessary for understanding slope failures. The primary slope stability references listed above can be supplemented by general texts on soil mechanics such as Bowles (1984) and Lambe and Whitman (1969).

Hammond and others (1992), besides giving clear and detailed introduction to the use of the infinite slope equation, contains an excellent discussion of the statistical variability of soil parameters, an extensive compilation of geotechnical parameters for slope modeling, and a useful literature review on the contribution of roots to soil strength. Although this manual is designed specifically for the LISA and DLISA computer models, it should be reviewed by anyone using the infinite slope model.

A recent overview of the state of landslide science is given in Sidle and Ochiai (2006), which includes analysis of the economic effects, types of landslides, natural and human factors, and hazard and risk assessment. Although not focused on stream bank stability *per se*, there is much useful information.

Classification of Bank Failures

The slope failures observed on Vermont stream banks can be classified using the system of Varnes (1978) and Cruden and Varnes (1996). Figure 3.1 shows the five main types of movement: Falls topples, slides, spreads, and flows. All except the spreads are commonly seen in Vermont. Slope failures composed entirely of coarse material are prefixed with "rock", those with 20 to 80% of the material coarser than sand are prefixed with "debris", and those with less than 20% coarser than sand are prefixed with "earth". Several states of activity are recognized: Active slides have moved within at least the last few months, slides haven't moved in the last year or so are dormant, and slides that no longer are likely to move due to a change in geomorphic or climatic conditions are relict. Further details related to activity, rate of movement, water content, and pattern of movement (retrogressive, etc.) are given in Cruden and Varnes (1996). A simplified summary of slope movements is given in Table 3.1. The information gathered with the data sheets in Appendices A and B will allow a detailed classification using the Cruden and Varnes classification. Common terms for the components of a complex rotational slide and flow are shown in Figure 3.2.

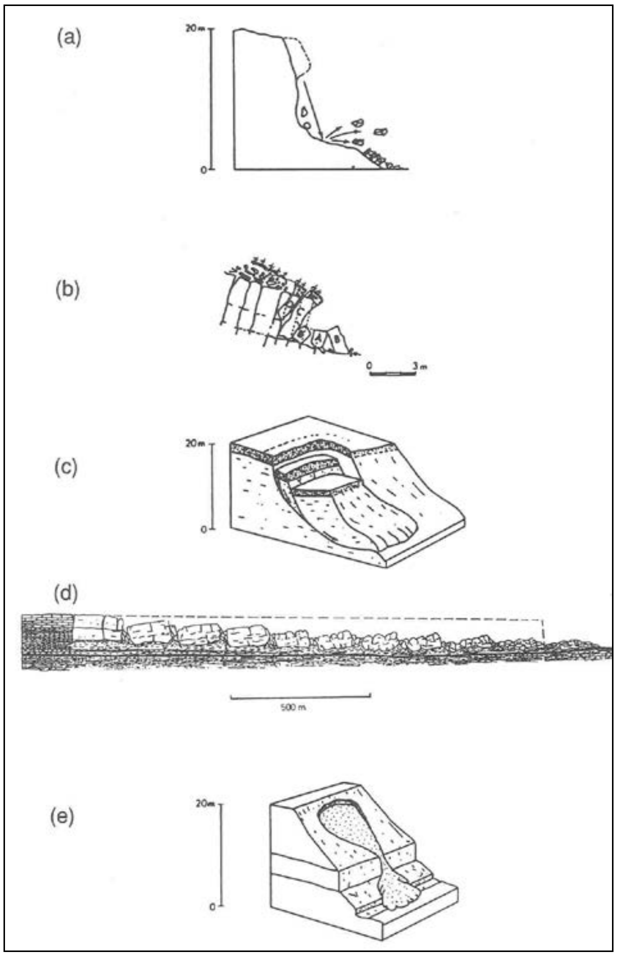

Figure 3.1. Types of landslide movement. a. Fall. b. Topple. c. Slide. d. Spread. e. Flow. All except for spreads are common on Vermont stream banks. From Cruden and Varnes (1996, Figure 3-19).

Creep is a "barely perceptible and nonaccelerating downslope movement" (Bloom, 1998, p. 174). The process operates to varying degrees on almost all slopes, whether of rock or soil. Creep may affect the upper few centimeters of soil on a bank or operate at depths of one or perhaps several meters. Creep may be an important process in the weakening of some slopes, particularly steep, wooded slopes on weathered till. Note that creep movements tend to occur at a steady (but perhaps seasonal) rate. If the movement is accelerating over time it is perhaps an indication of impending slope failure, as described below. See Bloom (1998) for more details.

Table 3.1. Simplified classification of slope movement types. Modified from Varnes (1978). Types common on Vermont stream banks are in bold.

Type of Movement	Type of Material		
	Bedrock	Engineering Soils	
		Predominantly coarse	Predominantly fine
Falls	**Rock fall**	Debris fall	**Earth fall**
Topples	**Rock topple**	Debris topple	**Earth topple**
Slides	**Rock slide**	**Debris slide**	**Earth slide**
Spreads	Rock spread	Debris spread	Earth spread
Flows		**Debris flow**	**Earth flow**
Complex	**Combinations of two or more types of movement**		
Creep	**Several types**		

Although fresh landslides may have forms that correspond reasonably well to Figures 3.1 through 3.3, the slides that occur on stream banks tend to be rapidly altered by the stream at the base, by ground water sapping or piping, by surface runoff down from the top, and by surface earth flows. The end result of a bank failure that is more than a few months old may be somewhat difficult to classify. However, if there are other nearby slides in similar materials that are at different stages in their evolution, these may be used to interpret the older slides.

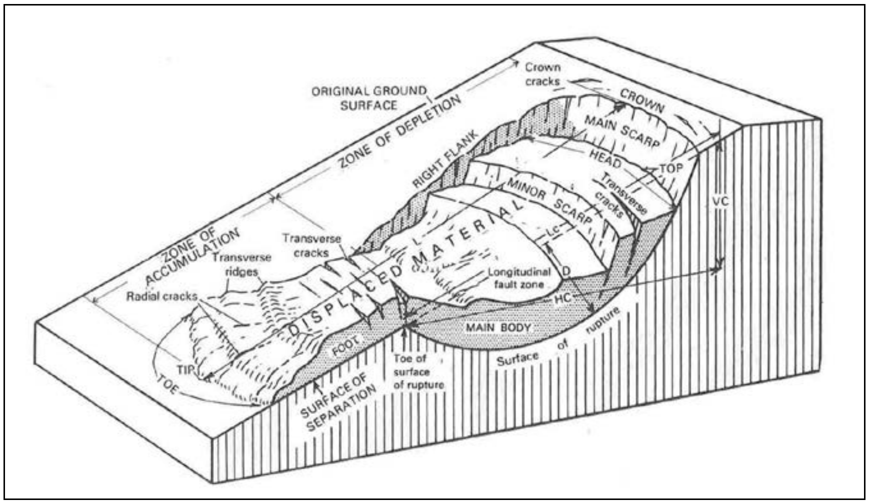

Figure 3.2. Generalized complex rotational slide/flow showing principal features. Stream bank failures with this overall form are common on clayey to sandy lacustrine deposits throughout Vermont. In stream bank settings the displaced material is commonly eroded away by stream flow to a greater or lesser extent. From Cruden and Varnes (1996, Figure 3-3).

The most common types of landslides in Vermont are the slides, which take two general forms as shown in Figure 3.3; rotational and translational. The translational slides generally occur on failing banks underlain be weathered, dense till while the rotational slides are more common on unstable slopes underlain by sandy to clayey lacustrine deposits. Both rotational and translational failures imply that the material has internal cohesion, otherwise the material would disintegrate into some sort of flow. They are described in more detail in the following sections.

Rotational Slides

Rotational slides are common in the stratified deposits that are widespread in the larger stream valleys of Vermont, especially the cohesive glaciolacustrine silts, silty clays, and clays. These slides range in size from a few meters up to the extensive bank failure that occurred in 1910 on the Winooski River, extending for about 300 meters along the bank (described in Chapter 2).

The characteristic form of the rotational slide, as shown in Figure 3.2, has a curving fracture or shear surface that intersects the ground either on the bank or behind the top of the bank. It is then seen to curve down to a bed or lamination either within the bank or at the base. The shear may then extend all the way out to the free face or, more commonly, curve upward to take a path of least resistance to the free surface. Slide material often undergoes considerable deformation during failure and as the displaced material moves downward, the lower parts of this must, if they stay at least partly together, ride up over

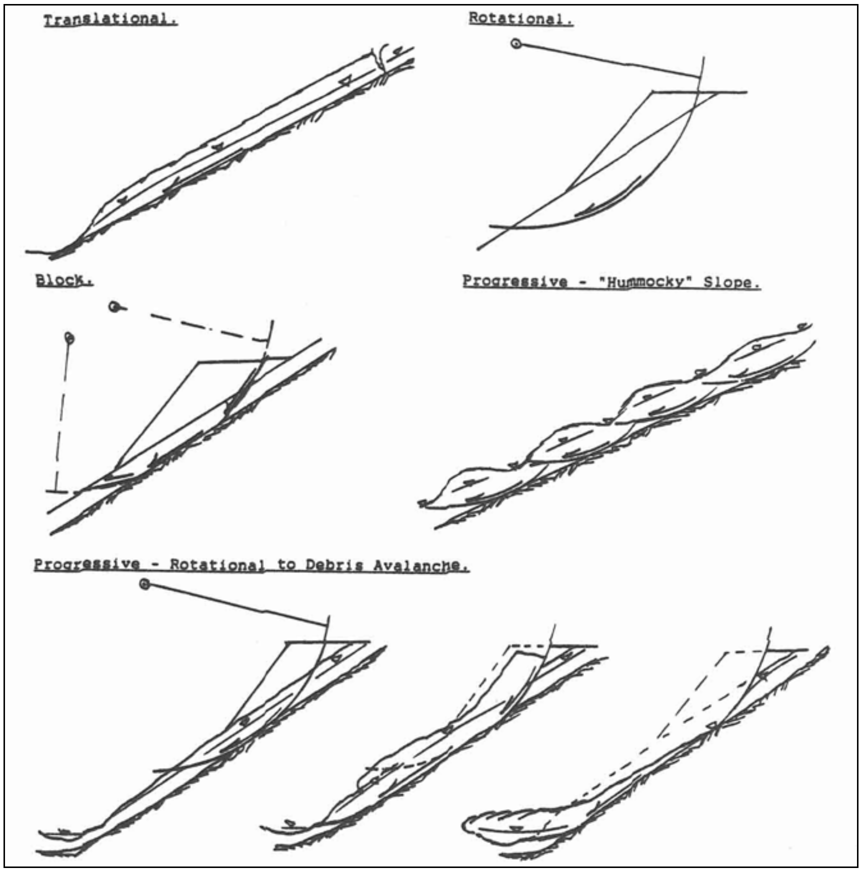

Figure 3.3. The translational and rotational forms of slope failures and composite forms. The pure translational slide would have a tension crack at the top and be completely translational from there down, However, actual translational slides will often have some shearing motion in the upper part and may well break out in the lower parts as one or more rotational shears. The lower set of three sketches shows a rotational slide progressively changing to a debris avalanche or flow as a result of the disaggregation of the sliding mass. From Prellwitz and Remboldt (1994, Figure 5A.2).

the lower end of the rupture surface (where the rupture broke up toward the old ground surface). It is also common for pieces of the displaced material to stack up on top of or push over earlier blocks or masses of displaced material. Seen in plan view from above, such rotational shear surfaces are commonly arcuate and concave out toward the stream.

A typical example of a rotational slide is shown in Figure 3.4. The slide does not have the simple form shown in Figure 3.2. Instead, there are multiple blocks and scarps, formed as the slide retrogressed away from the stream. Some of the displaced blocks have deformed in a plastic fashion while others have fractured into smaller blocks. As much of the material is highly disturbed has lost much of its strength, flows are widespread (such slides commonly break up into flows in their lower portions).

Earth flows in the lower portions of rotational slide/flows are in some places so extensive that they mask the original brittle nature of the slope failure.

a.

b.

Figure 3.4. Complex rotational earth slide and flow in lacustrine sediments on the Mad River in Waitsfield. a) Top scarp. Note down-dropped block with grass. Mike Blazewicz for scale. May, 2003. b) The complex form of a fresh landslide deposit. View from main scarp of landslide shown in previous figure, looking down onto the displaced material at the foot of the complex rotational earth slide and flow. A fresh earth flow is seen at lower left. The trees out on the surface rode down as blocks on top of the flow--some a few days after the main failure had occurred. Most of the foot is a jumble of blocks of varved silt and silty clay in a matrix of remoulded silt and silty clay. The toe is being rapidly eroded away by the river. May, 2003.

As will be discussed further in a later section, zones within a deposit that have experienced previous slope failure, either in the recent past or thousands of years ago, may not have anywhere near their original peak shear strength and may thus serve as zones of critical weakness. The impact of such weak zones on slope stability is one of the principal reasons for the careful examination process described in Chapter 5. Such zones are properly modeled using residual (rather than peak) shear strength.

Translational Slides

Eroding stream banks that are underlain by the dense till that is common throughout Vermont commonly fail through relatively shallow landslides. On wooded till slopes that have not experienced landsliding for a considerable time, the upper several feet is typically some combination of till that has weathered in place and/or colluvial material derived from till. In both cases the material retains the wide range in grain sizes of the parent till and is significantly weaker than the underlying unweathered till. This upper material is often relatively impermeable and thus slow to drain. If the toe of such a slope is eroded by a stream. the contrast in strength between the weathered till/colluvium above and the dense, relatively unweathered till below results in the slope having a tendency to fail along the boundary. Thus, although the slides can extend great distances up and down the slopes and along the slopes, the slides in till rarely "bite" into the hillside deeper than 10 feet or so at a time.

As is common with failures on natural slopes, more than one process may operate in a translational slide. The cohesion due to roots from shrubs and trees may help hold the slope together in large patches, yet failure has to happen somewhere. The first visible fractures will be in the form of tension cracks at the upper boundaries and perhaps strike-slip faults along the sides of the failing area. Some blocks will slide intact all the way down to the base of the slope while others will disaggregate into flows. Figure 3.5 shows a large translational slide in dense till on Ira Brook in Ira. Observations over several years confirm the shallow nature of the failure. The early stages of a shallow slope failure on till are shown in Figure 3.6. A large bank failure in dense till on Water Andric in Danville is shown in Figure 3.7. As this figure shows, such a slide can be very effective at transporting whole trees and large parts of their root systems as intact blocks down to the base. The remaining weathered material up on the raw landslide slope may be transported largely in the form of flows.

Figure 3.5. Ira Brook landslide. Shallow translational slide in till. Looking upstream. Cow for scale at top of bank. Note slabs of soil with trees falling down the bank, 10/24/2003.

Figure 3.6. Shallow translational slide on weathered till and lacustrine silty clay on a wooded slope in Hardwick. Soil block in center with birch tree has slid down to the right. November, 2003.

Figure 3.7. Slope failure on Water Andric, Danville. a) The overall form of movement here is a translational earth slide and earth flow. Continued toe erosion has enlarged the slide, leading to toppling of trees out onto the water-saturated flow surface. b) Closeup of upper part of the slope. George Haselton examines a thin layer of varved lacustrine sediments: Although the majority of the slope is till, in the uppermost few meters the till is overlain by varved lacustrine clay, sandy diamict, and a final layer of varved lacustrine clayey silt. The overall slope failure here is dominated by the till. Note the thin root zone.

Similar landslides occur in coastal Alaska and British Columbia on steep mountainsides mantled with shallow, relatively impermeable till over bedrock (Sidle and Ochiai, 2006). Because of the extreme topographic relief, these landslides are much more extensive than in Vermont. Besides the earth slides and flows that result from slope failure on Vermont till slopes, there are also abundant debris slides, flows, and avalanches. However, as in Vermont, failures commonly follow events that lead to high pore water pressure in the thin blanket of soil--in the case of the Pacific Northwest, heavy autumn rains.

Flows

Flow-type slope failures are found in two main settings in Vermont. The displaced material of translational and rotational slides is commonly disaggregated into small- or medium-scale earth or debris flows as shown in Figure 3.4b and channelized debris flows on steep mountainsides can move large quantities of boulders, cobbles, and finer material. The debris flow shown in Figure 3.8 is a typical example of this second type.

Figure 3.8. Looking up a recently active debris flow path on the west side of Smugglers Notch, Cambridge. A heavy rainstorm mobilized material up to three feet across over the entire length of the channel, which is about 900 feet long. Orange notebook in center of channel for scale. Note levees of coarse debris on both sides. July, 2006.

The channelized debris flows may originate from slope failures on the slope or be initiated by rock fall from a cliff. The source material for the flow in Figure 3.8 appears to have been the channel and sides of the gully. Although there were signs of recent runoff from a ravine that cuts into the steep cliff above, no signs were observed of fresh rockfall from the cliffs above the gully.

Several instances of debris avalanches or debris flows in and near Vermont were mentioned in Chapter 2. The debris avalanches are distinguished from debris flows by being spread out over a large area on a mountainside rather than confined to a channel. Although both of these types of landslide have occurred in Vermont, and they should be considered when assessing riparian corridor hazards in and near mountainous terrain, they are not discussed further in this report.

Wedge and Cantilever Failures

In addition to the bank failure types described above, two additional types are found in Vermont. These are the wedge and cantilever slide types, as illustrated in Figure 3.9. The wedge failures can form either with a simple planar shear surface dipping down toward the river or with a vertical tension crack in the back and a planar shear surface at the base. The cantilevers can fail either with a vertical shear surface in the back or by rotation around an axis at the back of the block. All of these appear to be limited to fairly low stream banks that are less than 10 to 15 feet high and they are best developed in modern alluvium and older stream terrace materials, although the cantilever failures may form in any eroding bank that has cohesive over non-cohesive materials. Although there may be a cohesive upper layer (usually due mostly to root cohesion) or a cohesive layer near the base, such failures are generally in non-cohesive materials. The wedge with planar shear surface, wedge with vertical tension crack and planar shear surface, and the cantilever with vertical shear surface can be modeled by the Bank Stability and Toe Erosion Model of Simon and others (2001) that will be described in Chapter 6.

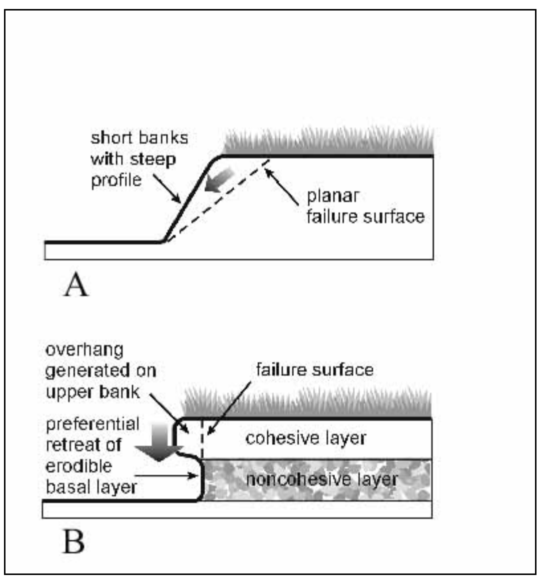

Figure 3.9. Planar and cantilever types of stream bank failures. a) Planar failure. b) cantilever failure. Some cantilevers may fail by shearing and drop down into the stream while others may fail by rotating out around a pivot point near the base of the overhang (toppling). Modified from Simon and others (2001, Figure 1).

Figure 3.10. Cantilever failures on the west bank of the Dog River, Norwich University, Northfield, Vermont. Shovel and notebook for scale. Flow is away from observer. Sandy alluvium has been undercut during recent high flows and is in the process of collapsing. Note collapsed sod temporarily protecting base of bank in distance. April, 2007.

In Vermont, wedge slides are sometimes encountered on failing stream banks composed of silty to sandy alluvium, although they typically have steeper failure surfaces than shown in Figure 3.9a. Cantilever failures on an eroding bank of the Dog River are shown in Figure 3.10.

Banks with a mechanically strong layer overlying a weaker one may form a cantilever as the material below is either eroded away directly by the stream or by ground water sapping or piping or else it fails due to removal of support or increased pore pressure. In either case, the overlying layer is left unsupported and will commonly fail by either shear or rotation. The cantilevers that fail by shearing could be classed as falls using Figure 3.1 and those failing by rotation could be classed as topples. Cantilevers are a fairly common occurrence on outside bends of meanders. In Vermont, they appear to be most common in modern alluvium or stream terrace deposits that consist of one or more fining-upward sequences of channel or bar deposits overlain by levee and overbank deposits. The coarse-grained channel or bar materials below are unlikely to have much cohesion while the finer-grained materials above can have considerable cohesion due to root mats and, if fine enough, matric suction (this last only as long as the material is not fully saturated).

General Causes of Slope Failure

Although toe erosion is certainly a major factor in the overwhelming majority of eroding stream banks in Vermont, it is important to recognize that slope failures, including those along streams, can be attributed to a wide variety of causes, many of which are interrelated. The factors listed in Table 3.2 are modified from Cruden and Varnes (1996, p. 70) and Wieczorek (1996). Any factor that contributes to low strength in the mass of soil or rock or which increases the shear stress can play a role in a slope failure. It should be noted that surface and ground water conditions play important roles in many of these factors. Many of the factors listed below will be discussed in more detail in later sections of this report.

Table 3.2. General Causes of Slope Failure. Modified from Cruden and Varnes (1996, p. 70) and Wieczorek (1996).

- Weak, sensitive, or weathered materials
- Discontinuities in the soil or rock mass (faults, joints, etc.)
- Permeability contrasts that lead to increased pore pressures
- Stiffness contrast, as in stiff material overlying less stiff material
- Fluvial erosion at the toe of the slope
- Wave erosion on large lakes
- Headcutting, sapping, and piping
- Intense rainfall
- Rapid snowmelt
- Earthquakes
- Freeze-thaw weathering
- Stream alterations of a wide variety of types
- Oversteepening of a slope through excavation
- Placing fill or other loads on a slope or at the crest
- Reservoir drawdown (impact, if any, would be on the reservoir bank)
- Removal of woody vegetation from hillsides and streambanks
- Intentional or inadvertent addition of water through stormwater diversion, irrigation, or leaking pipes
- Artificial vibration

Influence of Vegetation on Stream Bank Stability

Riparian vegetation has a wide variety of effects on both the erodibility and the stability of stream banks. Besides the major influences of woody and non-woody vegetation on channel and floodplain roughness, and the manifold impacts of large woody debris in river systems, vegetation can also strongly influence whether or not a particular section of a stream bank will be eroded or will collapse (Wynn, 2006). However, the interrelationships between flowing water, above-ground stems, below ground roots, bank materials, and ground water are quite complex. While some authors have made the case that woody riparian vegetation enhances stream bank stability by the addition of tensile strength to the soil (Thorne and others, 1981), others have pointed out the shallow nature of most root systems and have therefore suggested that roots contribute little to the stability of a high bank (see references in Lawler and others (1997).

The mass of trees on a slope can certainly add a certain amount of driving force to a slope failure. Although at times considerable, this tree surcharge is often compensated by increased root cohesion (Hammond and others, 1992).

A particular controversy exists over the question of the relative effectiveness of herbaceous and woody vegetation in stabilizing stream banks. Several studies have documented that streams with forested riparian buffer zones were measurably wider than those with herbaceous buffers, implying that the banks have been subject to slope failure and are therefore less stable. However, as research has continued it appears that more factors are involved in the channel width than just the presence of woody or non-woody vegetation (see references in Wynn, 2006). Anderson and others (2004) studied a wide range of stream systems and concluded that channels with watersheds less than 10 to 100 km^2 had narrower channels in those reaches with herbaceous vegetation than in comparable wooded reaches while those with drainage areas greater than 10 to 100 km^2 had narrower channels in the reaches with dense woody vegetation. The authors suggest that factors such as bed and bank material, land use, valley and floodplain characteristics need to be considered in sorting out the actual determinants of channel width. Slavin (1977) concluded from his study of sites on the Browns River that, due to their great weight and the large areas of ground that they disturb when they fall over due to bank erosion, trees do not contribute positively to bank stability. However, most other studies conclude otherwise. For example, recent stream bank erodibility testing in southwestern Virginia by Wynn and Mostaghimi (2006) indicates that large diameter roots associated with woody vegetation increase the resistance to fluvial erosion, thereby supporting the idea that woody vegetation contributes to bank stability.

Although the influence of vegetation on stream banks may be complex, the general picture that emerges from slope stability studies is that woody vegetation generally augments the stability of slopes, both through adding root cohesion in the upper parts of the soil and by removing soil moisture through transpiration and thus lowering the ground water surface (Easson and Yarbrough, 1988; Riestenberg and Sovonick-Dunford,1983; Sidle and Ochiai, 2006, especially p. 89-110). A detailed literature review on the subject is contained in Hammond and others (1992, Appendix B). A more recent compilation of root strength data is in Sidle and Ochiai (2006, Table 3.2).

Although additional research is no doubt needed, it may be helpful to note the comment by Wynn (2006, p. 10) that "[p]ersonal observations in the field have shown that forested streams in the eastern US have nearly vertical, stable streambanks that provide habitat for aquatic species native to that region". This agrees with the present author's experience with Vermont stream reaches that are perceived be in more or less of a "reference" condition. The inference being that woody vegetation contributes far more to holding a bank together than tearing it apart.

4. Stream Bank Weathering and Erosion

General Statement

As already described in Chapter 2, the processes operating to alter stream banks can be divided into weathering, fluvial erosion, and slope failure (Lawler and others (1997). Slope failure has already been discussed in Chapter 3. This chapter will examine the weathering and fluvial erosion processes. A section is also included on the sapping and piping that result from ground water flow out from banks.

Weathering

Some freshly exposed stream bank materials can be observed to undergo dramatic changes in their properties in a matter of months or even weeks. This is one of the reasons that geologists find the need to scrape away the outer layers of most surficial deposits before making observations. A freshly eroded stream bank surface in dense silt-matrix till is extremely hard and difficult to penetrate. When such till is fresh, a vigorous blow from a heavy pick can hardly penetrate, but a year later, the till at this same spot is easily scraped down to a depth of several centimeters with a shovel. Undisturbed Vermont soil profiles having till as parent material commonly show evidence of alteration at depths of 1.5 meters or greater.

The processes that result in these changes to materials near the Earth's surface are collectively known as weathering. A variety of physical, chemical, and biological processes are probably involved in the alterations that are observed on Vermont stream banks. The emphasis here is on the processes that can rapidly alter the characteristics of freshly eroded banks. More details on weathering processes can be found in Birkeland (1999) and Bloom (1998).

It is not always clear which processes resulted in the softening of a freshly eroded bank. Is it cyclic wetting and drying? Freeze thaw? Has leaching of carbonates played a role? Are there other processes? Despite the uncertainties, there is some evidence that one of the important processes, as described below, is freeze-thaw cycling.

In late fall and early winter, as the frost first begins to penetrate deeply into the ground, some raw exposures of silt or clayey silt become coated with spectacular needles of ice that have grown out of the soil surface, pushing grains and small aggregates of soil ahead of them (Figure 4.1). Lawler (1993) has discussed the influence of this needle ice formation on bank erosion at a site in Scotland. Needle ice formation typically occurs in late fall and early winter and results in a "fluffed" soil surface with lower cohesion upon thawing. The result is increased likelihood of erosion. Gatto and others (2001) review the effects of freeze-thaw processes on soil erosion and point out that there is a rough annual cycle of soil strength in the frost-susceptible soils (soils with silt or clayey silt matrices): Soils have high strength in the winter while frozen, are weakest in the spring when recently thawed and still saturated, and subsequently recover strength over the summer and fall as drainage takes place and particle cohesion returns. Thus newly thawed soils can be quite susceptible to slope failures. The depth of this cycle of strength is dependent on the amount of frost penetration. Field observations by the author indicate that over long periods of time (greater than one year) the stream bank surface is being progressively weakened. I f erosive force is absent and vegetation is established, the material may well stabilize. However, it is usually the case that the frost-softened material will be swept off during the next heavy rains or high water and new material will be exposed for softening.

Figure 4.1. Needle ice formed on a recently excavated silt-matrix till, Woodstock. The needle ice appears to form in frost-susceptible soils in late fall or early winter if soils are sufficiently moist while temperatures dip will below freezing without snow cover. This is an important process for softening fresh exposures in dense fine-grained soils. Knife for scale.

Another widespread process in Vermont soils is the leaching of calcite or other carbonate minerals from the upper portions of soils by low-pH groundwater. Field observations indicate that where such minerals are present in the deeper parts of the C horizons, the overlying layers are depleted. Soil profiles in Vermont tills derived from carbonate rocks commonly show signs of this leaching. In cases where the calcite was an intergranular cement, the reduction in soil strength will probably be substantial.

Carbonate-cemented sand strata are common in the lacustrine deposits of the state. The cement has typically formed at some time subsequent to the deposition of the sand as carbonate-rich waters percolated through the material. As with the till, dissolution of this carbonate in low-pH waters typical of the near-surface environment will probably reduce the strength of the deposit.

Sapping and Piping

Sapping and piping are important processes that can work to modify stream banks. Both processes involve removal of granular material by ground water flowing out from a free face along a contact between two materials. Sapping is spread out along the contact while piping is localized into one or more subterranean channels that daylight onto the free face. These processes operate somewhere in the realm between the softening of bank material and the final fluvial erosion that removes the material from the bank.

An example of slope degradation due to sapping is shown in Figure 4.2. This site is at the back of an active landslide in varved lacustrine deposits in Jeffersonville. Water is seeping out from sandy layers overlying silty clay layers. The silty clay layers acts as confining layers that prevent or at least inhibit vertical movement of ground water. As the water seeps out from the sands, it carries grains of sand with it, hollowing out each sandy layer and leaving behind an overhang of silty clay.

Figure 4.2. Sapping at interfaces of varved sand over silty clay. Each varve consists of a "summer" layer composed of very fine sand is overlain by a silty clay "winter" layer above. This silty clay serves as a barrier to easy vertical movement of water and thus water in the next season's summer layer seeps out onto the face. Water was observed to be seeping out from the lower several centimeters of each of these summer layers. The two lines of dark hollows on the slope above the shovel head are areas where sand has been sapped (eroded out) by the water, leaving the winter layer above cantilevered out over the hollow. At the back of an active landslide scarp above the Brewster River, Jeffersonville. Photo by Jonathan Kim, Vermont Geological Survey.

An example of piping followed by roof collapse to form a gully is described below by Henry David Thoreau (from Blake, 1896). The site was on a bank of the west side of the Sudbury River in Concord, Massachusetts:

> *February 28, 1855. I observed how a new ravine was formed in that last thaw at Clamshell Hill. Much melted snow and rain being collected on the top of the hill, some apparently found its way through the ground frozen a foot thick, a few feet from the edge of the bank, and began with a small rill washing down the slope the unfrozen sand beneath. As the water continued to flow, the sand on each side continued to slide into it and be carried off, leaving the frozen crust above quite firm, making a bridge five or six feet wide over this cavern. Now since the thaw, this bridge, I see, has melted and fallen in, leaving a ravine some ten feet wide and much longer, which now may go on increasing from year to year without limit.*

There are several ways in which such piping on stream banks can happen. In the example given above, the strong upper horizon appears to be due to an upper layer of frozen ground, underlain by loose sand. At other sites, the strong upper horizon may be due to roots or to cohesive soils. Figure 4.3 shows a gully at the Sleepers River Research Watershed in Danville, Vermont, where piping occurred in weathered till that had been roofed by a densely rooted upper horizon. An underground channel daylights here, goes back under the topsoil, and is exposed again about 100 feet downhill. The person is standing in the channel.

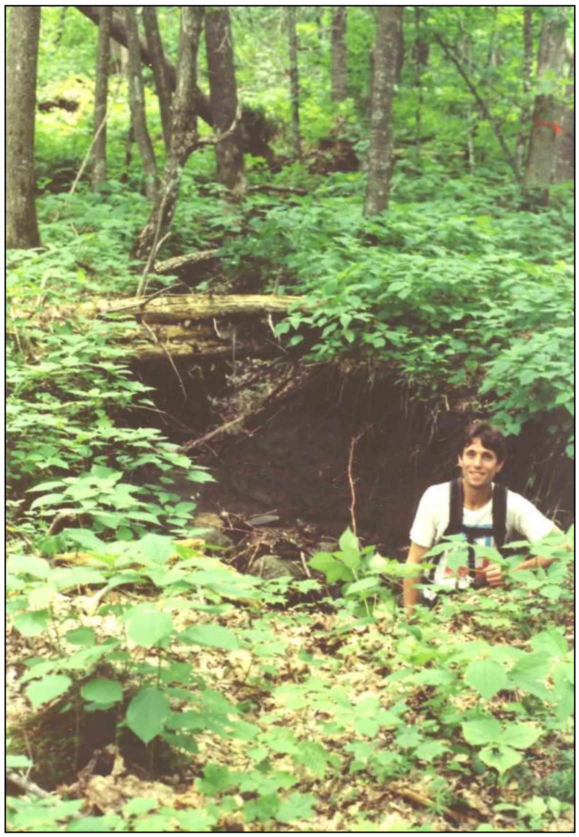

Figure 4.3. Gully formed due to failure of the roof of a piping structure. View looking downhill. The pipe daylights here, goes back under the topsoil, and is exposed again about 100 feet downhill. Jamie Shanley of the U.S. Geological Survey for scale. Site SR 7, W-9 Watershed, Sleepers River Research Watershed, Danville, Vermont. June 25, 2000.

Fluvial Erosion by Hydraulic Shearing

The material on a stream bank will be eroded when the shear stress applied by flowing water on the bank is high enough to cause particles or aggregates of particles to be detached from the surface. Such shear is particularly effective when the material has been softened by the weathering processes described above. This removal of material from the bank is likely to lower the factor of safety by reducing the resisting force of the bank. Thus, unless there is some compensating change that increases the factor of safety, it is probably a prelude to further bank failure.

The analysis of hydraulic shear stress and erodibility given below is simplified and focuses on information needed for the modeling of bank erosion described in Chapter 6. For more detailed analyses, see Knighton (1998), Gordon and others (1992), and Hanson and Simon (2001).

Particles in a fluvial system can be in one of three states: at rest on the bed or bank, being actively transported by the flowing water, or in the act of settling out of the water. The so-called Hjulstrom diagram in Figure 4.4 relates particle size and mean flow velocity to the state of transport (erosion. transportation, or deposition). Note that the boundary between erosion and deposition drops from particles of 0.001 mm diameter to about 0.5 mm diameter. It then rises as particle size continues to increase up through coarse sand and the gravels. This means that fine to medium sands are easier to erode than both coarser and finer materials. The diagram also illustrates that once entrained, fine grained particles below about 0.01 mm can be transported at very low velocities. These are valuable insights, but the diagram also has severed limitations. First, the data comes from flume studies of sediments with uniform grain sizes. Second, it's not the mean flow conditions in the flume that determine whether or not particles move, it's the conditions right down near the bed surface. Third, the range of flow depths was limited. Because of these limitations, the Hjulstrom diagram is useful mostly as a conceptual model. In order to develop a quantitative model it is necessary to consider the concept of critical hydraulic shear stress outlined below.

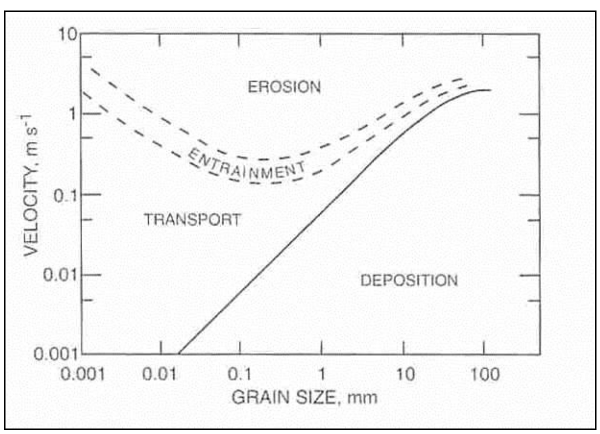

Figure 4.4. Hjulstrom diagram relating particle size and mean velocity to the state of transport. Data from flume studies with uniform particle sizes. From Knighton (1998, Figure 4.5b).

Hydraulic Shear Stress

The hydraulic shear stress (also called the tractive force) is the stress exerted by flowing water against particles in the bed of the stream (or the submerged part of a bank). As a first approximation to relating flow conditions to entrainment of particles, the

Average boundary shear stress τ is calculated as follows

$$\tau_0 = \gamma_w R S \qquad \text{Equation 4.1}$$

where

τ_0 = average boundary stress (Pa)
γ_w = density of water (kN/m^3)
R = hydraulic radius or local depth (m)
S = water surface slope (m/m).

Hydraulic radius R is calculated from

$$R = A / P \qquad \text{Equation 4.2}$$

where

A = area of channel (m^2)
P = wetted perimeter of channel (m).

Note that τ_0 is an average for the stream channel as a whole. The shear stress at a point of interest (perhaps the base of the outside of a meander bend) may be far higher than this. Therefore, it is more appropriate to calculate the shear stress on a part of a bank or bed τ (Pa) as follows

$$\tau = \gamma_w h S \qquad \text{Equation 4.3}$$

where

τ = local boundary stress (Pa)
h = local depth (m).

Below some critical value of hydraulic shear stress, a given particle will not move. Above it, the particle will be set in motion. The value at which the particle is entrained is called the critical shear stress. Figure 4.5 from Knighton (1998, after Williams, 1983) shows the relationship of grain size to critical shear stress for gravel-size particles. For several reasons there is a broad zone that separates the fields of definite motion from definite stability. It is simply not possible to define a single critical shear stress for all particles of a given size. The reasons relate both to channel and flow characteristics and also grain characteristics. For example, turbulence makes it difficult to accurately determine the flow conditions near a particle. The shape of a particle and its orientation relative to flow has an important influence in the ease of entrainment. Also, a particle does not exist in isolation--variations in size and arrangement of nearby particles will have great influence on whether or it will begin to move. Yet another problem arises because all of the preceding discussion has assumed uniform, non-varying flow, a condition often not present in real streams. See Knighton (1998) for more details.

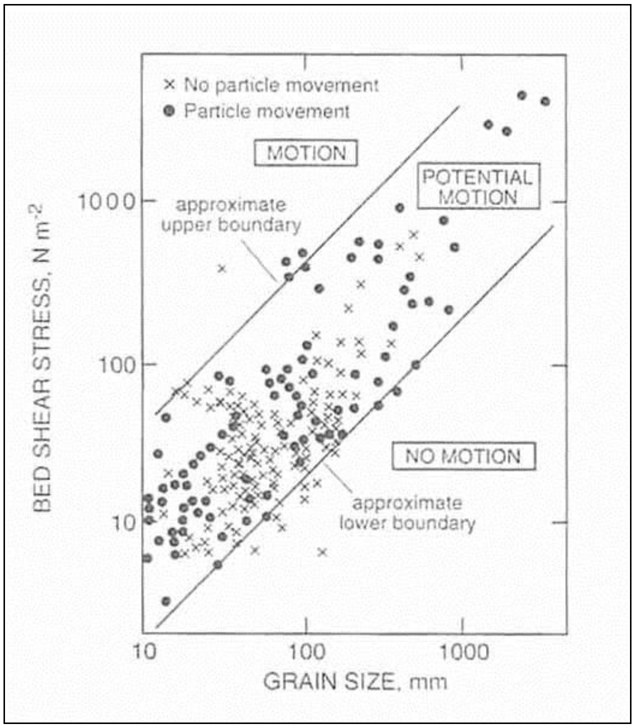

Figure 4.5. Grain size versus bed shear stress. Note that the two diagonal lines define a broad zone of critical shear stress (labeled "potential motion") above which particles are in motion and below which particles are stable on the bed. From Knighton (1998, Figure 4.5c).

Erodibility

Once the flow conditions required for entrainment of particles have been met, it still remains to determine the volume of material that will be removed. The more that the applied shear stress exceeds the critical value, the greater the amount that will be removed in a given time. This means that the hydraulic erosion of material from a bed or bank can be described by a rate of erosion ε as defined by Hanson and Simon (2001)

$$\varepsilon = k_d(\tau_e - \tau_c) \qquad \text{Equation 4.4}$$

where

k_d is the erodibility coefficient (m^3/N-s)
τ_c is the critical shear stress (Pa)
τ_e is the effective shear stress (Pa).

Reasonable values for effective shear stress can be estimated by use of equation 4.3 for bed shear stress. Greater accuracy is achieved if velocity profile measurements are made in the vicinity of the bank as described by Gordon and others (1992, Section 6.5.3).

Critical shear stress and erodibility present greater difficulties. For non-cohesive materials, the critical shear stress can be estimated from diagrams such as Figure 4.5, although for the reasons described above, the actual *in situ* values are subject to great uncertainty. For cohesive materials, such as silty clay or clay, little information on critical shear stress has been available. In recent years field and laboratory jet testing devices have been developed to provide both critical shear stress values and erodibility values for cohesive materials (Hanson, 2001; Hanson and Simon, 2001). Although data for Northeastern U.S. materials is not yet available, some of the results have been incorporated in the Bank Failure and Toe Erosion Model of Simon and others (2001) that will be discussed in Chapter 6.

The research accomplished to date does, however, highlight the tremendous range in critical shear stresses for cohesive materials. Figure 4.6, for example, shows critical shear stress and erodibility values for stream beds in Midwestern loess (Pleistocene wind-blown silt deposits). Despite obvious differences, it would be surprising if Vermont materials did not show a similar wide range in values. Thus, successful bank erosion modeling will require Vermont-specific information, both on critical shear stress and erodibility.

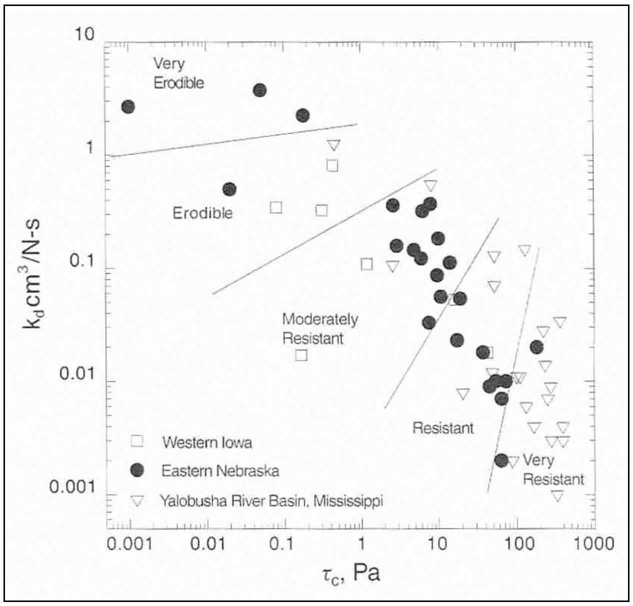

Figure 4.6. Critical shear stress versus erodibility for cohesive loess from the Midwestern U.S. Note the wide variation in both variables. Although no equivalent data is known for Vermont materials, the variation in values is likely to be similarly wide-ranging. From Hanson and Simon (2001, Figure 8).

Although field observations clearly demonstrate that the common stream bank materials in Vermont vary greatly in their erodibility, it should be noted that even the freshest exposures of the stiffest clays and firmest, most boulder-rich, silt-clay matrix tills are dramatically eroded during high stream flows.

Effects of Vegetation on Bank Erosion

The influence of vegetation on stream bank stability was discussed in the previous chapter, with some overlap into the realm of bank erosion, so the discussion here is limited to some comments on the fashion that vegetation can alter the erodibility of a bank.

Field observations by the author and many citations in the literature amply demonstrate the effectiveness of woody vegetation in reducing the velocity of flood waters. It is common to see much more flood damage on unbuffered farm fields than well-buffered floodplains in similar geomorphic positions.

The influence of vegetation on the bank itself, as discussed in the previous chapter, is much more complex

It is also certainly true that vegetated and/or cohesive blocks that have toppled or fallen to the base of a bank following a cycle of erosion and bank failure may have some ability to protect the bank from further erosion (Knighton, 1998). Slavin (1977), in his study of erosion on the Browns River in northwestern Vermont, emphasized this armoring effect. However, as Knighton points out, the effect is probably quite temporary when viewed over timespans of many years,

Location of Slope Failures Relative to Meanders

As a general rule, stream bank erosion is highest on the outer bank of a meander, particularly on the downstream 1/3. In such a setting, a tall bank may be the location of many landslides over the span of some years or decades, with the site of greatest slope instability shifting as the point of impact on the bank shifts. Thus, if the meander is migrating downstream, the zone of active landsliding will also migrate, leaving behind inactive (or at least less active) landslides. Such progressions of upstream-increasing landslide ages can be seen both in the field and by careful analysis of aerial photographs. Where present, they provide evidence of a continuity of process and can be of use in predicting the locations of future bank instability. These generalizations break down on reaches that are undergoing serious widening or planform adjustment, where large sections of stream bank may be unstable, regardless of their position on the meanders. The Rapid Geomorphic Assessment component of the Phase 2 assessments (completed as part of the Vermont Stream Geomorphic Assessment protocols) will help in identifying such reaches (Agency of Natural Resources, 2002b, Section 7).

Rates of Stream Bank Retreat

The end result of the complex set of processes described in the preceding chapters is an overall retreat of the slope (constructional processes that may be occurring elsewhere in the riparian system are not being considered here). Considering the wide variety of processes that operate, it should not be surprising to find that bank erosion rates are extremely variable.

In a study of bank erosion along rivers in Devon, Great Britain, Hooke (1979, 1980) found rates of erosion of banks to vary widely. Field measurements were made at 17 sites over a 2 1/2 year period from 1974 to 1976 and compared with rates derived from analysis of three sets of maps dating from 1842 to 1975. Mean rates from field measurements varied from 0.08 - 1.18 m/yr with a maximum rate of 2.58 m/yr. Mean rates derived from the map analysis were 0 - 0.68 m/yr for 1842 to 1903, 0.05 - 1.35 m/yr for 1903 to 1958 or 1968 (dates of the middle set of maps varied), and 0.46 - 1.79 m/yr for 1958 or 1968 to 1975. An analysis of the processes of bank erosion indicated that direct corrasion (erosion by the scouring action of flowing water) and slumping were the major mechanisms moving material from the bank into the stream. No one factor emerged as a good predictor of rate of erosion. Using a plot of drainage area versus rate of erosion (Figure 4.7), Hooke (1980) compared these results with those from 28 other studies from around the world. Drainage area in these studies varied over 6 orders of magnitude while erosion rate varied over 5 orders of magnitude, with a general positive correlation but very wide scatter, something that is not at all surprising given the variety of stream types and river bank materials involved.

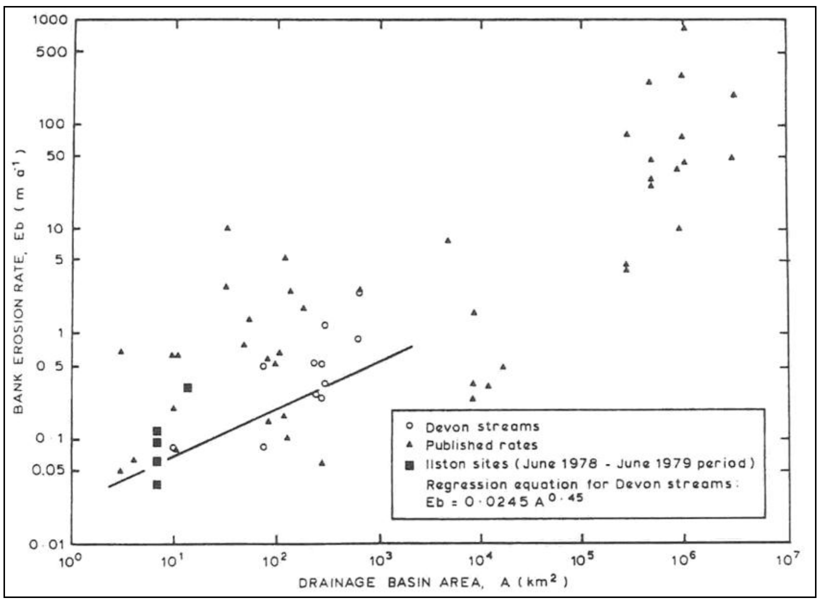

Figure 4.7. Drainage basin area versus bank erosion rate. Lawler and others (1997, Figure 6.7).

An ongoing study at an unstable meander bend of the Third Branch of the White River at the Randolph Landfill in Randolph, Vermont provides an excellent example of the variability and potential rapidity of stream bank erosion. Measurements were made along an approximately 400 foot (120 meter) section of the top of the bank from May of 2002 to March of 2005 and showed an annual rate of bank retreat of at least 9.7 feet per year (3.0 meters per year (Figure 4.8). Note, however, that three of the stations were eroded away entirely, so the actual rate would be somewhat higher. After May of 2005 the erosion rate jumped dramatically, with several more stations being destroyed. Between May of 2002 and December of 2005, the apex of the meander retreated about 235 feet (72 meters) for a rate of 21 feet per year (6.4 meters per year). Comparing these rates with Figure 4.7, the Randolph site (with a 274 square kilometer drainage area) is clearly at the high end of the ranges for similar basins. If longer sections of the stream were monitored, the overall rate would probably be somewhat lower. Although short-term averages probably have little meaning in such situations, but this case does illustrate just how fast some stream meanders can shift. An analysis of modern and historic maps, orthophotos, and aerial photos of the Randolph site shows that the bank retreat at this particular site is just a part of a widespread rapid meander migration (planform adjustment) underway in this reach. This is consistent with the findings of a study of the fluvial geomorphology of the Third Branch watershed by Barg (2002), who first documented the rapid erosion rate at this.

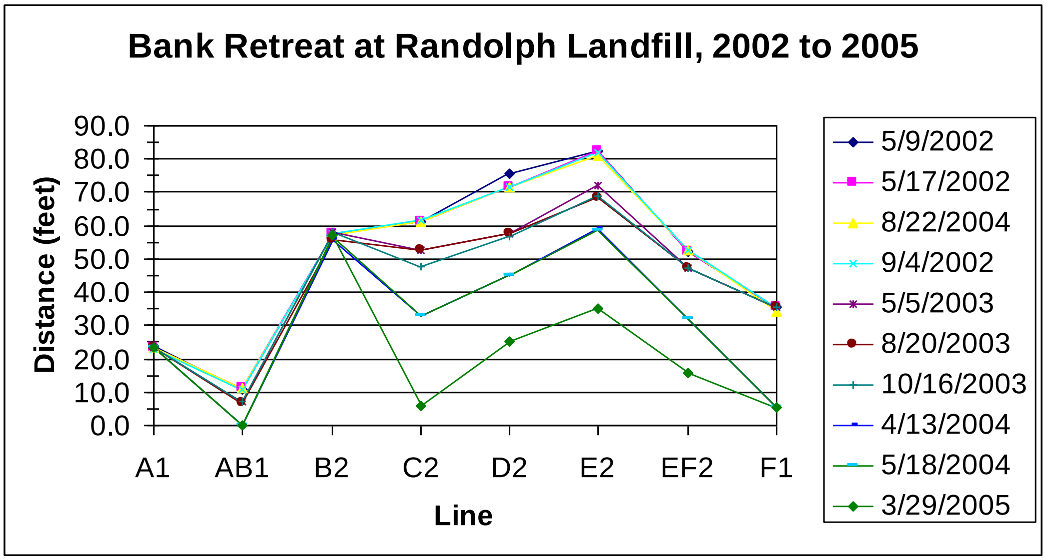

Figure 4.7. Bank retreat monitoring at the Randolph Landfill, Third Branch of the White River. In the time period shown, bank retreat averaged at least 9.7 feet per year (3.0 meters per year). Subsequently, the rate increased dramatically. See text for details.

5. Procedure for Detailed Site Investigations

This section contains suggested procedures for detailed investigations of surficial deposits, stream banks, and landslides. The Surficial Geologic Data Sheet (Appendix A) and the Slope Stability Datasheet (Appendix B) are provided to help the observer notice the details relevant to bank stability analysis. The sections that follow will describe some of the items on the Surficial Geologic Data Sheet in more detail. The items on the Slope Stability Data Sheet have mostly already been discussed in Chapters 3 and 4.

Field References

For more details on the wide variety of features that may be encountered during stream bank investigations, it may be helpful to have access to the publications listed below. The *Field Guide for Soil and Stratigraphic Analysis* (Midwest Geosciences Group, 2002) is a useful field card that summarizes many of the items discussed in the following section. *The Field Book for Describing and Sampling Soils* (Schoeneberger and others, 2002) provides an overview of the collection of field information from a soil science viewpoint. *The Field Description of Sedimentary Rocks* (Tucker, 1982) will serve as a good guide to observing and interpreting sedimentary structures (in unconsolidated sediments as well as in rocks).

Surficial Geologic Data Sheet

See Appendix A for the data sheet.

Location

Give a brief verbal description of the location.

Northing and easting

The choice of coordinate system and datum should be made at the start of the project. The best choice in most cases will probably be the Vermont State Plane Coordinates in meters with the NAD 83 datum. This is directly compatible with the GIS data produced by the State of Vermont. Another commonly used combination is the UTM Grid, Zone 18 or 19 (depending on the location in the state) with the NAD 27 datum. This choice eases plotting on U.S. Geological Survey topographic maps as this grid is printed on many of the maps and is at least present as tick marks around the margin.

Stratigraphic log

The log is intended to be a flexible way of recording detailed observations of the units encountered. The items to include are listed below.

Depths and thicknesses

Record the vertical thicknesses of the units in feet. To simplify plotting of derivative maps, record the elevation of the top of the section and record depths in feet below the land surface.

Unified Soil Classification System (USCS) Group

The USCS is a standard method used by soils engineers to describe unconsolidated deposits. For coarse-grained materials (gravels and medium to coarse sands) the classification is based on the predominant grain size and the range of sizes (grading) present. Finer sands and the silts and clays are classified using the grain size, liquid limit, plasticity, and other properties. Table 5.1 shows a summary of the USCS for field classifications. Appendix D describes field methods for estimating the required properties. Figure 5.1 is a plasticity diagram for use with laboratory data on Plasticity Index and Liquid Limit. The USCS Group classification should be recorded for each of the major units described in the stratigraphic log. Soils having characteristics of two groups can be described by combining symbols.

Table 5.1 Unified Soil Classification System. Field classifications can be made for most soils by the use of this table in combination with the field tests in Appendix C. From Walker and Cohen (2006).

Unified Soil Classification System

Compiled by Scott Burns, Portland State University

The USCS (United Soil Classification System) standard is a system for classifying soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index. It is used when precise classifications are required. Use of this standard, in almost all cases, will result in a single classification group symbol and group name.

Soil Classification

Major Divisions			Group Symbols	Typical Names
COARSE-GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size.	Clean gravels - less than 5% fines.	GW	Well-graded gravels, gravel-sand mixtures.
			GP	Poorly graded gravels, gravel-sand mixtures.
		Gravels with fines - more than 12% fines.	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size.	Clean sands - less than 5% fines.	SW	Well-graded sands, gravelly sands, little/no fines.
			SP	Poorly graded sands, gravelly sands, little/no fines.
		Sands with fines - more than 12% fines.	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS More than half of material is smaller than no. 200 sieve size.	SILTS AND CLAYS Liquid limit less than 50	Inorganic	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			ML	Inorganic silts and very find sands, rock four, silty or clayey fine sands, or clayey silts, with slight plasticity
		Organic	OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS Liquid limit 50 or more	Inorganic	CH	Inorganic clays of high plasticity, fat clays.
			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		Organic	OH	Organic clays of medium ot high plasticity, organic silts.
Highly Organic Soils			PT	Peat and other highly organic silts. Primarily organic matter, dark in color, and with an organic odor.

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U.S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (next page), and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. For a complete description of the Unified Soil Classification System, see "Technical Memorandum No. 3-357," prepared for Office, Chief of Engineers, by Waterways Equipment Station, Vicksburg, Mississippi, March 1953.

First published by GSA Engineering Geology Division.

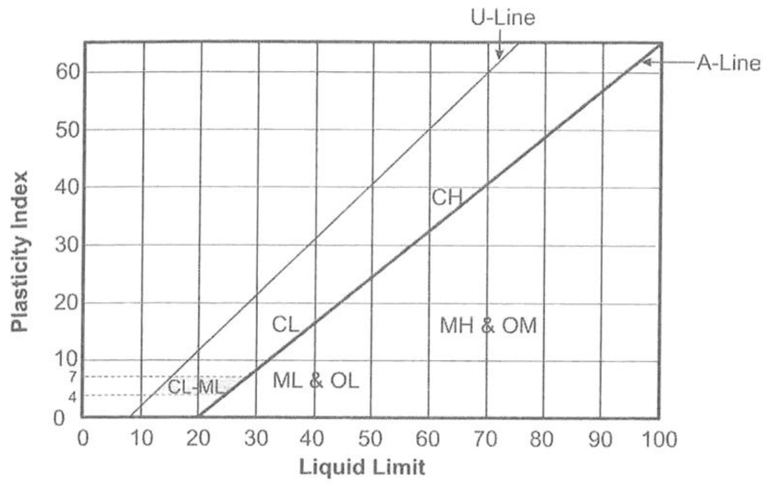

Figure 5.1 Liquid limit versus plasticity chart for fine-grained soils. For use with Table 5.1 when laboratory data on Liquid Limit and Plasticity Index are available. M = silt, C = clay, O = organic-rich. Clays will normally plot above the A-Line, silts below. Most soils plot below the U-Line. Silts or clays that plot to the left of the 50% liquid limit are low plasticity, those that plot to the right are high plasticity. From Walker and Cohen (2006).

Texture

The grain size of stream bank materials is important for understanding the geotechnical behavior of the bank, both because of the influence on soil shear strength and the influence on hydraulic ease of ground water movement. In soil science terminology the term "texture" refers to grain size distribution. Many classifications have been proposed for specific purposes. The standard classification for geologic analysis is the Udden-Wentworth scale described in Table 5.2.

Table 5.2. Udden-Wentworth grain size classification. Modified from Boggs (1995, Table 4.1).

Millimeters	Udden-Wentworth Size Class	
>256	Boulder	Gravel
16 - 256	Cobble	
4 - 16	Pebble	
2 - 4	Granule	
1 -2	Very coarse sand	Sand
.5 - 1	Coarse sand	
.25 - .5	Medium sand	
.125 - .25	Fine sand	
.0625 -.125	Very fine sand	
.031 - .0625	Coarse silt	Silt
.0156 - .031	Medium silt	
.0078 - .0156	Fine silt	
.0039 - .0078	Very fine silt	
<.0039	Clay	Clay

Density and consistency

Density and consistency are material characteristics that essentially refer to the ability to resist penetration. In standard geotechnical usage the term density is used to refer to coarse grained deposits and the term consistency refers to fine grained materials: silts and clays. The standard geotechnical classifications for these are given in Tables 5.3 and 5.4 and a field classification method is provided in Appendix C.

Table 5.3. Relative density classification. N-values refer to the Standard Penetration Test a commonly used field test performed while conducting split-spoon augering. From Renteria (1994).

N-value	Relative Density
0 - 4	Very loose
5 - 10	Loose
11 - 29	Medium dense
30 - 49	Dense
>50	Very dense

Table 5.4. Consistency classification for fine-grained soils. Unconfined compressive strength can be roughly determined by penetrometer or torvane tests. see section on shear strength below. From Renteria (1994).

N-value	Unconfined compressive strength (tsf)	Consistency
0 - 2	< 0.25	Very soft
3 - 4	0.25 - 0.50	Soft
5 - 8	0.50 - 1.0	Medium
9 - 15	1.0 - 2.0	Stiff
16 - 30	2.0 - 4.0	Very Stiff
> 30	>4.0	Hard

Color

The color of surficial materials can be helpful in interpreting the environment of deposition and the drainage of a material. The standard technique is to use a Munsell color chart. As soil color can change as the moisture content changes, it is probably best for general descriptions to compare as sample to the chart while in a moist state. Lighting can also make a difference in the perception of soil color, so the test should be done in direct sun when possible. Besides the matrix color, also record the color, distinctness, and abundance of mottles and other redoximorphic features (see Schoeneberger, 2002 for details on this).

Sorting

Sorting is the degree to which grains are of a uniform size. A very well sorted material has a uniform grain size while a very poorly sorted material has a wide range in grain sizes. The soil engineer's term "grading" is the inverse of sorting, with well graded soils having a wide variety of grain size and uniformly or poorly graded soils being of uniform size. "Gap graded" soils have a coarse and fine material with little of intermediate size. The degree of sorting or grading can have a dramatic effect on the hydraulic conductivity of a material--if the sorting is poor, many of the interstices between large grains are filled with smaller ones and flow may be greatly impeded. A sorting classification is shown in Figure 5.2.

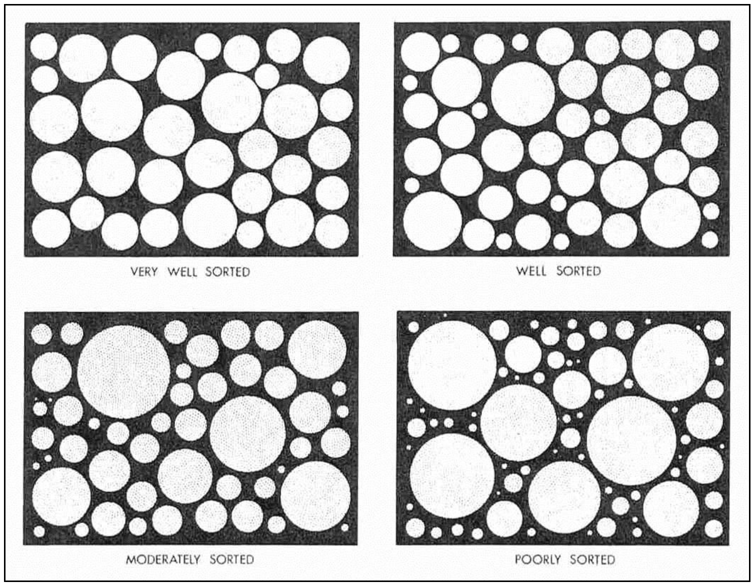

Figure 5.2. Sorting of particles. Very well sorted materials have quite uniform grain size (upper left) while poorly sorted materials have a wide variation in grain size (lower right). From Boggs (1994).

Moisture

As discussed in preceding sections, water or moisture content is of crucial importance in evaluating bank stability. Pay close attention to any signs that water may be seeping or flowing out at particular locations and record the locations. Even if you are visiting the site at a dry time of year, surface staining and vegetation may serve as indicators. The moisture content of each of the units should be estimated as follows: dry material feels dry to the touch, moist material feels damp but there is not visible water, and wet materials have visible water.

Cohesiveness

Each stratigraphic unit should be classified as cohesive or non-cohesive based on the results of the USCS classification described above. As described in Chapter 3, cohesion strongly influences the mechanical behavior of stream bank materials. In general, soil cohesion increases from near zero in gravels and sands to maximum values in clays. The high cohesion values in some clay soils arise from the large surface to grain-volume ratios of clay minerals. Intergranular forces can serve to tightly bind the clay mineral grains together, resulting in high cohesion values. Cohesion is not, however, related solely to grain size. It is also a function of compaction history, clay mineralogy, and pore water chemistry. For more details on this subject see Sidle and Ochiai (2006).

Plasticity

The property of being able to be deformed plastically without fracture is known as plasticity. There is a rough field classification of plasticity that can be performed by kneading a sample, rolling it out in the hand into a rod, and seeing if it can hold together when suspended. The criteria are listed in Table 5.5 and described in more detail in Appendix E. There is also a related laboratory test to determine the water content at which a sample changes from non-plastic to plastic. This is called the Plastic Limit and is described in Bowles (1984) and Kohler (1994), among others.

Table 5.5. Field criteria for determining the plasticity of cohesive soils.

Will not support 6 mm diameter roll if held on end.	Non-plastic
6 mm diameter roll can be repeatedly rolled and supports itself, 4 mm does not.	Low plasticity
4 mm diameter roll can be repeatedly rolled and supports itself, 2 mm does not.	Medium plasticity
2 mm diameter roll can be repeatedly rolled and supports itself	high plasticity

Stratification and Soil Structure

Surficial materials exposed in stream banks display a wide variety of physical features at a scale larger than the individual grains. Some of these features formed as sediment was deposited while others formed long afterwards. "Stratification" is the general term for the primary depositional layering of sediments. As described in Table 5.6, strata can vary from thick beds to thin laminae. It is possible for a deposit to be massive, that is, completely uniform throughout. This is most commonly encountered in deposits of till, although it is quite common that a careful search will reveal signs of stratification even in these.

Table 5.6. Stratification. From Boggs (1995)

Very thickly bedded	>100 cm thick
Thickly bedded	30 - 100 cm thick
Medium bedded	10 - 30 cm thick
Thinly bedded	3 - 10 cm thick
Very thinly bedded	1 - 3 cm thick
Laminated	< 1 cm thick

The term "structure" has unfortunately been used in three very different senses. Geologists use structure in two senses. They define primary sedimentary structures as the features formed during and shortly after the deposition of sediments, such as bedding, lamination, cross bedding, ripple marks, rain drop prints, faults and folds resulting from collapse, water escape structures, etc. By contrast, secondary structures form at some time after deposition. These include a wide variety of faults and folds that form long after deposition, as well as concretions formed through chemical interaction of ground water and sediments. In each of these geologic senses of the term "structure" the terms have strong genetic connotations. Excellent descriptions of sedimentary structures are given in Tucker (1982).

In contrast to the wide variety of structures recognized by geologists, soil scientists describe the structure within soil horizons using a fairly simple geometric classification (Table 5.7). Some of these have their origin as sedimentary features while others are the result of soil-forming processes. These terms provide a rough way to describe how the individual grains in the deposit form aggregates or bodies as they are broken out of the side of the bank. Materials with single-grain structure will be approximately or completely non-cohesive and their removal from a bank will be a fairly simple function of fluvial shear stress at the bank. Massive materials, by contrast, are cemented together in some fashion and will either have to be transported as blocks or broken apart into individual grains.

Table 5.7. Soil structure. Modified from Schoeneberger and others (2002).

Massive	Individual soil particles entirely bound together into one aggregate
Single-grain	Individual soil particles not bound to one another at all
Granular	Spheroidal peds or granules usually packed loosely
Blocky	Irregular, roughly cubelike peds with planar faces (angular to subangular)
Platy	Flat peds, usually roughly horizontal
Prismatic	Vertical, pillarlike peds with flat tops

Stratigraphy

Besides the fine detail of the sedimentary and soil structures, it is also very important to describe the overall depositional patterns, viewed both in cross-section and in planform. This is the stratigraphy of the deposits. These shifting patterns are due to changes in source areas and changes in the energy distribution within the depositional system (changes in water velocity, flow depth, turbulence, etc.).

Common patterns to be encountered in unconsolidated sedimentary deposits include the small-scale rhythmic sedimentation of varved lacustrine deposits, large-scale coarsening of grain size upward within lacustrine deposits as the water body fills in over time, and fining upward of fluvial deposits due to changing from bed load to suspended or wash load. See Boggs (1994) for detailed descriptions of sedimentary environments and depositional patterns.

Fractures

Geologists and soil scientists have long noticed that surficial deposits may be fractured, in places to depths of several meters. Prominent fractures should be described as encountered. The features to observe include the geometry of the fractures, the fracture density, continuity, cross-cutting relationships. As the fractures are developed in non-lithified materials below the surface of the ground, great attention should be paid to fracture infillings and alterations along the walls of the fractures. A good description of the general terminology for fracture description is in Bureau of Reclamation (1998, Chapter 5).

Weathering

Look for signs of weathering. Are freshly exposed parts a different color or consistency from older parts? As discussed in Chapter 4, weathering can have profound effects on the strength of stream bank materials.

Reaction to HCl

A simple test for the presence of carbonate minerals is made by placing a drop of dilute (10%) hydrochloric acid on a sample. Fizzing indicates the presence of carbonate. Carbonate-rich parent materials will often show strong leaching in their weathered upper horizons, as described in Chapter 4.

Clasts

In the broadest sense, clasts are sedimentary particles broken off of a larger body. In the sense of these field descriptions, the clasts are the large particles that stand out in a finer matrix. Their size, shape, arrangement, and lithology may be useful in interpreting the source area and environment of deposition. Clast characteristics are particularly important to note in till or diamict deposits.

Fabric

Fabric refers to the orientation of the particles in a sedimentary deposit. In these field descriptions the fabrics of interest will be those visible to the naked eye. Examples include the imbricated arrangement of pebbles, cobbles, or boulders arranged by flowing water so as to face upstream or the preferred alignment of the long axes of clasts parallel to the glacial flow direction as seen in some tills.

Roots

As discussed in Chapters 3 and 4, roots can have a considerable strengthening effect on stream bank materials. It is important that both the distribution and size of roots be noted. On most freshly eroded stream banks, the roots will be seen to be concentrated in the upper meter or so of the bank, although cases of very deep penetration are encountered. Estimate the root density and the size distribution of roots in each unit using Tables 5.8 and 5.9. Root cohesion will be estimated as part of the bank stability modeling in Chapter 6.

Table 5.8. Quantity of live roots. Modified from Schoeneberger and others (2002).

Few	Less than one per unit area
Common	>= 1 to < 5 per unit area
Many	>= 5 per unit area

Table 5.9. Root size. Modified from Schoeneberger and others (2002)

Fine	<2mm (1 square cm unit area)
Medium	>=2mm to <5mm (100 square cm unit area)
Coarse	>=5mm to <10mm (100 square cm unit area)
Very Coarse	>=10mm (1 square meter unit area)

Contacts
If exposed, contacts between units should be described by specifying whether the lower contact of each unit is sharp (<= 2 cm thick) or gradational (> 2 cm). If gradational, note the thickness of this transition interval.

NRCS Soils data
If a soil survey is available from the Natural Resources Conservation Service, determine the Soil Series, slope class, and modifiers for the site in question. Some stratigraphic sections may span more than one Series or at least more than one slope class. Record the Soil Series and the NRCS parent material. The NRCS parent material designation may not agree with the geologic interpretation from the field, but it can be of help in understanding the local relationship between NRCS Soil Series and the geologic parent materials As of this writing, NRCS has completed mapping of all of Vermont except for Essex County, which is still in progress.

Landscape position
Record the overall slope of the land at the site and the aspect (as an azimuth). Classify the landscape position using the classification below.

Interfluve / Hilltop / Hillslope / Ridge / Saddle / Talus / Terrace tread / Terrace riser / Floodplain / Stream bank / Valley bottom / Swale or gully / Depression / Other (describe)

Erosion at site
Sheet erosion is incipient erosion that has not yet developed permanent channels. Rills are small, incipient channels. Gullies have steep banks and can range from a foot or two in width to over 100 feet. If there is a gully or landslide, fill out the Slope Stability Data Sheet.

Slope failure
If a slope failure is observed, fill out the Slope Stability Data Sheet. Refer to Chapter 3 for classifications of slope failures.

Photos
Always include a scale and, if possible, a chalkboard or wipe-board with the site number, date, etc. A sketch showing the key features of the area photographed is very helpful. Carefully specify the location and orientation of the photo. Digital photos have become the standard and they certainly simplify record keeping and organization. However, they do not eliminate the need for writing down what is in each photo. You will never be in a better position to describe what's in the photo than immediately after taking it.

Columnar section or cross section
Space is provided for a columnar section or cross section to supplement the stratigraphic log. A measured cross section is usually needed for the slope stability modeling described in Chapter 6.

Interpretations
Make an effort to objectively describe the characteristics of the layers and the landforms. If the documentation is adequate, later investigators will then be able to make their own evaluations of the origin of the deposit. If you don't feel confident that you understand the origin of a particular unit or landform, refrain from speculating and just describe it.

Shear strength data
Shear strength data can be derived from a wide variety of *in situ* and laboratory tests. Undrained shear strength (S_u) data is relatively simple to obtain but is unfortunately not applicable to most of the materials encountered in Vermont stream banks. It is only applicable to saturated clays and silty clays when the rather special case that the angle of internal friction $\Phi = 0^{\circ}$ is reasonable. In these cases, Torvane™ and pocket penetrometer readings can give crude indications of the unconfined compressive strength (qu), from which S_u can be calculated with the following formula under the assumption that $\Phi = 0^{\circ}$

$$S_u = C = \tfrac{1}{2}q_u \qquad \text{Equation 5.1}$$

Much more accurate data can be obtained from laboratory tests of unconfined compressive strength or field or laboratory vane shear tests. For other materials, triaxial or direct shear tests are required. In all cases, the laboratory measurements must be made on relatively undisturbed samples. See Wu (1996) and Renteria (1994) for information on testing of soil shear strength.

Slope Stability Data Sheet
See Appendix B for the data sheet. Most of the items on the data sheet have already been discussed in Chapters 3 and 4.

6. Stream Bank Stability Evaluation Method

General Statement

The overall stability at a stream bank site can be assessed by combining slope stability analysis and calculation of the rate of fluvial erosion. The analysis needs to take into account changing surface water and ground water conditions, the initial stability of the slope, the ability of the stream to remove failed or unfailed material through erosion due to excess shear stress. Following erosion of material by the stream, the stability of the slope will once again need to be calculated. Fresh materials exposed at the surface following a bank failure episode may, over time, undergo weathering processes which substantially soften them, thus adding to the likelihood of continued bank failure. In the real world, the stability of a stream bank is probably constantly fluctuating due to changes in a variety of factors as described in Chapter 3. At the same time, the rate of erosion of particles or chunks from the bank is also constantly in flux (see Chapter 4). Due to the difficulties of modeling turbulent stream flow, the rate of erosion is probably even more subject to uncertainty than the stability of the slope. However, even approximate calculations can be of some value in understanding the overall stability of a stream bank and understanding the effects of proposed channel and bank stabilization treatments. This chapter describes three methods for calculating stream bank stability by means of geotechnical methods, one of which includes a toe erosion component and allows iterative calculation of episodes of bank failure and toe erosion.

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Figure 6.1. Flow chart to be created to show procedure to follow for undertaking stability and toe erosion modeling.

All of the slope stability models described below are limit equilibrium models; that is models that test the balance of driving forces against resisting forces. On any stable slope there is a balance between the forces that hold the soil horizons in place and the forces that tend to drive the soil down the slope. If the resisting forces are decreased (perhaps by removal of material at the toe or by increased pore pressure), then the balance can tip in favor of instability. Failure will also become more likely if the driving force is increased, perhaps by increased loading on the slope. See the discussion on factors influencing slope failure in Chapter 3. Each of the models considers only those forces acting within the plane of a vertical cross-section that is roughly parallel to the direction of movement. In some cases we will assume a homogeneous material. In others we will be able to consider several layers. Thus, all of the models make simplifying assumptions about the slope conditions and must be used with full awareness of their limitations.

Each of the methods described below will result in an estimate of a Factor of Safety (FS). Values below 1.0 indicate that, according to the model, driving forces exceed resisting forces and failure will occur. Values above 1.0 indicate the opposite. Although each of the methods will generate FS values to one or more decimal places, the interpretation of their significance is not straightforward. If a model correctly represents the mode of failure occurring on the stream bank and the necessary parameters have been accurately measured, FS values greater than, say, 1.2 to 1.25 may indicate great stability. However, if the model has been misapplied or some critical parameter is wrong, the FS value may be misleading no matter how high it is.

A further point is that the limit equilibrium models can only tell us whether or not failure will occur on a slope with a given set of conditions. Although they can provide an estimate of the location and shape of a failure surface, they cannot directly quantify the amount of movement that would then occur (Duncan, 1996). To begin to quantify the deformation on a failing slope, it is necessary to move to the more sophisticated finite element analysis approach, one that is too complex for the present purpose. Fortunately, there is some justification in taking the simpler, limit equilibrium approach; namely that once a landslide occurs on a stream bank, the slide materials are so weakened that they will generally slough down as quickly as the stream can sweep them away. Thus, a limit equilibrium model can be used to calculate the volume of material that will comprise a slide block and then stream flow and soil erodibility data can be used to calculate the rate of removal of slide material.

Deterministic versus Probabilistic Analysis

Stability analyses can be divided into two broad classes: Deterministic and probabilistic. These are described briefly below and in more detail in Hammond and others (1992, especially Chapter 1). The deterministic analysis uses single input values for each of the factors and calculates a single result. The probabilistic analysis uses variable input numbers and is run multiple times. Many combinations of the different input numbers are evaluated and a distribution of factor of safety values is produced. If exact values could be obtained for the soil shear strength parameters, ground water conditions, and other factors, the deterministic analysis would be the simplest way to evaluate slope stability and in the past it has been the most common approach used in geotechnical studies of slope stability. However, the reality is that the values of the factors influencing slope stability are anything but constant. In recent years the ease with which computer models can be run has made it much more feasible to vary the input data and look at the distribution of results. Since it is well known that soil strength parameters and ground water conditions vary at a site both in space and over time, and both field and laboratory measurements have their own variability, probabilistic analysis does show considerable promise.

It is important to understand that both the deterministic and probabilistic methods are attempts to deal with variable slope conditions. Prior to computer-aided calculations, the best way to do this was to run a deterministic method with conservative assumptions and to not consider a slope stable unless the FS was somewhere well above 1.0 (perhaps 1.2 or 1.3). In recent decades, computerized models of both sorts give the user the ability to vary the parameters and see how the FS varies. The particular advantage of the probabilistic analysis adds is the ability to systematically vary the parameters, *in the way they are believed to vary in the field*, to quantify the likelihood that slope failure will occur. However, for a probabilistic analysis to work, there must be a knowledge of the likely distributions of values for the parameters. Without rational justifications for the input numbers, any of these methods could become what Hammond and others call a "...game of numbers" (1992, p.7). The trial runs on Vermont stream banks that are described below indicate that the method can work reasonably well.

Although probabilistic analyses could be used with any slope stability models, the only one available for use with this study is the version of the infinite slope model known as LISA 2.0 (Hammond and others, 1992). The other models are deterministic.

Back-calculation

Because of the difficulty in obtaining all of the necessary stability parameters for a slope failure, it is often very helpful to employ back-calculation to supply a missing value. In this technique, reasonable measurements or estimates are entered for all but a single parameter, the Factor of Safety is assumed to be 1.0 at time of failure, and the missing value is calculated. In conjunction with the sensitivity analysis described below, this can provide a check on measured values or those derived from charts of likely values.

Sensitivity Analysis

Sensitivity analysis involves applying a computer model to a slope in such a way that the FS value is near 1.0 and then the input parameters are varied through reasonable ranges to see which ones have the greatest influence on changing the slope from "stable" to "unstable" or vice versa. This method commonly shows that parameters such as slope angle, pore water conditions, and cohesion (soil and/or root) cause the factor of safety to vary more than parameters such as tree surcharge and soil bulk density. The analysis is helpful in order to help decide which parameters should be accurately measured and which can be adequately estimated from charts and tables. The tables in Appendix D can be used to supply some initial values.

Slope Conditions

In order to undertake any sort of bank stability analysis it is necessary to have some idea of the slope geometry and the geotechnical characteristics of the materials underlying the bank. Chapter 5 outlined procedures for gathering field data, including the Unified Soil Classification. Based on the field data it is possible to make some first estimations of geotechnical properties using the compilations in Hammond and others (1992, Chapter 5, especially Tables 5.3 and 5.5), and Appendix D in this report. Whenever possible, this data should be supplemented or replaced with site-specific data.

Slope Geometry

Slope geometry is often among the best known slope parameters, but even here, care needs to be taken to correctly evaluate the situation. Three basic scenarios are outlined below. In most cases a sufficiently accurate profile can be measured with a tape and clinometer. See Chapter 5 for further details.

1. If an uneroded bank is being investigated to see if it is currently stable, then initial modeling will use the un-eroded geometry. If the stability after erosion is to be evaluated, see the next item.

2. If a stream is rapidly eroding away the toe of a bank, then the geometry of the freshly eroded bank is what should be input into the slope stability model. This case is analogous to a fresh artificial excavation, a situation normally dealt with in slope stability analysis by use of peak strength values.

3. In any situations where the toe of the slide is being eroded away, the slope has to be considered to be poised for further failure. The only cases where this would not be true would be those where the slope was unloaded or drained after the failure had taken place. Even then, toe erosion might well cause renewed instability. If a stream has eroded a bank, which has then collapsed, and the slide material is still in place, then the situation is more complicated. If toe erosion is nonexistent, some slides may fave fairly high post-failure factors of safety. However, slides with a steep head scarp may still need to collapse back until the slope is laid back to a gentler angle. In other cases, the slide may retrogress due to headcutting from surface water running out onto the slide or from ground water sapping or piping. In these last cases, the end result could be gully formation.

Geotechnical Properties

Note that soil depth as used in the infinite slope equation is measured vertically (**not** perpendicular to the slope).

Tree surcharge can be measured but at least a preliminary estimate can be obtained by use of Hammond (1992, Chapter 5).

Root cohesion is estimated from Hammond (1992, Chapter 5).

Dry unit weight can be estimated if material has been classified using the Unified Soil

Classification System. See Hammond and others (1992, Chapter 5, especially Tables 5.3 and 5.5). Additional tables of typical unit weights are given in Section 4B.4 of Rose (1994).

The laboratory measurement of soil moisture content is relatively simple and at least a few samples should be run for each site.

The specific gravity of soil particles is needed for input into the LISA 2.0 program. In Vermont, this varies over a rather limited range and in the absence of site-specific data it is probably sufficient to use values of 2.65 for sands and coarse silts and 2.75 to 2.85 for finer materials.

Drained versus Undrained Conditions

A soil that does not drain readily (such as silty clay or clay) may develop an excess of pore pressure in some parts due to an applied load. Such a soil is said to respond in an undrained fashion. Better-draining sands and gravels that can dissipate such pressures as fast as they are applied are said to be in a drained condition. Materials such as silty clays and clay with low permeability ($<10^{-7}$ cm/sec) will normally be treated as undrained during slope stability analysis while those with high permeability ($>10^{-4}$ cm/sec) will drain rapidly if the water table is lowered and will normally be considered drained. Silts, which have intermediate permeabilities, will vary in their response depending on their permeability and the rate of loading (Duncan, 1996). As described in Chapter 3 and Appendix C, excess pore pressure can significantly reduce the effective normal stress and thus reduce the shear strength of a material.

Fully saturated soils that are loaded while in an undrained state can be modeled with the angle of internal friction $\Phi = 0^{\circ}$ and cohesion $C = 1/2q_u$ (Duncan, 1996). On natural slopes this may be a comparatively rare case, perhaps limited to rapid loading of soft, saturated clays and silty clay. This appears to be about the only natural situation where the simple pocket penetrometer and Torvane can provide appropriate, albeit rough, shear strength measurements (see Chapter 5).

Total Stress versus Effective Stress

For the most part, effective stress shear strength parameters should be used for natural slope stability modeling (Hammond and others, 1992, p. 69). Total stress shear strength parameters appear to be limited to rapid loading of undrained, saturated soft clays. An eroded stream bank in clay may need to be modeled using both sets of strength parameters, depending on whether it is short-term or long-term stability that is being considered. In the short term the bank may behave in an undrained fashion and be modeled using the $\Phi = 0^{\circ}$ and $C = 1/2q_u$ total stress parameters. In the long term that bank will probably behave in a drained fashion and effective stress parameters will be used.

Peak versus Residual Shear Strength

As described in Appendix C, shear strength tests of soil commonly rise up to a peak value and then show a considerable drop in strength at higher strains. The peak values appear to be most applicable to intact materials that have not previously been subjected to slope failure while the residual values are more applicable to continued failures of existing landslides (Goudie, 1990).

Ground Water

As discussed in Chapter 3 and Appendix C, increased pore pressure can markedly decrease the stability of a stream bank. All of the slope stability models used here make at least some use of pore pressure data and the results tend to be quite sensitive to changes in pore pressure. Therefore, actual pore pressure data should be obtained whenever possible. Discussions of the design and installation of standpipes and piezometers for pore-pressure measurements are found in several chapters in Turner and others (1996). In the absence of monitoring devices, much can be learned about water conditions within the slope by looking for signs of seepage. There is often a depth below which the material is saturated.

In most of the models described below, the effect of ground water is limited to a positive, hydrostatic pore pressure. Seepage pressures due to flowing water are not considered and except in the case of the ARS Bank Stability and Toe Erosion Model, negative pore pressures are not taken into account.

There appears to be a marked contrast in the extent of soil saturation needed to initiate slope failure between the shallow translational slides on the one hand and the deeper rotational slides and the planar slides on the other hand.

Field observations in Vermont suggest that the slopes need to be quite wet before extensive failure will occur, and thus the assumption of full saturation is probably a good starting point for modeling. In support of this idea, the LISA 2.0 model runs conducted for this study indicate that shallow translational landslides on steep banks appear to require at least nearly complete saturation before failure will occur (see the sections on Stetson Brook and Great Brook below). This is consistent with the statement by Sidle and Ochiai (2006, p. 82) that "[m]any steep hillslopes or hollows with shallows soils (<1 to 2 m deep) may require nearly saturated or even artesian conditions to induce slope failure." Note that Sidle and Ochiai are discussing thin soil over bedrock while the typical shallow translational slide in Vermont forms in the upper 1 to 2 meters of somewhat weathered till overlying relatively fresh and very dense till. In the Vermont case the failure occurs at the contact between the weaker weathered till above and the stronger fresh till below. Note, however, that some recent research does suggest that more complex "variably saturated flow" models may be needed to accurately model shallow translational slope failures (Dutton and others, 2005).

In marked contrast to the shallow translational slides, model runs for the deep rotational landslides in a variety of cohesive and non-cohesive materials and the wedge or planar slides in non-cohesive materials suggest that they will often fail long before full saturation of the entire slope is achieved. Instead, saturation of the lower parts of the slope is sufficient to induce instability.

Slope Stability Modeling

The slope stability models described here are for three types of slope failure: shallow translational slides, deeper rotational slides, and planar slides using slices. The infinite slope model for shallow translational slides is described in detail, partly because of its relative simplicity and partly because of it's widespread applicability. Two broad approaches to solving the deeper rotational slides are described: The use of stability charts and the analysis of rotational slides by subdivision of the failing soil mass into slices. Finally, a special case of the slice technique for planar slides is briefly discussed.

Translational Slides--Infinite Slope Model

Slope failures that are planar in cross section and are long relative to their depth can be modeled using the infinite slope equation as described in Hammond and others (1992) and Prellwitz (1994). The geometry is shown in Figure 6.2. It is assumed that the ground surface, the failure surface, and the groundwater surface (phreatic surface) are all approximately parallel. It is also assumed that the material can be modeled as a single homogeneous layer.

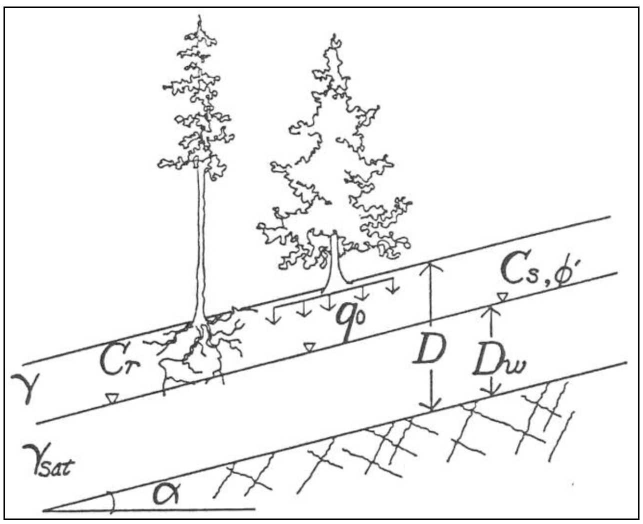


Figure 6.2. Geometry of the infinite slope model. See text for description. From Hammond and others, (1992, Figure 3.1).

The infinite slope model is well-suited to analyzing the factor of safety of slopes that have soil horizons developed roughly parallel with the ground surface, overlying unaltered parent material below, such as the relatively thin slope failures on stream banks that are composed of dense till (described in Chapter 3).

$$FS = \frac{C_r + C'_s + \cos^2\alpha\,[q_0 + \gamma(D - D_w) + (\gamma_{sat} - \gamma_w)\,D_w]\tan\phi'}{\sin\alpha\cos\alpha\,[q_0 + \gamma(D - D_w) + (\gamma_{sat} D_w]} \qquad \text{Equation 6.1}$$

where FS = factor of safety

α = slope of the ground surface, phreatic surface and failure surface (degrees)

D = total vertical soil depth (ft)

D_w	=	saturated soil thickness above failure surface (ft)
q_0	=	tree surcharge (psf)
C_r	=	tree root strength as cohesion (psf)
C'_s	=	effective soil cohesion (psf)
ø'	=	effective soil angle of internal friction (degrees)
γ	=	moist soil unit weight (pcf)
γ_{sat}	=	saturated soil unit weight (pcf)
γ_w	=	unit weight of water (pcf)

LISA and DLISA

A relatively easy-to-use computerized version of the infinite slope model known as Level I Stability Analysis Version 2.0 (LISA 2.0) has been produced by the U.S. Department of Agriculture, Forest Service. The program is available as both probabilistic and deterministic versions and is described in detail in Hammond and others (1992). The probabilistic program (LISA) repetitively calculates a factor of safety as input variables are varied through likely ranges of values. The output from this Monte Carlo simulation technique can be used to estimate probability of failure. The deterministic version (DLISA) is helpful for running sensitivity analyses in order to help identify the most critical slope parameters.

Example outputs for three eroding banks are given in the next section. Note that a single run of 1000 iterations is presented for each of the four sites. Because this is a Monte Carlo simulation, the resultant probability of failure will vary from run to run, even though the input variables and their distributions remain the same. Therefore, the program should be run multiple times for each study site in order to explore the variation in possible outputs. See Hammond and others (1992, Chapter 1) for more explanation.

An example of a sensitivity analysis from Hammond and others (1992) is given in Figure 6.3. This shows that the factor of safety varies most rapidly with changes in slope (in this case α) and the ratio of the height of the phreatic surface (ground water table) to soil depth (*Dw/D*).

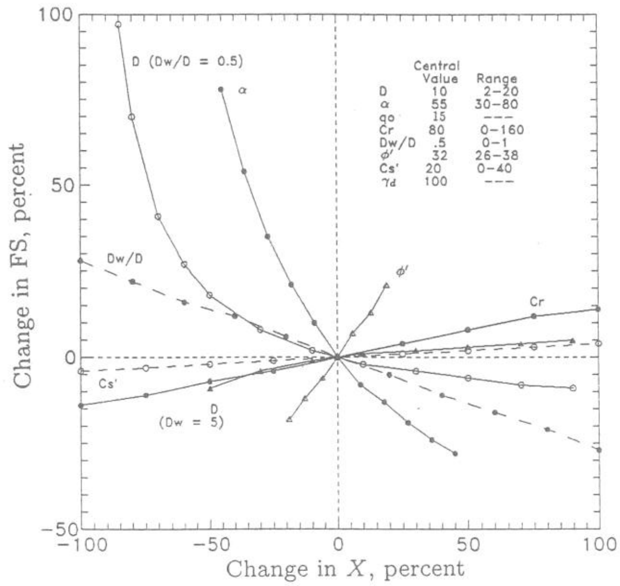


Figure 6.3. Example sensitivity analysis output from DLISA. Systematic changes in input parameters are examined in order to determine which parameters have the most influence on changing the factor of safety. In this particular example, the factor of safety is highly influenced by changes in the slope (α) and the ratio of the height of the phreatic surface to soil depth (Dw/D). A value of Dw/D = 1 indicates saturation up to the ground surface. From Hammond and others, 1992, Figure 3.2).

The Stetson Brook study site is a shallow translational landslide in dense till. The brook is a tributary of the Mad River located in the southwestern corner of Warren in Washington County (Figure 6.4). The watershed of the site is forested and is entirely within the Green Mountain National Forest. The site is on the north side of the brook, on the downstream half of an outside bend of the stream. The slope is about 35 feet high and the slide surface has an angle of 38º (78%) where not mantled with slide deposits (Figure 6.5). The upper portions of the slope are underlain by a typical dense, clast-rich, silt-matrix till. Near the base of the slope the till has a higher clay content and contains few large clasts. This clayey silt-matrix till at the base may have formed from a readvance of a glacier over earlier clay-rich lacustrine deposits. Although an old Forest Service road is located at the top of the slide, the presence of the road does not appear to be the primary cause of instability at this site. Instead, this and other landslides in the watershed were activated (or at least reactivated) by severe flooding in June of 1998 (Kathy Donna, U.S. Forest Service, personal communication, 2003). It appears that toe erosion and high ground water levels led to the failure. The Forest Service attempted to revegetate the slope in the fall of 2001 but the stream still impinges on the toe of the slope and as the fresh till has softened, flows and slides have continued. With the sliding off of the soil and trees, incision into the unweathered till is rather slow, perhaps largely occurring gradually as the material weathers. If the stream shifts it's position at the base it is expected that new sections of soil and trees on the bank will be destabilized.

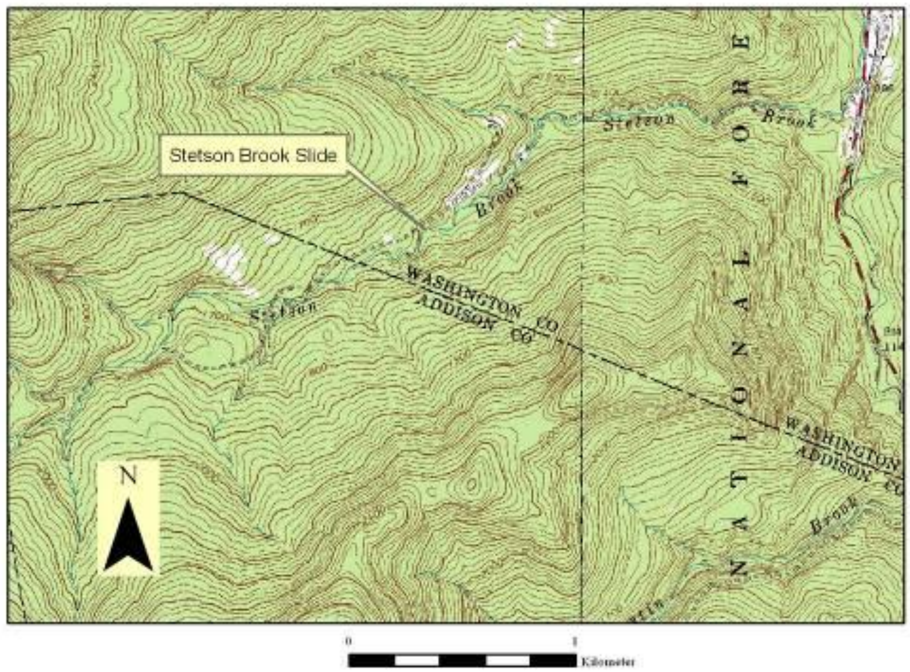


Figure 6.4. Location map for landslide in till on north bank of Stetson Brook, Warren, Lincoln and Warren quadrangles.

Figure 6.5. Stetson Brook, Warren. Shallow translational slide in till on the north side of the brook. Looking downstream at the slide cutting into a Forest Service road, 6/13/2003. Note the constant slope and shallow depth. This is characteristic of slides on dense till.

In order to conduct the stability analysis it was necessary to come up with values for all of the variables in equation 6.1. Field measurements supplied the values for soil depth and ground slope and laboratory tests were undertaken to determine dry unit weight, moisture content and specific gravity of particles. Tree surcharge and root cohesion were estimated from the literature (Hammond and others, 1992). In some materials it would be very feasible to determine friction angle and soil cohesion from laboratory or field measurements but the dense tills are extremely difficult materials to work with. The values were instead chosen from the literature (Hammond and others, 1992). See Appendix C for more discussion on this topic. The ground water ratio (water table depth divided by soil depth, both measured vertically) was estimated to be about 1.0 based on the observation that such slopes in Vermont can often be saturated without failure as long as no additional destabilizing influence occurs.

The first model for this site is an estimate of pre-failure conditions (Table 6.1). The bulk density of partly weathered till was used (this is probably somewhat of an overestimate, but no value was available for a heavily weathered till). With a reasonable friction angle of 33º it was necessary to fully saturate the soil and to assume quite low cohesion values in order to induce failure. This model run suggests that the slope would perhaps not have failed if it could have been better drained. It would be worth investigating how much water is brought onto this section of the slope by the road at the top.

A sensitivity analysis of the effect of changes in soil cohesion on the factor of safety is shown in Table 6.2. All other values were held at the mean conditions shown in Table 6.1 while effective soil cohesion was varied from 0 to 250 psi. This sort of analysis is very helpful in determining which factors have the most influence on the stability of the slope.

Table 6.1. Example output from the LISA 2.0 infinite slope stability analysis computer program for pre-failure conditions at the landslide on Stetson Brook in Warren.

User name	George Springston
Time of simulation	02-09-2007 12:57:03
Map unit	STETSO.MPU
Number of iterations	1000
Random number seed	366710841
Probability of failure	**.441**

INPUT DATA

NATURAL DATA

Soil depth	(ft)	Uniform	Min.:	3.00	Max.:	5.00
Ground slope	(%)	Constant	Value:	78.00		
Tree surcharge	(psf)	Normal	Mean:	20.00	Std.:	3.00
Root cohesion	(psf)	Normal	Mean:	50.00	Std.:	5.00
Friction angle	(deg)	Normal	Mean:	33.00	Std.:	1.00
Soil cohesion	(psf)	Normal	Mean:	100.00	Std.:	10.00
Dry unit weight	(pcf)	Normal	Mean:	118.00	Std.:	3.00
Moisture content	(%)	Normal	Mean:	16.00	Std.:	2.00
Specific gravity		Constant	Value:	2.82		
Groundwater ratio	(Dw/D)	Constant	Value:	1.00		

DESCRIPTIVE STATISTICS OF SIMULATED VALUES -- NATURAL SLOPE

		MINIMUM	MAXIMUM	MEAN	S.D.
Soil depth	(ft)	3.00	5.00	3.99	0.58
Ground slope	(%)	78.00	78.00	78.00	0.00
Tree surcharge	(psf)	10.73	29.27	19.83	2.88
Root cohesion	(psf)	34.55	65.45	50.08	4.95
Friction angle	(deg)	29.92	35.88	33.01	1.04
Soil cohesion	(psf)	69.10	129.15	99.60	10.29
Dry unit weight	(pcf)	108.77	126.64	118.02	3.12
Moist unit weight	(pcf)	123.83	144.13	136.35	3.63
Saturated unit wt.	(pcf)	132.60	144.13	138.57	2.01
Moisture content	(%)	10.21	21.69	15.94	2.05
Groundwater ratio	(Dw/D)	1.00	1.00	1.00	0.00
Factor of safety		0.81	1.34	1.03	0.10

Histogram of natural slope factor of safety

Range	# Values	
0.81 - 0.86	17	===
0.86 - 0.91	69	============
0.91 - 0.95	167	=============================
0.95 - 1.00	199	===================================
1.00 - 1.05	159	============================
1.05 - 1.10	145	==========================
1.10 - 1.15	112	====================
1.15 - 1.20	75	=============
1.20 - 1.24	33	======
1.24 - 1.29	20	====
1.29 - 1.34	4	=

-------------------------------- Histogram Statistics ------------------

Number of iterations	**: 1000**	**Sample minimum**	**: 0.81**
Sample mean	**: 1.03**	**Sample maximum**	**: 1.34**
Sample median	**: 1.02**		
Sample standard deviation	**: 0.10**		
P[FS <= 1]	**: 0.441**		

Table 6.2. Sensitivity analysis of change in factor of safety with changing soil cohesion, Stetson Brook site. All factors have been held at the mean values used in the pre-slide model described above, except for soil cohesion.

Effective soil cohesion (psf)	Factor of safety
0	0.65
50	0.83
100	1.01
150	1.19
200	1.37
250	1.55

The second model run shows one scenario for conditions on the slope once failure has occurred on the slope (Table 6.3). The probability of failure has been reduced to near zero by decreasing the depth of soil involved in failure and eliminating tree surcharge, despite elimination of root cohesion and increasing the soil bulk density. Note that this apparent increase in stability was achieved even without increasing the friction angle and soil cohesion, factors that are quite likely to be greater in the unweathered till. This is all not to say that material will not move off of the slope. Heavy rains will wash material off of even the unweathered sections and most certainly the freshly exposed dense till will weather, softening over the course of months and seasons. Whether this softened material will build up on the slope until it fails as a sheet or whether it will instead be washed off as one or more debris flows is dependent on many factors. The important point of this model run may be that the slope failure has at least temporarily increased the stability of the deposit in terms of a large-scale failure.

Observations at this and other eroding dense till sites in Vermont do indeed suggest that the failure mode shifts from translational slide to debris flow following removal of the mantle of soil and trees. This appears to be due to an increase in the shear strength parameters as the translational slide has taken away the weaker, weathered material and exposed the stronger, unweathered till below. The unweathered material can indeed be made to fail, but it will commonly take even greater undercutting by the stream and/or some weathering to bring it about. Thus, on this particular section of the stream bank it is likely that increments of weathering and/or toe erosion will be necessary to initiate new slope movement.

The final dense till site is located on Great Brook in Plainfield in Washington County (Figure 6.6). This is one of many large landslides along the brook and is representative of many slides in dense till throughout the state. The till has a silt-rich matrix and has abundant clasts. When fresh it is very dense and almost impossible to dig with a shovel. At this site a mantle of weathered till is accumulating on the lower portions of the slide (Figure 6.7). Although most of this may be debris flow material, some may be till exposed by landsliding that has subsequently weathered in place. The stability analysis presented in Table 6.4 is intended to represent conditions in the weathered till out on the slide surface.

The slide parameters were obtained in a similar fashion to those at the Stetson Brook site, although back-calculation was used to refine the estimate of the friction angle. As personal observations over the last nine years indicate that the weathered portions of this slope sporadically fail in response to heavy rains and high stream flow, it is reasonable to assume that the factor of safety fluctuates in the vicinity of 1.0. Assuming this and entering values for all of the other factors based on measurements and estimates from the literature, the soil cohesion values were varied until a reasonable friction angle was obtained. Note that these are similar values to those used in the Stetson Brook models.

Although the shallow, translational slides described above are the common failure mode in dense till, it is possible for deeper slides to occur in this material. If erosion at the base of a slope causes translational landsliding of a surface layer of weak weathered till, and the slide deposits are swept away, the remaining dense till may be fairly stable at that slope angle. However, if toe erosion continues, a point may be reached at which the till will fail, perhaps to a much greater depth than in the initial failure. These deep failures appear to be uncommon but, when encountered, they need to be modeled with the rotational slope failure methods described in the next section.

Table 6.3. Example output from the LISA 2.0 program for post-failure conditions at Stetson Brook. Note reduced probability of failure due to decrease in soil depth and reduced tree surcharge, despite the reduced root cohesion and increased soil unit weight.

```
User name                        George Springston
 Time of simulation               02-30-2007 20:07:34
 Map unit                         STETSO.MPU
 Number of iterations             1000
 Random number seed               581539332
 Probability of failure                   .001
                                     INPUT DATA
NATURAL DATA
 Soil depth             (ft) Uniform    Min.:    1.00   Max.:    2.00
 Ground slope            (%) Constant   Value:  78.00
 Tree surcharge        (psf) Constant   Value:   0.00
 Root cohesion         (psf) Constant   Value:   0.00
 Friction angle        (deg) Normal     Mean:   33.00   Std.:    1.00
 Soil cohesion         (psf) Normal     Mean:  100.00   Std.:   10.00
 Dry unit weight       (pcf) Normal     Mean:  134.00   Std.:    3.00
 Moisture content        (%) Normal     Mean:   16.00   Std.:    2.00
 Specific gravity            Constant   Value:  2.82
 Groundwater ratio (Dw/D) Constant   Value:   1.00

 DESCRIPTIVE STATISTICS OF SIMULATED VALUES -- NATURAL SLOPE
                              MINIMUM    MAXIMUM       MEAN        S.D.
 Soil depth             (ft)     1.00       2.00       1.49        0.29
 Ground slope            (%)    78.00      78.00      78.00        0.00
 Tree surcharge        (psf)     0.00       0.00       0.00        0.00
 Root cohesion         (psf)     0.00       0.00       0.00        0.00
 Friction angle        (deg)    29.91      35.56      33.04        0.99
 Soil cohesion         (psf)    69.10     130.90      99.72       10.00
 Dry unit weight       (pcf)   124.73     141.69     134.12        2.96
 Moist unit weight     (pcf)   141.34     153.84     148.95        1.93
 Saturated unit wt.    (pcf)   142.90     153.84     148.96        1.91
 Moisture content        (%)     9.82      22.18      16.05        2.00
 Groundwater ratio (Dw/D)       1.00       1.00       1.00        0.00
 Factor of safety                0.98       2.12       1.45        0.22

        Histogram of natural slope factor of safety
        Range        # Values
   0.98 -    1.08       9      ==
   1.08 -    1.18      83      ===============
   1.18 -    1.29     189      ===================================
   1.29 -    1.39     185      ==================================
   1.39 -    1.50     150      ============================
   1.50 -    1.60     138      ==========================
   1.60 -    1.70      89      ================
   1.70 -    1.81      79      ===============
   1.81 -    1.91      42      ========
   1.91 -    2.02      29      =====
   2.02 -    2.12       7      =
------------------------------- Histogram Statistics ------------------
Number of iterations      :  1000       Sample minimum     :    0.98
Sample mean               :    1.45     Sample maximum     :    2.12
Sample median             :    1.42
Sample standard deviation:     0.22
P[ FS <= 1 ]              :    0.001
```

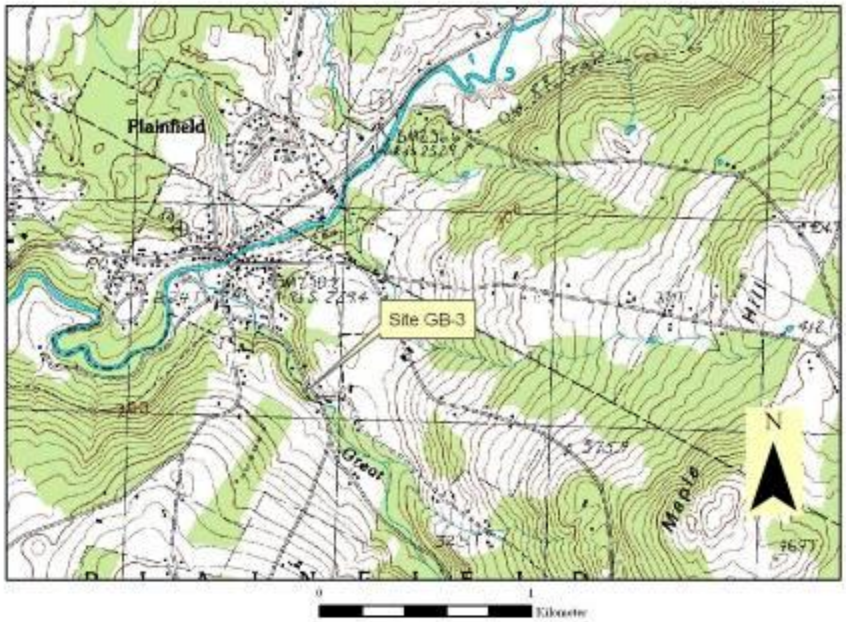


Figure 6.6. Location map for landslides in dense till on west bank of Great Brook, Plainfield, Plainfield quadrangle.

Figure 6.7. Landslide in dense till at Site GB-3, Plainfield.

Table 6.4. Example output from the LISA 2.0 program for Site GB-3 on the west side of Great Brook in Plainfield.

```
User name                      George Springston
 Time of simulation             02-09-2007 12:28:17
 Map unit                       GB-3.MPU
 Number of iterations           1000
 Random number seed             125892281
 Probability of failure                .483

                                  INPUT DATA
NATURAL DATA
Soil depth            (ft) Uniform    Min.:    3.00   Max.:    5.00
 Ground slope           (%) Constant   Value:  78.00
 Tree surcharge       (psf) Constant   Value:   0.00
 Root cohesion        (psf) Constant   Value:   0.00
 Friction angle       (deg) Normal     Mean:   33.00   Std.:    1.00
 Soil cohesion        (psf) Normal     Mean:  140.00   Std.:   10.00
 Dry unit weight      (pcf) Normal     Mean:  118.00   Std.:    3.00
 Moisture content       (%) Constant   Value:  16.40
 Specific gravity           Constant   Value:  2.82
 Groundwater ratio (Dw/D) Uniform      Min.:    0.90   Max.:    1.00

 DESCRIPTIVE STATISTICS OF SIMULATED VALUES -- NATURAL SLOPE
                                 MINIMUM    MAXIMUM       MEAN        S.D.
 Soil depth             (ft)       3.00       5.00        3.99        0.57
 Ground slope            (%)      78.00      78.00       78.00        0.00
 Tree surcharge        (psf)       0.00       0.00        0.00        0.00
 Root cohesion         (psf)       0.00       0.00        0.00        0.00
 Friction angle        (deg)      29.91      35.95       33.04        0.99
 Soil cohesion         (psf)     109.62     170.90      140.54       10.12
 Dry unit weight       (pcf)     108.73     126.84      118.11        2.97
 Moist unit weight     (pcf)     126.56     144.26      137.27        3.13
 Saturated unit wt.    (pcf)     132.57     144.26      138.63        1.91
 Moisture content        (%)      16.40      16.40       16.40        0.00
 Groundwater ratio (Dw/D)          0.90       1.00        0.95        0.03
 Factor of safety                  0.82       1.27        1.01        0.09

        Histogram of natural slope factor of safety
        Range        # Values
   0.82 -    0.86      17      ====
   0.86 -    0.90      93      ====================
   0.90 -    0.95     163      ===================================
   0.95 -    0.99     152      =================================
   0.99 -    1.03     167      ====================================
   1.03 -    1.07     130      ============================
   1.07 -    1.11     114      =========================
   1.11 -    1.15      80      =================
   1.15 -    1.19      53      ===========
   1.19 -    1.23      26      =====
   1.23 -    1.27       5      =
------------------------------- Histogram Statistics -------------------
Number of iterations      :  1000     Sample minimum     :    0.82
Sample mean               :    1.01   Sample maximum     :    1.27
Sample median             :    1.00
Sample standard deviation:     0.09
P[ FS <= 1 ]              :    0.483
```

Rotational Slides

The deeper sorts of slope failures encountered in Vermont are commonly rotational failures. Often they can be classified as rotation slides in their upper parts and flows in the lower parts. The flow in the lower portion results from a severe loss of strength during the course of the deformation. Some of these slides can be approximated as failures along segments of a circle while others are more complex.

In an attempt to keep the analysis of rotational slides as simple as possible, the focus will be on three of the many slope stability modles: the slope stability chart methods of Duncan (1996) and Michalowski (2002) and Bishop's modified method of slices (Duncan, 1996). Anyone actually undertaking slope stability analysis is advised to look beyond this report for more detailed discussions. An excellent overview of these and other methods is given by Duncan (1996). The purpose here is only to introduce the reader to the structure of these models and to provide example results for typical Vermont stream bank slope failures.

If their limitations are adhered to, the stability charts can provide a relatively simple way to evaluate the stability of a slope. The stability chart method is most applicable for slopes that have a simple geometry, simple or at least horizontally layered materials, and, usually, a circular failure surface or slip circle. The charts allow for consideration of tension cracking at the back of the slide, variable pore water conditions, and can provide the approximate location of the critical failure surface. A variation in shear strength properties can be dealt with by an average based on the length of critical failure surface that is subtended within each material. A weighted average of the unit weights can be determined from ratios of thickness overlying the critical failure circle. Details can be found in Duncan (1996).

Several methods of calculating limit equilibrium analyses have been developed in the last few decades (Duncan, 1996). Hand calculations of models are possible and are helpful for validating models, but most slope stability calculations are now done with computers. The computer program STABLE.2002 by MZ Associates is used in this report. It was chosen mostly because it was available as freeware on the Internet and is reasonably capable. The program includes modules for the methods of Bishop, Morgenstern-Price, and Sarma. Once slope characteristics, ground water conditions, and material properties are entered, the program will calculate a number of iterations in order to find the fracture circle with the lowest factor of safety. This, along with the locations of other slip circles, can be displayed on a simple cross section of the slide. The program is fairly simple to learn to operate.

The eroding bank at the Randolph Landfill has already been described in Chapter 4. Slope stability models were run using the STABLE computer program, as well as the slope stability charts of Duncan (1996, after Janbu, 1968) and Michalowski (2002). All gave roughly similar results. A comparison of the results of these methods is shown in Table 6.5. The output from STABLE is shown in Figure 6.8.

The Winooski River site is located in Plainfield on the north bank of the Winooski River (Figure 6.9). The bank is 37 feet high with an overall angle of 39º. The materials consist of 33 feet of varved, lacustrine silt and silty clay overlain by 4 feet of pebble-cobble gravel, medium sand, and silt. The upper 2/3 of the bank is shown in Figure 6.10. Erosion has been underway at the site for many years. Slope stability models were run using the STABLE computer program, a hand calculation of the Ordinary Method of Slices, and the slope stability charts of Michalowski (2002). The results are shown in Table 6.5 and as with the Randolph site, the three methods gave reasonably similar results. The output from STABLE for the Winooski River site is shown in Figure 6.11.

Table 6.5. Summary of slope stability modeling for Randolph Landfill and Winooski River sites.

****

****

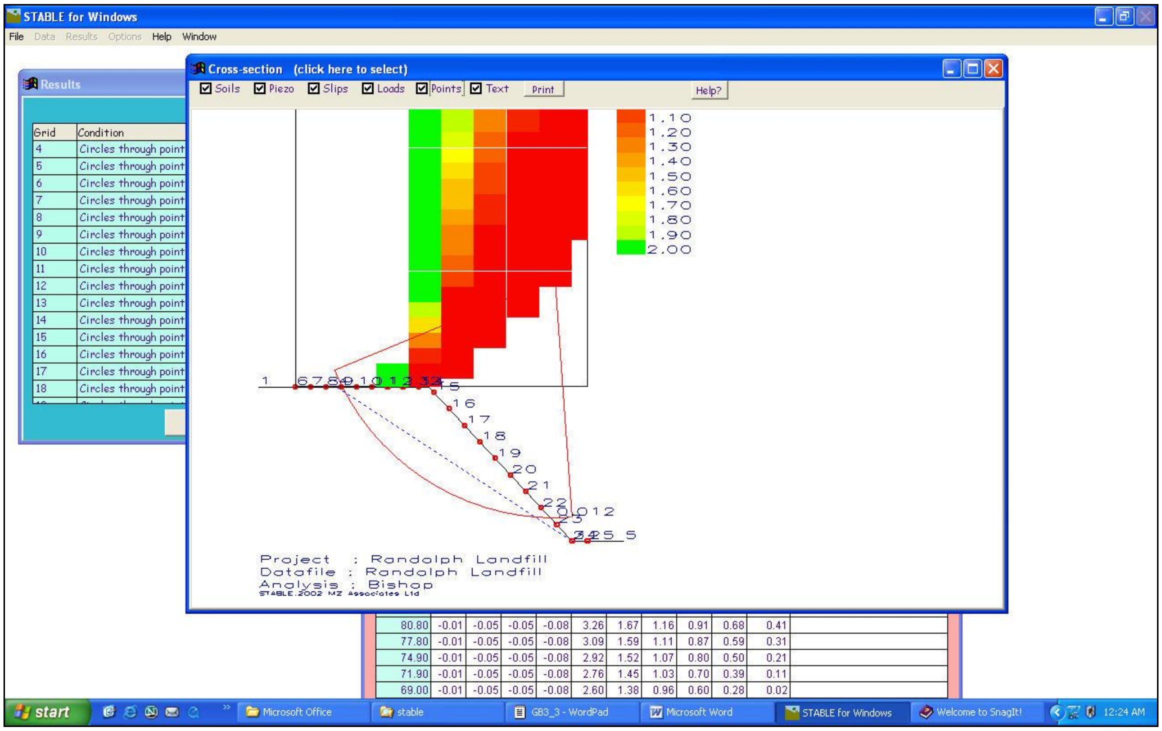


Figure 6.8. Image of output screen for the slope stability modeling program Stable. The site is at the Randolph Landfill on the Third Branch of the White River. The program generated 20 slip circles using Bishop's method and plotted the circle with the lowest factor of safety as a red arc.

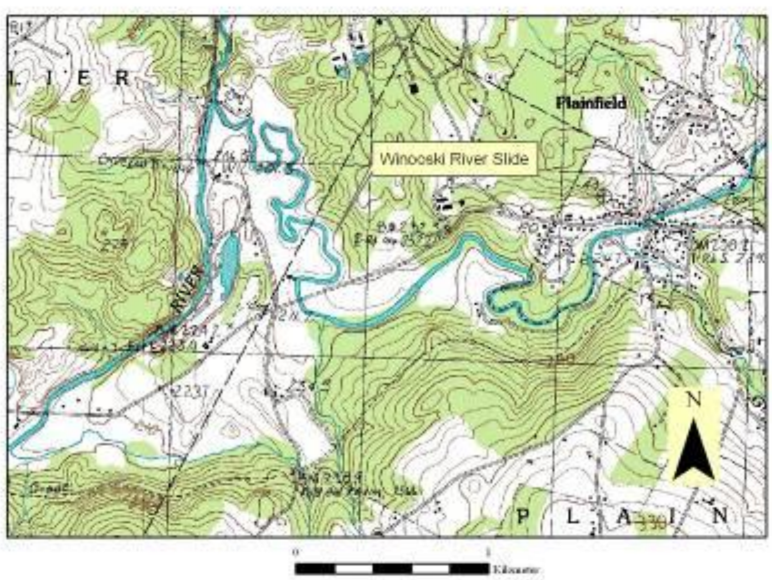


Figure 6.9. Location map for landslide in lacustrine deposits on the east bank of the Winooski River in Plainfield, Plainfield quadrangle.

Figure 6.10. Landslide in varved lacustrine deposits, Winooski River, Plainfield. Note steep main scarp in rear, towers of silty material above and behind Allen Clark of the Plainfield Conservation Commission, and disaggregated slide deposits at his feet and to his right. October 20, 2005.

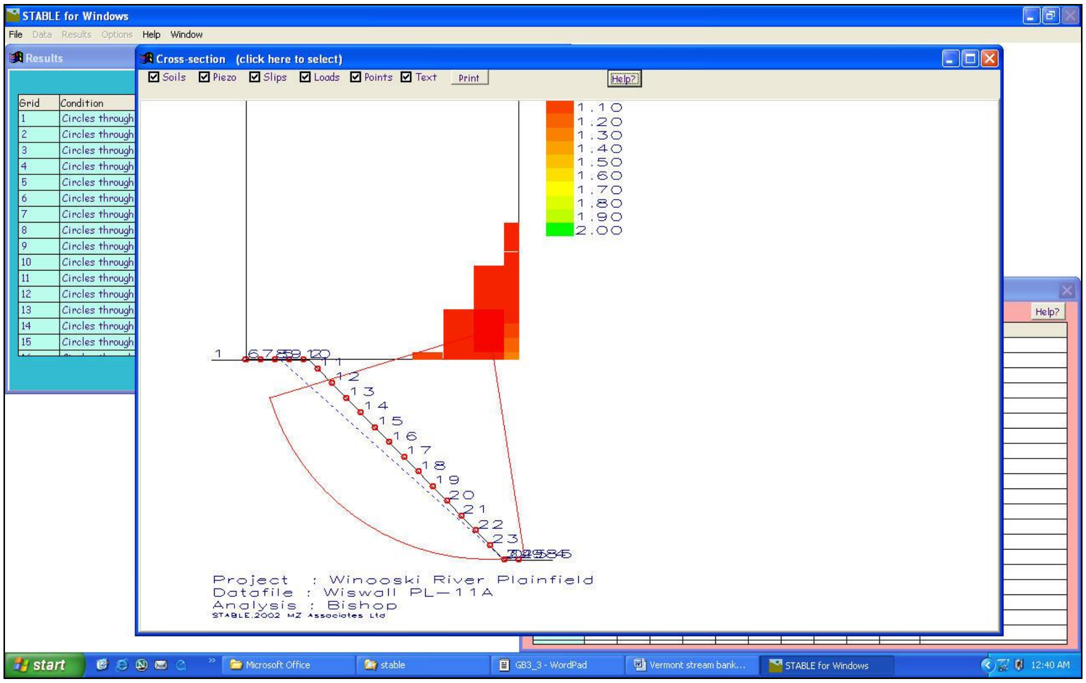


Figure 6.11. Image of output screen for STABLE analysis of Winooski River site in Plainfield. The program generated 20 slip circles using Bishop's method and plotted the circle with the lowest factor of safety as a red arc.

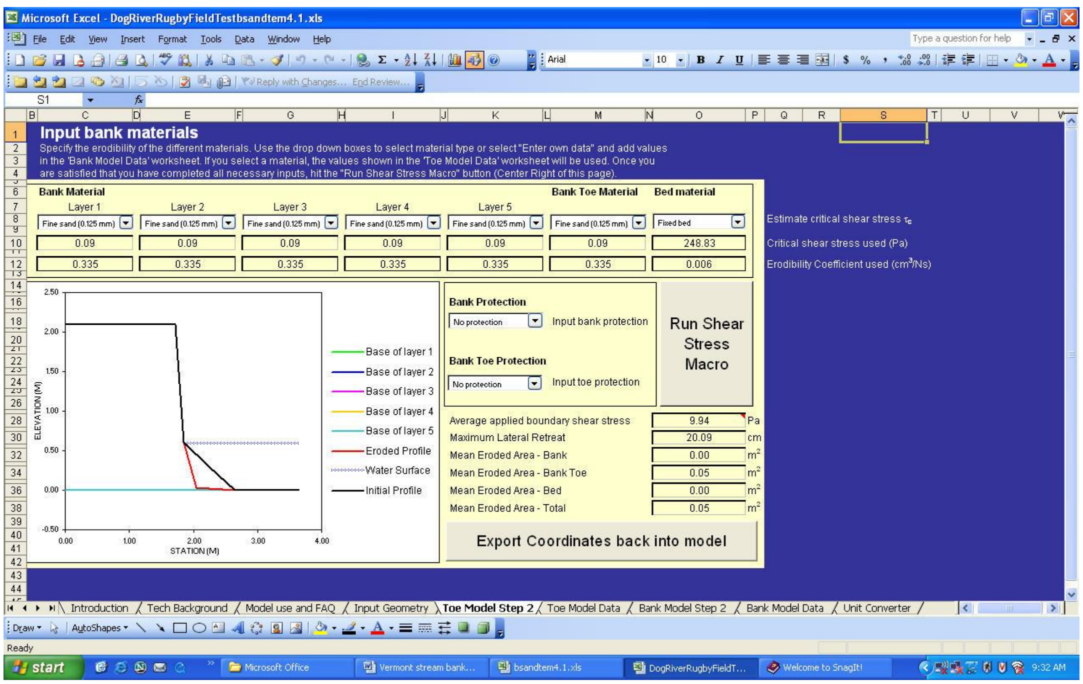
Wedge and Cantilever Failures

This section describes the modeling of two types of wedge failure (wedge with planar shear surface and wedge with a tension crack and planar shear surface) and cantilever failure with shear surface. In Vermont, these and similar failure types are commonly encountered in eroding banks underlain by modern alluvium or older stream terrace deposits. These types of failure are probably applicable to some other deposits as well, such as relatively low banks in ice-contact sands and gravels. In most cases in Vermont the materials are predominantly noncohesive. The wedge with planar shear surface is illustrated in Figure 3.9a and the cantilever is illustrated in Figure 3.9b. In Vermont these types of failures appear to be limited to relatively low banks, probably less than 10 to 20 feet in height. Taller banks generally fail by one of the mechanisms described in earlier sections. All three of these can be modeled using the Bank Stability and Toe Erosion Model developed by the Agricultural Research Service of the U.S. Department of Agriculture (Simon and others, 2001). The version utilized here is Static Version 4.1.

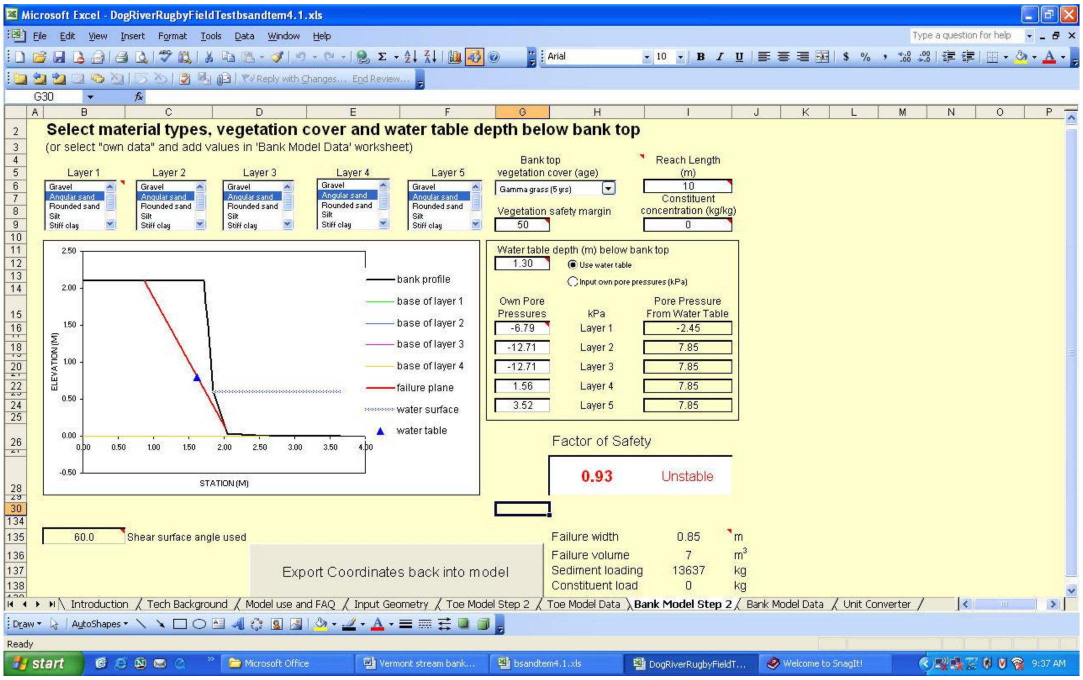
The ARS Bank Stability and Toe Erosion Model was developed to permit iterative calculations of bank failure and toe erosion. Inputs include bank geometry, material properties for both the bank and the toe deposit, and flow parameters. Suggested values for material properties are available as drop-down menus or custom data can be input. The water table can be entered and negative pore pressure in unsaturated materials can be accounted for. The model has particular value as a means to evaluate the effect of proposed treatments on bank stability. Once the general information for a site is entered, it is relatively simple to make changes to ban characteristics and evaluate the effects of high stream flow and/or high pore water pressure.

An example of an ARS Bank Stability and Toe Erosion Model is shown in Figure 6.12, where screenshots from the program show three increments of toe erosion and bank failure for a site on an eroding bank on the Dog River in Northfield, Washington County. The site is located on an outside bend of the west side of the river, adjacent to the Norwich University Rugby Field. In the first image, the bank has already experience past episodes of erosion and slope failure and a toe deposit is to be eroded away (outlined in red at base). The fluvial erosion has occurred in the next image and the failure plane has formed (red sloping line). In the final image, the toe has been undercut and the next stability analysis would show renewed bank instability.

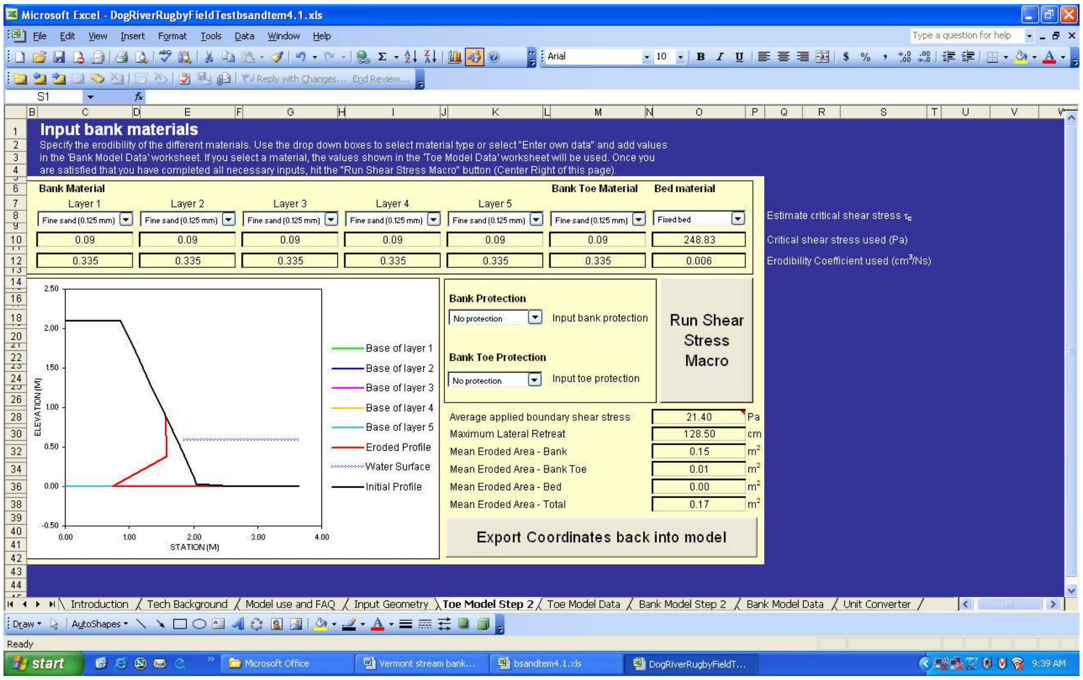
Although this is a relatively crude example, with the geotechnical data and flow data being estimated rather than measured, it serves to illustrate the potential of the ARS model.



6.12a.



6.12b.



6.12c.

Figure 6.12. ARS Bank Stability and Toe Erosion Model run for an eroding bank on the Dog River in Northfield. a. This screen shot represents initial conditions on an eroded bank as toe erosion is about to occur. b. Erosion has occurred and the bank is now subject to a new increment of toe erosion. c. The next increment of toe erosion has occurred, preparing the bank for another episode of failure.

Current limitations of the ARS model include lack of Vermont-specific data for both materials and vegetation. Specifically, data would be needed on the unit weight (bulk density), shear strength parameters, erodibility coefficients, and critical shear stress of varved lacustrine sediments and both dense silt-matrix till and loose sand-matrix till, as well as data on root cohesion and surcharge (weight of vegetation). Some of the unit weight and shear strength data in Appendix C of this report will be helpful in this regard. Although the types of failures used in the ARS model are probably not likely to occur in banks composed entirely of varved silt and clay or dense till, these will commonly be encountered in the lower portions of such bank failures. If this model is to be of use in Vermont it is very important to develop root cohesion and surcharge data for plant species common to Vermont riparian zones.

7. Summary

The scale of Vermont bank failures should not be underestimated. Although many are quite small, there are well-documented cases of landslides on stream banks involving many thousands of cubic meters of material. The larger ones have the potential to overwhelm stream channels and cause massive damage.

Bank failure is the result of a combination of three sets of processes: Weathering, fluvial erosion, and slope failure. Weathering processes such as the leaching of soluble minerals and freeze-thaw action soften the materials. Fluvial erosion due to the shearing stress of flowing water against the bank removes particles or aggregates of particles and sweeps them downstream. A variety of slope failure types are found along Vermont Rivers, with the most common types being summarized below.

Banks composed of glacio-lacustrine or glacio-fluvial sand, silt, silty clay, or clay commonly fail as rotational slides that may cut deep into the bank. The result is often a complex rotational slide and flow, with the surfaces of blocks near the back of the slide tilted backward away from the river and at the base a more or less disaggregated mass of slide material that has flowed out toward the river.

In marked contrast to the rotational failures in stratified deposits, failures in dense till are often surprisingly shallow. The difference appears to lie in the lack of extensive shear surfaces in most tills. Although the material is very inhomogeneous, containing an incredible range of grain sizes from clay-size particles to boulders the size of a house, there are commonly no extensive surfaces that can serve as easy shear surfaces. The result is that a steep, eroding bank of fresh, dense till does not fail as a whole. Instead it tends to spall off shallow slabs a fraction of a meter thick. If there is weathered till above or to the sides, such material will fail as some combination of translational slides and flows, depending on how well the surface is tied together by vegetation. Translational slides commonly appear to require near-total saturation with ground water before failure is initiated.

Low banks less than 10 to 15 feet high that are composed predominantly of non-cohesive modern alluvium or older stream terrace deposits tend to fail as a wedge with a planar shear surface dipping down toward the stream, a wedge with a tension crack and planar shear surface dipping down toward the stream, or a cantilever (overhang) with a vertical shear surface at the back. Some of the wedges with tension cracks have a very deep tension crack and short shear surface and may end up toppling outward as they begin to slide. It is also common for these low banks to be undercut by fluvial shear and to have a sort of "beam" cantilever failure in which the outer edge of the cantilever rotates down while pivoting at the tension crack at the back. Such failures may leave vegetated blocks on the bank, providing some temporary "armoring" until they are broken up or swept away.

Bank erosion rates span the entire range from very low (essentially zero measured over some decades) to the extremely rapid erosion of large hillsides in a single night of flooding.

Water is an important factor to consider in all types of slope failures. The influence can range from promoting weathering, to increasing the bulk density of the mass, to decreasing the effective shear stress within the soil or on a discontinuity. Stream erosion acting at the base of a slope can oversteepen the slope, leading directly to failure. this is probably the single most important factor in destabilizing stream banks. Water that fills the pores of the soil can be under pressure and can reduce the effective stress sufficiently for a once-stable bank to collapse. Conversely, negative pore pressure in unsaturated soils may provide a certain amount of cohesion, thus increasing the strength of the soil (as long as it remains unsaturated). In the harsh climate of Vermont the freezing of water in the soil or in discontinuities also needs to be considered: The force of crystallization may directly open fractures or ice blanketing the outer part of a face may result in higher pore pressure behind the face. Needle ice formed on a soil surface may soften the material once it thaws, leaving it more susceptible to stream erosion. In summary, there is probably no more important a set of factors to be considered in slope stability studies than those related to surface and ground water.

An eroding section of stream bank contains many clues that can aid in understanding the processes that are dismantling it. The observer needs to try to answer the following questions: Which parts of the banks are most freshly eroded and which have been stable for a while? Is the bank breaking apart in large blocks or being swept away piece by piece? What sizes of particles are being swept away and what is being left behind? Which parts are currently saturated with water and which are dry? The close observation of all of these bank characteristics is critical for the successful modeling of bank stability.

Because of the variety of surficial geologic materials underlying Vermont stream banks, no one geotechnical model is appropriate for all sites. All of the models used in this study are "limit equilibrium" models that calculate the balance between driving forces and resisting forces in order to generate a factor of safety. If this factor is significantly greater than 1.0, the slope is stable. If significantly below 1.0, it is unstable. A computerized infinite slope model is used for the shallow, translational slides, both chart-based and computerized slip circle methods can be used for deeper rotational slides in glacio-lacustrine and glacio-fluvial deposits, and computerized wedge and cantilever failure analysis models can be used for low banks composed predominantly of non-cohesive materials.

The infinite slope program, LISA 2.0, can be run in either a probabilistic mode or a deterministic mode. In the probabilistic mode the program generates many solutions (typically 1000) while varying the slope, material, and ground water properties through ranges specified by the user. The frequency distribution of results gives an estimate of the probability of failure. In the deterministic mode the program can be used to explore how the stability of the slope changes with variations in each of the input parameters, thus helping the user to identify which input parameters are the most critical.

For the deep, rotational slides, stability was calculated using the stability charts of Duncan (1996, after Janbu, 1968) and Michalowski (2002) and Bishop's modified method of slices using the computer program STABLE. The charts are fairly easy to use and give results that compare well with the more sophisticated Bishop's modified method. For sites with simple geometry and stratigraphy, the chart methods will probably be sufficient for calculating rough factors of safety.

The Bank Stability and Toe Erosion Model of the U.S.D.A. Agricultural Research Service (Simon and others, 2001) is a good first step for calculating both stability and erosion on low banks composed predominantly of non-cohesive materials. The program can incorporate three types of slope failure: Wedge failure with a planar shear surface sloping down toward the stream, wedge failure with a vertical tension crack at the back and a planar shear surface sloping down toward the stream, and cantilever (overhanging) failure with a vertical shear surface at the back. Thee model cannot take into account toppling or beam failure of cantilevers. This model does, however allow for modeling of erosion of the toe and bank by fluvial shear. The stability and erosion parts of the model can be run repeatedly to simulate a sequence of fluvial erosion and slope failure events.

The models described above, while imperfect, can provide important insights into current processes operating at the site and can be used to assess the outcome of proposed bank and stream restoration efforts.

8. Further Research

In order to be able to make quantitative predictions of bank stability and erodibility, there is a great need for studies which integrate fluvial geomorphology with bank stability analysis. Such studies would need to consider soil shear strength, soil pore pressure and/or pore water suction, soil erodibility coefficients and critical shear stress for the surficial materials, and near-bank hydraulic shear stress due to stream flow. With this detailed information is should be possible to creat more sophisticated bank erosion models similar to those of Rinaldi and others (2004) and Simon and others, 2003).

An important aid to modeling of the erosion of stream banks by hydraulic shear would be to obtain Vermont-specific values for the critical shear stress and erodibility variables in the excess shear stress equation described above. In particular, values are needed for the following types of cohesive deposits in both weathered and non-weathered states:

Dense till of several varieties;
Varved lacustrine silts and silty clays;
Waterlain tills (diamicts deposited in glacial lakes from the raining out of debris from icebergs and floating glacial ice tongues)
Glacio-marine silt and clay deposits of the Champlain Valley.

These could be obtained through field measurements with a device such as the submerged jet tester described by Hanson (2001) and Hanson and Simon (2001).

In order to model actual amounts of material being eroded off of banks that fail by shallow translational landslides it may be worthwhile to combine the infinite slope model with a hydraulic shear component such as the one used in the ARS Bank Stability and Toe Erosion Model. As the infinite slope equation does not contain a slope length parameter (it is assumed infinite) it will be necessary to decide whether or not the whole slope length is involved in the failure. If it is, then the volume per unit width of landslide can be obtained by multiplying the slope length by the depth of the failure surface. The resulting volume of landslide material would then be fed into the hydraulic shear component of the model.

Further research is needed into jointing in glacio-lacustrine deposits and in till to determine to what extent these joints are near-surface structures that form in response to stress relief and/or dessication following bank erosion and to what extent they are of tectonic or glacio-tectonic origin and may extend deep into the deposits. An understanding of jointing could be critical to modeling pore water conditions and shear strength in the glacio-lacustrine deposits and till.

If the stability and erosion of common Vermont stream banks can be modeled successfully, stream bank protection and restoration treatments can be designed to better resist erosive forces and contribute to overall slope stability.

9. References

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Appendix A. Surficial Geologic Data Sheet
Vermont Geological Survey

Location______________________________ Site No. ______________________________
Observer______________________________ Date_________________________________
Organization___________________________ Town_________________________________
USGS Map_____________________________ UTM(NAD27) _________mN,________mE

Stratigraphic Log. Depth from top of section and thickness of each unit are measured in feet and tenths. Field determination of Unified Soil Classification System Group Symbols is described in Appendix A. Descriptions for each unit should include density or consistency, color of matrix (and other features such as mottles), textural description (using the grain size scale of Wentworth, 1922), sorting, moisture, cohesiveness, plasticity, sedimentary structures, weathering, reaction to HCl, morphology and lithology of clasts, clast fabric, root quantity and size, nature of lower contact, and interpretation. Grain size descriptions should be made in millimeters.

Elevation at top/base of section _______ feet/meters

Depth	Thick-ness	USCS Group	Description

Soils
NRCS Soil Series___________________________
NRCS Parent material______________________

Bedrock
Is bedrock present? Yes / No
If yes, where?

Landscape position
Slope _________ Aspect________
Landform (describe)

Erosion at Site
None / Sheet / Rill / Gully / Eroding stream bank

Slope Failure
Type of slope failure: None / Rock creep / Soil creep / Landslide

Photos

Interpretation

Columnar section or cross section. Specify scale and orientation.

Appendix B. Slope Stability Data Sheet
Vermont Geological Survey

Location ______________________________ Site No. _________________________
Observer______________________________ Date______________________________
Organization___________________________ Town______________________________
USGS Map_____________________________ UTM(NAD27) _______mN,_______mE

Note: This form is intended primarily for landslides involving debris and earth and for gullying. See the text for references to rock slope stability procedures. The Surficial Geologic Data Sheet should be filled out for each slope stability investigation site.

Slope Failure

Type of slope failure: None / Rock creep / Soil creep / Landslide

Landslide Type: Fall / Topple / Simple rotational slump / Complex rotational slump-flow / Slide / Flow / Other (describe)

____
____

Slide material: Rock / Debris / Earth
Activity: Active / Inactive / Relict
Initial and subsequent rate of movement:
____

Overall slope failure dimensions
Total Length______ Elev. Crown______
Elev. Tip______ Height______
Overall angle______

Surface of rupture dimensions
Length______ Width______
Depth______

Displaced mass dimensions
Length ________ Width ________
Depth ________

Description of main scarp, if present
____
____

Description of crown cracks, if present
____
____

Retrogression prominent? Yes / No / Unsure
Condition of toe: Intact / Partly eroded / Totally eroded

Gullying

Gully present? Yes / No
Activity: Active / Inactive / Relict

Length ________ Top width_______
Depth ________ Overall gradient_____
Left slope _____ Right slope_____

General questions

Springs? Yes / No
Seeps ? Yes / No
Piping? Yes / No
If yes to any, describe horizon water is coming from. Describe extent of piping:
____
____
____
____

Has surface or ground water been diverted onto the slope or into the gully? Yes / No Describe:
____
____
____
____
____

Are headcuts visible in stream or in gully? Yes / No Describe:
____
____
____
____
____

Has damage resulted? Yes / No Describe type of damage to structures, costs, injuries, or deaths:

Describe history of movement or gullying, including comments on weather at failure

Stream Geomorphic Assessment results:
Channel Evolution Model stage ______
RGA Condition Rating______
Stream Type Departure? Yes / No
Channel Adjustment processes

Has a monitoring plan been prepared? Yes / No
Attach relevant documents
Photos: Attach separate sheet if needed

Interpretation

Cause of slope failure or gullying.

Comments

Profile of site. Specify scale and orientation.

Appendix C: Soil Mechanics

In order to evaluate the stability of a stream bank it is necessary to have information on the topography of the slope, the material properties of the soil, and the distribution of pore water. The following sections provide background information on soil shear strength parameters and on pore water conditions.

Data on the topography of a stream bank is relatively simple to collect, but it is important to evaluate the relevant topographic situation. In cases where the bank has not yet been eroded, the existing topography is used. In cases where a stream is actively eroding the toe, and sliding has already occurred, it is likely that the critical stability situation arises when the next episode of erosion occurs and the toe is swept away. In all of the stability analyses described in this report the assumption is being made that the stability can be analyzed in the plane of a cross section or profile running up the slope--the assumption of plane strain.

Material properties include moisture content, specific gravity of the particles, the bulk density of the soil (moist or dry), porosity, void ratio, degree of saturation, and shear strength (cohesion and angle of internal friction). Descriptions of these parameters and the methods of measurement are given in Hall and others (1994) and in standard soil mechanics texts such as Bowles (1984) and Lambe and Whitman (1969). Although not all of these are necessary in every case, it will almost always be necessary to have information on bulk density and shear strength.

Although site-specific measurements are the best source of information, it is often necessary to estimate the properties based on charts and tables for similar materials. For this, the USCS classification described in Chapter 5 is extremely useful. Using the USCS classification, estimates of bulk density (unit weight), cohesion, and angle of internal friction can be estimated using the compilation of data in Appendix D of this report, Chapter 5 of Hammond and others (1992) and Rose (1994).

Other factors such as pore pressure conditions, root cohesion, and tree surcharge (stress applied due to the weight of trees on the slope) are not material properties as such, but also must be considered. These are discussed further below.

Shear Strength of Soils

A very clear introduction to the measurement and interpretation of shear strength data is Renteria (1994), on which the following overview is based. The stress at a point within a body of soil can be described in terms of three principal stresses, all mutually at right angles to each other. The greatest principal stress is σ^1, the intermediate principal stress is σ^2, and the least principal stress is σ^3. The Mohr-Coulomb failure criterion described below relates the normal stress at a point on a potential failure surface to the shear stress required for failure.

Mohr-Coulomb Failure Criterion
The Mohr-Coulomb Failure Criterion for soils states that the shear strength at failure(τ) is

$$\tau = C + \sigma \tan \Phi \qquad \text{Equation C.1}$$

where

C = cohesion
σ = normal stress on the failure surface
Φ = angle of internal friction.

This is illustrated in Figure C.1. The greater the normal stress applied across a surface, the greater the shear stress required for failure on that surface. This relationship is represented by the line on the graph with friction angle Φ. The cohesion C represents the stress required to initiate failure with zero normal stress.

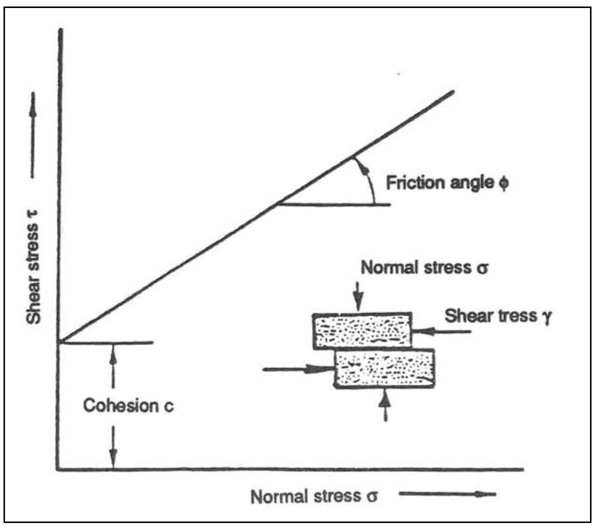


Figure C.1. Simplified Mohr-Coulomb failure envelope. The diagram shows how an increase in the normal stress (the stress component perpendicular to the failure surface) results in an increase in the shear stress that was required to lead to activation of the failure surface. From Denning (1994, Figure 4A.9).

The failure envelope for a soil can be developed by experimentally applying a known stress to bring a sample to failure while measuring the resulting strain on the sample. An example of one test is shown in Figure C.2. To construct a failure envelope, tests are generally conducted at three or more confining pressures and the results plotted on a Mohr diagram such as those shown in Figure C.3.

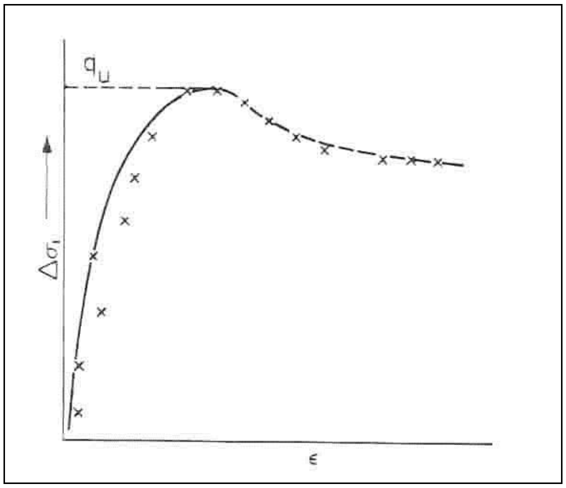


Figure C.2. Typical stress-strain curve resulting from an unconfined compressive strength test. Strain is plotted on the x axis and increasing greatest principal stress on the y axis. Stress increased rapidly with each increment of applied strain until a clear peak in strength was reached at q_u, after which the stress value resulting from continued strain dropped off to a fairly steady value (the residual strength). From Bowles (1984, Figure 13-3).

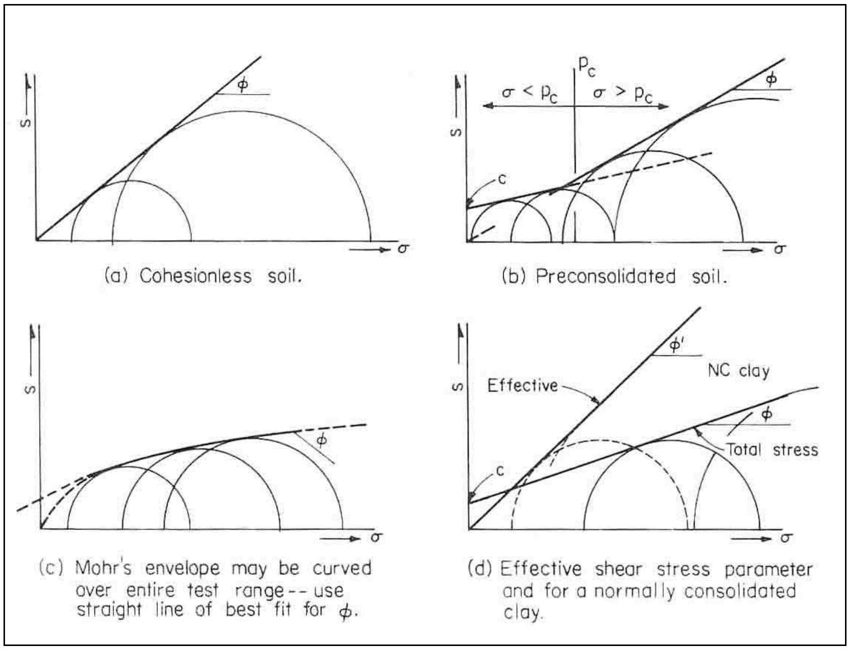


Figure C.3. Examples of schematic Mohr diagrams for soils. a) The result of a drained triaxial or direct shear test of a truly cohesionless soil (loose sand, etc.) b) A soil with an anomalously high shear strength at low normal stress values, due to some previous consolidation or cementation. The material has a higher shear strength than the observed overburden would indicate. At higher normal stresses, the material reverts to a failure envelope that . This is the standard interpretation of such concave-upward failure envelopes. c) A concave upward failure envelope. This is quite common and it is therefore important to determine the failure envelope for the range of anticipated normal stresses at the site in question. d) A comparison of the failure envelope produced from a typical undrained triaxial test in fine-grained soils (total stress) with that produced after subtracting the pore pressure component (effective stress). As described in the text, the effective stress values are what will normally be required for accurate analysis of slope stability. From Bowles, 1984, Figure 13-3.

The derivation and construction of Mohr diagrams are covered in soil mechanics texts such as Bowles (1984) and will not be described here. However, the diagrams are very useful for portraying critical concepts needed to understand bank stability, so certain key features will be pointed out. Figure 3.11a represents a simple case--a soil with zero cohesion. This means that a failure envelope would pass through the origin of the graph. Two tests were conducted to determine the strength at failure and each was portrayed by plotting the least principal stress applied to the sample (σ^3) and the greatest principal stress at the time of failure (σ^1). The difference between each pair of least and greatest principal stresses is used to define the diameter of a circle, which is plotted on the diagram. A point on this circle is tangent to the failure surface. Since in this simple case the failure surface also passes through the origin, the failure surface, if a straight line, can be drawn through the origin and tangent to the two circles. This line makes an angle Φ with the horizontal (normal stress) axis. In such a simple case the strength parameters can be described by the cohesion value (C = 0 stress units) and the angle of internal friction Φ (in degrees).

Figure C.3b shows a case where the failure envelope steepens with increasing normal stress. This sort of failure envelope is characteristic of soils with a high overconsolidation ratio. The standard interpretation is that such a material has been subjected to greater pressure than the existing overburden at the site can apply. The pressure could have resulted from a variety of sources, such as material that has been eroded away or a glacier that once overran it. Alternatively, some sort of cementation could have occurred. At the higher normal stresses, the envelope behaves in a more normal fashion, indicating that whatever strength the material possessed at lower stresses has now been exceeded.

Real soils will rarely have perfectly strain failure envelopes. More commonly they are curved as in Figure C.3c. Because of this possible curvature it is important to use data that was obtained at applicable confining pressures. Excessive confining pressures may result in unrealistically high cohesion intercepts and at least slightly low angles of internal friction.

Effective Stress

The effects of excess pore pressure on a failure envelope are illustrated in Figure C.3d, where two failure envelopes are shown for the same test sample. The results are typical of those of fine-grained soils that are subjected to triaxial testing. The envelope labeled "total stress" is the result of a direct plotting of the stress values as described for Figure C.3a. However, in this case, the sample is saturated and even if it was allowed to drain, the test can hardly be conducted slow enough to allow excess pore pressures to dissipate sufficiently. This total stress version of the failure envelope could be a realistic representation of the state of stress in a stream bank if saturated material was exposed due to rapid bank erosion, but in general, the result will yield too high a cohesion value and too low an angle of internal friction. The answer is normally to take the pore pressure into account by measuring the internal pore pressure in the sample during testing (not a simple matter) and then applying a correction as described in the next paragraph. The result is the failure surface produced using effective stress values. Note that in this example the cohesion intercept has been reduced to zero. Effective stress values are normally the ones to use in bank stability analyses.

The recognition of the influence of pore pressure on soil strength and slope stability is due largely to the work of Karl Terzaghi. In a series of books and papers published starting in the 1920's, Terzaghi developed the concept of effective stress and demonstrated how excess pore presssure could lead to slope instability (Terzaghi and Peck, 1948). The effective stress (σ') is defined as

$$\sigma' = \sigma - u \qquad \text{Equation C.2}$$

where

σ = total stress
u = pore pressure

For positive pore water pressures this has the effect of shifting each Mohr circle to the left as is the case in Figure C.3d. The resulting steeper angle of internal friction is labeled as Φ' and the effective cohesion value is C'. The shear strength then becomes

$$\tau = C' + (\sigma - u) \tan \Phi' \quad \text{Equation C.2}$$

This is the Mohr-Coulomb criterion for effective stresses. Throughout the bank stability modeling procedures that follow it will be important to carefully distinguish total stress parameters from effective stress parameters.

Terzaghi summarized his work on the effects of pore pressure, as well as the existing knowledge on landslides in his "Mechanism of Landslides" (Terzaghi, 1950). One of the examples used in that paper is Newland's Hudson slide described in Chapter 2 (Newland, 1916). Terzaghi theorized that excess precipitation caused an increase in pore pressure in the gravel under the varved clay. This, combined with heavy artificial loading from gravel piles, initiated the slope failure that blocked Claverack Creek. The critical role that excess pore pressure plays in soil strength has been demonstrated repeatedly both experimentally and in the field and the quest for actual measurements of pore pressure at the failure surface has become standard practice (Goudie, 1990; Bowles, 1984; Hall and others, 1994; Lambe and Whitman, 1969; among others).

Apparent Cohesion in Coarse-grained Soils

It is common to see eroded stream banks which, although they are composed of silt or fine sand, nevertheless stand at a steep angle for months at a time during dry weather. Although part of this short-term stability may be due to sheltering of the lower bank by vegetation and cohesion from roots, part of this short-term stability is often the result of an apparent cohesion that arises within partially saturated pores. This cohesion vanishes when the material become fully saturated, leading to renewed bank collapse. The source of this apparent cohesion is a negative pore water pressure (also called matric suction) arising from some combination of surface tension on water films and electrochemical bonding between particles (Fredlund and others, 1978; Rinaldi and others, 2004; Simon and others, 2001). Matric suction (ψ) is defined as

$$\psi = (u_a - u_w) \quad \text{Equation C.4}$$

where

u_a = atmospheric pressure

u_w = pore water pressure.

The apparent cohesion (C_a) is defined as

$$C_a = C' + \psi \tan \Phi^b \quad \text{Equation C.5}$$

where

C' = effective cohesion
ψ = matric suction
Φ^b = angle representing rate of increase of strength with increasing matric suction.

Thus, the apparent cohesion is a function of both the effective cohesion measured

The value for Φ^b varies with both the soil type generally varies between 10 and 20º, with a maximum value arising at saturation and being equal to the effective angle of internal friction for the particular soil (Simon and others, 2001). In the absence of site, specific data the Bank Stability and Toe Erosion Model of Simon and others (2001) that is described in Chapter 6 assumes a value of 5º for gravel and a value of 15 º for angular and rounded sand, silt, soft clay, and stiff clay. As discussed in chapter 6, the model does allow site specific data to be used.

The Mohr-Coulomb criterion for effective stress is then modified by the addition of an apparent cohesion term as below:

$$\tau = C' + (\sigma - u_a) \tan \Phi' + (u_a - u_w) \tan \Phi^b. \qquad \text{Equation C.6}$$

Peak Versus Residual Shear Strength

In analyzing slope failures on stream banks it is very important to understand whether the materials in the bank should be modeled using their peak strength values or residual strength values, which may be substantially below the peak values (Skempton, 1964, 1975; Goudie, 1990). Figure 3.10 shows a clear peak followed by a drop off to a substantially lower strength at higher strains. In some cases residual strengths can be determined by taking standard direct shear, unconfined compressive strength, or triaxial tests out to high strain values. In other cases the sample is remolded, either in by kneading or by continued shearing in a vane test. Some materials, particularly the clays classed as "sensitive" may have extremely low remolded shear strengths. If there has been no previous generation of a failure surface within a stream bank, then theoretically the peak strength values should be used to model the factor of safety. If, on the other hand, slope failure has already occurred and one or more failure surfaces or zones exist within the stream bank, then residual values may be more appropriate. Weathering must also be considered. The strength of a bank may be greatly reduced if weathering weakens the bank material. This may result in a complete change in the character of slope failure, such as from sliding to flowing. In such a case the shear strength of the weathered material could even be below that of the residual strength of the fresh soil.

Soil Strength Anisotropy and Preexisting Fractures

The materials underlying stream banks can at times be bewilderingly complex. The outer part of a bank may consist of earlier colluvium or landslide deposits that obscure the fresher materials beneath. The underlying material may consist of one single surficial deposit or several different ones. There may be a simple sequence of materials from bottom to top or they may be intimately mixed together. Once this has all been sorted out, it is important to realize that the strength properties of most of the common stream bank materials will vary depending on the direction of the stresses. A curving failure surface may therefore cut through horizontal strata nearly vertically in the back part of the slope, at about 45 degrees somewhere inside the slope, and then be horizontal somewhere around the lower or outer part of the slope (Figure 3.12). It is common for the shear strength of soils to be lowest for samples inclined at about 45 degrees to the bedding, with highest values for those perpendicular to bedding and intermediate values for those parallel to bedding (Aldinger, 1973). As most geotechnical tests are conducted on samples from vertical borings, most strength data is for the vertical orientation. The end result of this situation is that the available soil strength values may well be too high for the likely orientations of failure surfaces.

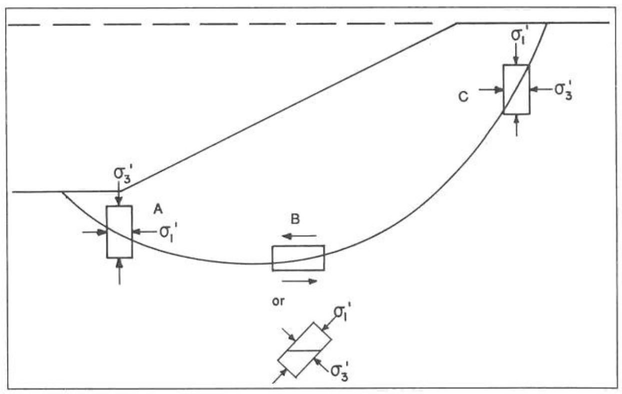


Figure C.4. Schematic view of the variation in orientation of principal stresses near a curved failure surface on a slope. Note that greatest principal stress is vertical at top, inclined near central section, and horizontal at toe. From Wu (1996, Figure 12-2).

Preexisting fractures within bank material may also dramatically reduce the strength of a bank and influence the form of sliding. Joints are fractures in rock or sediment which have not experienced movement parallel to their faces (although they may well have opened perpendicular to their faces). Depending on their orientation, joints may serve as either tension (release) surfaces or shear (sliding) surfaces. Where present, such fractures can exert a critical control on stream bank stability.

Fractures in till are described by Flint (1971, p. 159-160) and Brockman and Szabo, (2000), and detailed classifications of fracture-bounded peds (the structural elements of a soil) are given in standard soil science texts and manuals such as Birkeland (1999) and Schoeneberger and others (2002). Fractures in till deposits are of three main types: vertical fractures that decrease in frequency with depth, horizontal to subhorizontal fractures that also decrease in number with depth (fissility?), and fractures that may have steep to shallow dips, may occur in sets of relatively constant orientation and constant frequency, and may show displacement (Brockman and Szabo, 2000). The first are attributed to dessication, the second to some combination of shear during deposition of lodgement till, dessication, stress release, or freeze-thaw, and the third to glacio-tectonic shearing or lateral stress release (Brockman and Szabo, 2000). Roughly polygonal joints have been observed in dense till in central Vermont (Dunn and Springston, 2006).

Prominent sets of joints have also been noted in fine-grained glacio-lacustrine silts and silty clays in northwestern Vermont (Bierman and others, 1999) and the Connecticut River valley in Massachusetts (Emerson, 1898, p. 709-711). Some may have formed due to stress relief and desiccation associated with bank erosion. These may thus be a near surface feature and may not contribute greatly to weakening the deep parts of the bank. Emerson's observations indicated that in the lacustrine deposits "...the joints are found in limited areas having relation to recent erosions, or in bluffs produced by digging" (1898, p. 710). In other words, the joints formed in the vicinity of natural or artificial faces. He favored shrinkage arising from dessication as one cause but also suggested that slippage on underlying materials might also be involved (1898, p. 711). It remains possible that some joints may be of tectonic origin and thus may be widespread rather than localized near faces. If such joints exist, they could serve to dramatically weaken the deposits, both by reducing the bulk shear strength and by promoting infiltration of water.

Appendix D: Geotechnical Properties of Vermont Surficial Materials

Although detailed geotechnical analysis requires actual measurement of the physical and at times chemical properties of surficial materials, the typical ranges of values for the various classes of surficial materials can be of use in studies of stream banks and landslides.

Table E.1. Typical soil unit weight, friction angle and cohesion. Adapted from Table 4D.1 in Rose (1994). Note that unit weights are listed as typical values when saturated and when dry. This is generalized data and is probably not, by itself, suitable for analysis of specific slopes.

Cohesion type	Material	Unit Weight (saturated/dry)		Friction angle	Cohesion	
		pcf	kN/m^3	degrees	psf	kPa
Cohesionless	loose sand, uniform grain size	118/90	19/14	28-34[1]		
	dense sand, uniform grain size	130/109	21/17	32-40[1]		
	loose sand, mixed grain size	124/99	20/16	34-40[1]		
	dense sand, mixed grain size	135/116	21/18	38-46[1]		
	Gravel, uniform grain size	140/130	22/20	34-37[1]		
	Sand and gravel, mixed grain size	120/110	19/17	30-45[1]		
Cohesive	Very soft organic clay	90/40	14/6	12-16	200-600	10-30
	Soft, slightly organic clay	100/60	16/10	22-27	400-1000	20-50
	Soft glacial clay	110/76	17/12	27-32	600-1500	30-70
	Stiff glacial clay	130-105	20/17	30-32	1500-3000	70-150
	Glacial till, mixed grain size [2]	145/130	23/20	32-35	3000-5000	150-250

Notes:
[1] Due to curved stress envelopes, higher friction angles can occur in cohesionless materials at low confining stresses.
[2] This is apparently a dense, unweathered till. As discussed below, weathered tills in Vermont, even when they have been derived from quite dense parent tills, seem to have lower bulk densities and far lower cohesion values.

***Discuss the difference between index properties and engineering properties

***Discuss strong strength anisotropy in rhythmites using Fairfax deposit and also Hight and Leroueil (2003)

The most comprehensive study of engineering properties of a variety of deposits within Vermont is by Scanlon (1975). He undertook an analysis of the geotechnical properties of all of the major surficial units in the Vermont portion of the Brattleboro 15 minute quadrangle. These included USCS Group, percolation tests, an elementary bearing strength test, grain size analyses, liquid limit, plasticity index, pH, and x-ray diffraction analysis. Moisture contents, bulk densities, and specific gravity were not determined. Table A-1 summarizes much of this information.

Table D-1. Summary of geotechnical properties of surficial materials from the Vermont portion of the Brattleboro quadrangle. Adapted from Scanlon (1975).

Description	N	Gravel[a]	Sand[a]	Silt[a]	Clay[a]	Liquid Limit	Plasticity Index	USCS Group	pH
Ablation till	51	20.8	56.7	13.2	3.2	10 - 16	np[b] - 7	SM, SM-SC	4.5 - 6.8
Basal till	27	15.1	54.7	22.9	4.9	16 -26	np - 9	SM, SC, SM-SC	5.0 - 7.1
Fluvial sand	12	16.5	66.4	16.8	3.1	12 - 24	np - 8	SW, SM, SC, SW-SM	5.0 - 6.0
Kame terrace	14	21.9	52.9	20.6	3.8	13 -32	np - 9	GW-GM, SM, SM-SC, SW-SM, ML	4.9 - 6.3
Kame	2	33.5	62.0	8.5	1.5	13	np - 2	SM	5.2 - 5.6
Lake sand	8	7.8	83.0	8.4	1.0	11 - 17	np - 2	SW, SP, SM, SW-SM	5.2 - 6.5
Outwash	6	5.6	87.8	6.4	1.0		np	SW, SM, SW-SM	5.2 - 6.3
Lake gravel	6	50.3	46.7	2.7	0.6	10 - 11	np - 1	GW	5.1 - 6.5
Laminated silt/clay	2	-	4.5	65.0	29.5	38 - 40	14 - 18	ML	8.1 - 8.4[c]
Laminated silt	1	-	7.0	86.0	7.0	28	5	ML	

Notes: N = number of samples tested. Gravel/sand break at 4.76 mm and sand/silt break

at 0.074 mm. It is unclear which silt/clay break was used.
a. Mean value.
b. np = non-plastic
c. N = 3 (includes one laminated silt sample).

Scanlon's terminology for materials is retained in the table. Ablation till is probably some combination of subglacial melt-out till and/or flow till. Basal till is probably largely lodgement till. The lake gravels appear to be largely delta gravels (Scanlon, 1975, p. 29). The outwash deposits appear to be Stewart's (1975) valley train deposits. The laminated silt/clay and laminated silt are varved lacustrine deposits.

Grain size distribution in tills
The data on glacial till in the Brattleboro area confirms the general observation that tills in the region are generally poor in clay. In order to be able to compare Scanlon's data with other studies, the percentages were recalculated to eliminate gravel. Clay content in ablation till ranges from 1 to 11% with a mean of 4.4 % while in basal till it ranges from 1 to 19% and has a mean of 6.0%.

Cannon (1964) studied the mineralogy and grain size distribution of the sand-and-finer fraction in till in northern Vermont from the White River Junction area and Burlington northward with most of his 50 samples coming from the Vermont Piedmont. The ablation till contains less than 15% clay with 65 - 88% sand and 10 - 35% silt. The basal till contains less than 30% clay, with most samples containing less than 10% (Stewart and MacClintock, 1969, p. 26-27).

Shilts (1965) studied surficial deposits in the Green Mountains and Vermont Piedmont in northern Vermont. As part of his study he examined the grain size in 2 samples of ablation till and 10 samples of basal till. The ablation till contains less than 3% clay with 46 - 79% sand and 21 - 51% silt. The basal till contains 4 - 24% clay with 32 - 60% sand and 4 - 24% silt. The average clay content of the basal till is 12.9%.

Till samples from seven localities in Chittenden and Franklin Counties in northwestern Vermont contained showed a wider range in grain size, with the clay content in 31 samples ranging from less than 5% to 50% (Duggan, 1974). The samples had an average clay content of 18.7%.

**** Add in section on my grain size analyses: Grreat Brook, Mad River, etc.

**** Add in lacustrine silts at Waterbury Dam (USACOE)

****Add in lacustrine silts at Randolph Landfill (me)

****Add in fluvial deposits at Browns River (Slavin, 1977)

A comprehensive study has been made of the geotechnical properties of glacial Lake

Hitchcock deposits at the National Geotechnical Experimentation Site in Amherst, Massachusetts (DeGroot and Lutenegger, 2003). Tables A-2 and A-3 present some of the general and index properties. Note that the percentage of clay is substantially higher in the Amherst samples than in the lacustrine silt and clay samples from the Brattleboro quadrangle described in Table A-1.

An important feature described from the Amherst site is a weathered crust that has formed in the upper several meters of the deposit due to some combination of freeze/thaw, dessication, and other weathering processes (DeGroot and Lutenegger, 2003). This weathered zone is characterized by brown and gray/brown colors while the unweathered material is described as gray. The moisture content is substantially lower in the crust and the liquid limit exceeds the water content (Liquidity Index less than 1) while below about 3 meters the water content has increased such that even though the liquid limit has increased, the water content is now higher (yielding a Liquidity Index greater than 1).

Similar weathered crusts have been observed in fine-grained lacustrine deposits in Vermont and indicate that great care should be taken not to underestimate the depth to which alteration may occur, even in fine-grained materials.

Table D-2. Geotechnical properties of varved silt and silty clay at the National Geotechnical Experimentation Site in Amherst, Massachusetts. After DeGroot and Lutenegger (2003).

Depth (m)	Sand (%)	Silt (%)	Clay (%)	Specific gravity ($Mg.m^3$)	Bulk Density ($Mg.m^3$)	w (%)	LL (%)	PL (%)	PI (%)	LI	A
0.0 -1.4 (fill)	-	-	-	-	1.92	24	-	-	-		
1.4 - 3.1	2	62	36	2.88	1.89	37	39	28	11	0 - 1	0.31
3.1 - 6.1	1	47	52	2.88	1.73	52	51	31	20	1.1	0.38
6.1 - 24.0	0	45	55	2.88	1.66	62	51	30	21	1.5	0.38
Clay [1]	0	17	83	2.91	-	-	65	30	35	-	0.42
Silt [2]	1	72	27	2.87	-	-	38	28	10	-	0.37

Notes:
1. Properties listed after Clay are averages for the clay laminations.
2. Properties listed after Silt are averages for the silt laminations.

Table D-3. Typical chemical properties of varved silt and clay at the National Geotechnical Experimentation Site. After DeGroot and Lutenegger (2003).

Soil	pH	Organic matter (%)	Cation excange capacity (meq/100g)	Total carbonates (%)	Soluble salts (mmhos/cm)	Total surface area (m^2/g)
Bulk	8.4	0.5	8.2	3.3	0.2	53

Clay [1]	6.9	0.7	10.4	3.0	0.4	
Silt [2]	7.2	0.3	4.4	3.9	0.4	

1. Properties listed after Clay are averages for the clay laminations.
2. Properties listed after Silt are averages for the silt laminations.

The shear strength of the Lake Hitchcock materials at the site has been measured by a variety of techniques, some of which are summarized in Figure A-1. Below the weathered crust the undrained shear strength varies from roughly 20 to 50 kPa. Note, however, that all of these refer to shear strength with the principal stress axis oriented vertically (across the varves). As is the case with the Fairfax clay described below, the shear strength parallel to the varves is considerably lower than across them.

Figure D-1. Summary of undrained shear strength from field and laboratory tests (DeGroot and Lutenegger, 2003, Figure 17). The laboratory tests are undrained direct simple shear (DSS), anistropically consolidated triaxial compression (CAUC), and isotropically consolidated triaxial compression (CIUC). The field test are field vane (FVT) and dilatometer profiles (DMT).

Although the Index properties of the materials at Amherst are significantly different from the lacustrine materials in the Brattleboro quadrangle, this may be more a reflection of the limited number of samples at the Brattleboro sites. The samples appear to have come from shallow exposures and may not represent the unaltered material at greater depths. Other publications clearly show the presence of thick sections of silt/clay varves with similar physical appaearances to those at Amherst (See, for example Antevs, 1922, 1928; Ridge and Larsen, 1990; Ridge, 2003).

Table **. Fairfax varved clay from Aldinger (1973, Table 3.1)

Sample No.	Sand (%)	Silt (%)	Clay (%)	Specific gravity (Mg.m^3)	Bulk Density (Mg.m^3)	w (%)	LL (%)	PL (%)	PI (%)	LI	A
Light colored layer	0	72	28	2.80			33	27	7		0.25
Dark colored layer	0	16	84	2.84			72	30	42		0.5

Dark CH
Light ML
bulk density values were not determined in this study.
Probably leave this one out for now. Messy.

Sample No.	Sand (%)	Silt (%)	Clay (%)	Specific gravity (Mg.m^3)	Bulk Density (Mg.m^3)	w (%)	LL (%)	friction angle (total stress)	Cohesion (total stress)	Effective friction angle (degrees)	Effective cohesion (psi)
Fairfax vertical				2.82		31.4		14	10	33	0
Farifax inclined 45 degrees				2.82		32.8		14	2	20	3
Farifax horizontal				2.82		30.7		14	6	25.5	5

****Add in Circ Highway data

****Add in Weybridge slump (Solomon, 1975)

Sample No.	Sand (%)	Silt (%)	Clay (%)	Specific gravity (Mg.m^3)	Bulk Density (Mg.m^3)	w (%)	LL (%)	PL (%)	PI (%)	LI	A
1			34	2.76	2.0	37.6	42	23	19	.79	.56
2			60	"	"	34.6	65	24	41	.25	.68
3			74	"	"	40.6	54	27	27	.52	.36

Typical Geotechnical Properties for Vermont Soils

Sample No.	Sand (%)	Silt (%)	Clay (%)	Specific gravity (g-cm^3)	Bulk Density (PCF)	w (%)	LL (%)	PL (%)	PI (%)	LI	A
Stetson Brook, stony, unweathered	44.8	45.8	9.4	2.82	134	10.3	--	N.P.	--	--	--

Stetson Brook, stony, somewhat weathered				2.82	118	16.4	--	N.P.	--	--	--
Stetson Brook, non-stony, unweathered	11.3	74.3	14.4	--	--	--	--	--	--	--	--

Grain size was not measured in the somewhat weathered sample but it appears to be similar. Most of the bank appears to be composed of the more stony till.

A reasonable proxy for the dense tills so common throughout Vermont was studied by Radhakrishna and Klym (1974) at a site 60 miles east of Toronto, Ontario. The material is predominantly sandy silt with some clay, some pebble gravel, few cobbles, and very few boulders. It is nonplastic, with a water content of 6.5 to 8.5% and a bulk density of about 150 pcf. From the description the material appears to be some sort of a waterlain till deposited in a glacio-lacustrine setting. Field and laboratory tests indicated an approximate effective cohesion of 7 psi (48 kPa) and a phi effective of 35 degrees. These appear to be peak rather than residual values.

As a very crude approximation, weathered tills with a USC classification of SM-SC can be modelled as having a maximum dry unit weight of 110-130 pcf with a cohesion of about 300 psf (saturated) and phi effective of about 33 degrees. This is based on estimates for compacted fill in Renteria (1994, Table 4C.7).

Strength parameters for already failed landslide deposits:

Appendix E. Visual Identification of Soil Samples

From U.S. Army Corps of Engineers Field Manual EM 1110-1-1906, Appendix F-3.

Appendix F-3 (of EM 1110-1-1906)
Visual Identification of Soil Samples

E-1. Field Identification Techniques

Visual identification techniques reported herein generally yield results which are consistent with the Unified Soil Classification System (ASTM D 2487, 1993; ASTM D 2488, 1993; U.S. Army Engineer Waterways Experiment Station, 1960).[1] Because these techniques are primarily visual, subtle discrepancies may exist between the identifications obtained in the field and the classifications determined in the laboratory. However, the results are meaningful provided the inspector makes careful and consistent identifications. See Chapter 13 for details on handling and storage of samples and maintaining sampling records. The inspector's equipment required to conduct these tests is limited to a pocket knife, scale, magnifying glass, and a small container of diluted hydrochloric acid. Table E-1 can be used as a checklist for conducting a systematic visual identification of a soil sample; it is also useful for locating the appropriate table and/or figure which describe(s) a test procedure for visually identifying the soil.

E-2. Grain Size

The inspector must first determine whether the material is coarse grained or fine grained. To make this determination, spread a representative sample on a flat surface. Determine whether or not the predominant size fraction is discernible with the naked eye. Coarse-grained soils vary from particles in excess of 75 mm (3 in.) in diameter to particles just discernible with the unaided eye, such as table salt or sugar, whereas fine-grained soils are microscopic and submicroscopic. The predominant material of peat or muck is decaying vegetation matter. Table E-2 and Figure E-1 may aid in determining the grain size of the soil in question. If the predominant material is coarse grained, follow the procedures outlined in paragraph E-2*a*; if the soil is fine grained, follow the procedures outlined in paragraph E-2*b*.

a. Coarse-grained soils.

(1) *Coarse fraction*. Once the soil has been determined to be coarse grained, further examination is required to determine the grain size distribution, the grain shape, and the density of the in situ deposit (if applicable). The gradation of coarse-grained soils can be described as well graded, poorly graded, or gap graded.

Table E-3 and Figure E-2 can be used in selecting the appropriate descriptive terms. Soil particles can also be described according to a characteristic shape. Particle shape may vary from angular to rounded to flat or elongated. Appropriate descriptive terms are listed in Table E-4; particle shapes are illustrated in Figure E-3. The density of an in situ deposit of a coarse-grained soil is also valuable information. Results obtained by pushing a reinforcing rod into a surface deposit or from the Standard Penetration Test may indicate the density of an in situ deposit. Appropriate descriptive terms may be selected from Table E-5.

(2) *Fine fraction*. The plasticity characteristics of the fine fraction of a coarse-grained material also need to be determined. The tests for fine-grained soils (paragraph E-2*b*) which are described in

[1] References cited in this appendix are included in Appendix A.

Table E-6 and illustrated in Figures E-4 through E-10 can be used to characterize the fine fraction of the soil sample in question.

b. Fine-grained soils.

(1) *Coarse fraction*. The coarse-grained fraction, where applicable, should be described in terms of the size of the predominate grain size, i.e., sand or gravel. Paragraph E-2*a*, Table E-2, and Figure E-1 may aid in selecting appropriate descriptive terms.

(2) *Fine fraction.* Several tests may be useful in determining the plasticity characteristics of fine-grained soils or
fractions thereof; these tests include the dilatancy or reaction to shaking test, the dry strength test, and the toughness and plasticity tests. For each test, the fraction which passes the No. 40 U.S. Standard Sieve (0.42 mm) is used; this fraction corresponds to the fraction which is required for determination of Atterberg limits. For the purpose of the visual tests, however, screening is not important; the removal of coarse particles is adequate. Tests to determine the plasticity characteristics of the fine fraction are described in Table E-6, the dilatancy test is illustrated in Figures E-4 through E-7, the dry strength test is presented in Figure E-8, and toughness and plasticity tests are given in Figures E-9 and E-10, respectively.

c. Other tests. The dispersion (settlement in water) test and the bite test can be used to determine the presence of and relative amounts of sand, silt, and clay fractions (see Table E-6). Several other tests, such as the odor and the peat tests for determining the presence of organic matter, the acid test for determining the presence of a calcium carbonate cementing agent, and the slaking test for determining whether the "rocklike" material is shale, are listed in Table E-6. Strength descriptors of a clay sample are listed in Table E-7.

E-3. Soil Moisture and Color

Soil moisture and color are important indicators of soil conditions. For example, visible or free water from a soil sample can infer the proximity of a water table. The color of a moist soil sample tells much about the minerals and chemicals present in the soil, the drainage conditions, and the presence of organic matter. Soil color charts prepared for the U.S. Department of Agriculture (USDA) by the Munsell Color Company, New Windsor, NY 12553, are helpful for describing the color of soil samples. Soil moisture conditions or water contents can be described following the criteria presented in Table E-8. The importance of color for identifying and classifying moist fine- grained soils is shown in Table E-9.

E-4. Mass Structure and Mass Defects of Soil Formations

Mass structure and mass defects yield data about the geotechnical engineering behavior of a soil formation in question. For example, a varved clay would most likely have different engineering properties from an homogeneous deposit of one of the constituent soils. Likewise, slickensides indicate a clay deposit has been overconsolidated because of desiccation, surcharge loading, or both; an overconsolidated clay would have different engineering properties from a normally consolidated deposit of the same material. Descriptive terms for mass structure and mass defects of a soil formation are presented in Tables E-10 and E-11, respectively.

E-5. Description of Soils

As presented in the footnote in Table E-1, the description of a soil sample should contain appropriate terms to characterize the soil type and grain size, its moisture content and color, and mass structure and defects. Commonly used names and descriptions for selected soils are presented in Table E-12.

Table E-1
Order of Description for Soils

Criteria	Table No.	Figure No.
Soil types and particle sizes	E-2	E-1
Coarse-grained soils		
Description of gradation of coarse-grained soils	E-3	E-2
Description of grain shape of coarse-grained soils	E-4	E-3
Density of coarse-grained soils	E-5	
Fine-grained soils		
Field identification procedures for fine-grained soils	E-6	E-4/E-10
Strength or consistency of clays	E-7	
Moisture content	E-8	
Role of color for identification of moist fine-grained soils	E-9	
Terms for describing mass structure of soils	E-10	
Terms for describing mass defects in soil structure	E-11	
Commonly used descriptive soil names	E-12	

Example: Sand, fine, silty, tan, poorly-graded, dense, wet, subrounded, very friable with occasional clay lenses.

Table E-2
Soil Types and Particle Sizes

Principal Soil Type	Descriptive Term	Size	U.S. Standard Sieve	Familiar Example
Coarse-grained Soils	Cobble	76 mm or larger	Greater than 3 in.	Grapefruit or orange
	Coarse gravel	76 mm to 19 mm	3 in. to 3/4 in.	Walnut or grape
	Fine gravel	19 mm to 5 mm	3/4 in. to #4	Pea
	Coarse sand	5 mm to 2 mm	#4 to #10	Rock salt
	Medium sand	2 mm to 0.4 mm	#10 to #40	Openings of a window screen
	Fine sand	0.4 mm to 0.074 mm	#40 to #200	Table salt or sugar
Fine-grained Soils	Silt or clay	Microscopic and submicroscopic		
Organic	Peat or muck			Decaying vegetable matter

Note: Particles which are retained on the No. 200 U.S. Standard Sieve (0.074 mm) can just be discerned with the naked eye at a distance of about 25 cm (10 in.).

Table E-3
Description of Gradation of Coarse-Grained Soils

Descriptive Term	Meaning
Well graded	A good representation of all grain sizes is present
Uniformly or poorly graded	All grains are approximately the same size
Gap graded	Intermediate grain sizes are absent

Table E-4
Description of Grain Shape of Coarse-Grained Soils

Descriptive Term	Example
Angular	Irregular with sharp edges such as freshly broken rock
Subangular	Irregular with smooth edges
Subrounded	Irregular but smooth as a lump of molding clay
Rounded	Marble or egg shaped, very smooth
Flaky	Sheet of paper or flake of mica
Flat	Ratio of width to thickness greater than 3
Elongated	Ratio of length to width greater than 3

Table E-5
Density of Coarse-Grained Soils

Descriptive Term	Blows per Foot[1,2]	Field Test
Very loose	Less than 4	----------
Loose	4-10	Easily penetrated with a 13-mm- (1/2-in.-) diam reinforcing rod pushed by hand
Medium dense	10-30	Easily penetrated with a 13-mm- (1/2-in.-) diam reinforcing rod driven with a 2.3-kg (5-lb) hammer
Dense	30-50	Penetrated 0.3 m (1 ft) with a 13-mm- (1/2-in.-) diam reinforcing rod driven with a 2.3-kg (5-lb) hammer
Very dense	Greater than 50	Penetrated only a few centimeters with a 13-mm- (1/2-in.-) diam reinforcing rod driven with a 2.3-kg (5-lb) hammer

[1] From SPT (Appendix B, Table B-1).
[2] 1 ft = 0.3048 m.

Table E-6
Field Identification Procedures for Fine-Grained Soils[1]

Test	Test Procedures and Interpretation of Results
Dilatancy	Prepare a pat of moist soil with a volume equivalent to a 25-mm (1-in.) cube (Figure E-4). Add water, if necessary, to make the soil soft but not sticky, i.e., (reaction to the "sticky limit"). Place the pat of soil in the open palm of one hand and shake horizontally; strike vigorously against the other hand several times. If the reaction is positive, water appears on the surface of the pat; the consistency of the pat then becomes livery; and the surface of the pat becomes glossy (Figure E-5). Next, squeeze the sample between the fingers (Figure E-6). The water and gloss should disappear from the surface of the pat; the soil will stiffen and crack or crumble (Figure E-7). The rapidity of the appearance of water on the surface of the soil during shaking and its disappearance during squeezing help to identify the character of the fines in the soil. Very fine clean sands give the quickest and most distinct reaction, inorganic silts give a moderately quick reaction, and plastic clays have no reaction.
Dry strength	Mold a pat of soil to the consistency of putty. If the soil is too dry, add water; if it is too sticky, the specimen should be allowed to dry by evaporation. After the consistency of the pat is correct, allow the pat to dry (by oven, sun, or air). Test its strength by breaking and crumbling between the fingers (Figure E-8). The dry strength increases with increasing plasticity. High dry strength is characteristic of high plasticity clays. Silty sand and silts have only slight dry strengths, but can be distinguished by feel when powdered; fine sands feel gritty whereas silts feel smooth like flour. It should also be noted that shrinkage cracks may occur in high plasticity clays. Therefore, precautions should be taken to distinguish between a break which may occur along a shrinkage crack or a fresh break which is the true dry strength of the soil.
Toughness and plasticity	A specimen of soil which is about the size of a 25-mm (1-in.) cube should be molded to the consistency of putty; add water or allow to dry as necessary. At the proper moisture content, roll the soil by hand on a smooth surface or between the palms into a thread about 3-mm (1/8-in.) diam (Figure E-9). Fold the thread of soil and repeat the procedure a number of times. During this procedure, the water content of the soil is gradually reduced. As drying occurs, the soil begins to stiffen and finally loses its plasticity and crumbles at the plastic limit. After the thread has crumbled, the pieces should be lumped together and a kneading action should be applied until the lump crumbles. For higher clay contents, threads are stiffer and lumps are tougher at the plastic limit than for lower plasticity clays. A complementary test is the ribbon test. A roll of soil about 13-to 19-mm (1/2-to 3/4-in.) diam by 75 to 125 mm (3 to 5 in.) long should be prepared at a moisture content just below the "sticky limit". Flatten the roll of soil to thickness of 3 to 6 mm (1/8 to 1/4 in.) between the thumb and forefinger (Figure E-10). For high plasticity clays, a ribbon 20 to 25 cm (8 to 10 in.) long can be formed; shorter lengths correspond to lower plasticity clays whereas a ribbon cannot be formed when using non-plastic soils.
Dispersion test	Place a few hundred grams of soil in a jar containing water. Shake the jar containing the mixture of soil and water and then allow the soil to settle. The rate of settling can be used to judge the (settlement predominate soil type(s) whereas the thicknesses of the various soils can be used to judge the gradation of the soil. Sands settle in 30 to 60 seconds, silts settle in 30 to 60 minutes, and clays may in water) remain in suspension overnight. The interface between fine sands and silts occurs where individual grains can not be discerned with the unaided eye. The cloudiness of the water indicates the relative clay content.
Bite test	Place a pinch of soil between the teeth and grind lightly. Fine sands grate harshly between the teeth; silts have a gritty feeling but do not stick to the teeth; clays tend to stick to the teeth, but do not have a gritty feeling.
Odor	Organic soils have a musty odor which diminishes upon exposure to air. The odor can be revived by heating a moist sample or by exposing a fresh sample.
Peat	Peat has a fibrous texture and is characterized by partially decayed sticks, leaves, grass, and other vegetation. A distinct organic odor is characteristic of peat. Its color generally ranges from dull brown to black.
Shine	A moist, highly plastic clay will shine when rubbed with a fingernail or pocketknife blade: a lean clay will have a dull surface.
Acid test	The presence of calcium carbonate in a soil can be determined by adding a few drops of dilute (3:1 ratio of water to acid) hydrochloric acid to the soil. The relative amount of calcium chloride in the soil can be determined by the effervescence (fizzing reaction) which occurs. Degrees of reaction range from none to strong. For some very dry non-calcareous soils, the illusion of effervescence as the acid is absorbed by the soil can be eliminated by moistening the soil before the acid is applied.
Slaking test	Certain shales and other soft "rocklike" materials disintegrate upon drying or soaking. The test is performed by placing the soil in the sun or oven to dry completely. After the sample has been dried, it should then be soaked in water. The degree of slaking should be reported.

[1] These tests must be performed on the minus No. 40 sieve size (0.42 mm) particles, which is the division between medium and fine sand. For field classification purposes, screening is not intended; simply remove the coarse particles that interfere with the tests.

Table E-7
Strength or Consistency of Clays

Descriptive Term	Blows[1] per Foot[2]	Unconfined Compressive Strength kPa	(tsf)	Field Test
Very soft	< 2	<25	(< 0.25)	Core (height twice diameter) sags under its own weight while standing on end; squeezes between fingers when fist is closed
Soft	2-4	25-50	(0.25-0.5)	Easily molded by fingers
Medium	4-8	50-100	(0.5-1.0)	Molded by strong pressure of fingers
Firm	8-15	100-190	(1.0-2.0)	Imprinted very slightly by finger pressure
Very firm	15-30	190-380	(2.0-4.0)	Cannot be imprinted with finger pressure; can be penetrated with a pencil
Hard	> 30	> 380	(> 4.0)	Imprinted only slightly by pencil point

[1] From SPT (Appendix B, Table B-1).
[2] 1 ft = 0.3048 m.

Table E-8
Moisture Content

Condition	Estimated Water Content, percent	Example
Dry	0 - 10	Absence of moisture; well below optimum water content for fine-grained soils
Moist	10 - 30	Fine-grained - damp, near optimum water content Coarse-grained - no visible water
Wet	30 - 70	Fine-grained - well above optimum water content Coarse-grained - visible water
Water bearing	-------	Water drains freely, below water table

Table E-9
Role of Color for Identification of Moist Fine-Grained Soils

General:

Detect different soil strata
Detect soil type based upon experience in local area
Colors become lighter as water content decreases

Soil Type:

Inorganic soils have clean, bright colors: light gray, olive green, brown, red, yellow, or white
Organic soils have dark or drab shades: dark gray, dark brown, or almost black

Presence of Chemicals:

Iron oxides: red, yellow, or yellowish brown
Silica, calcium carbonate, or aluminum compounds: white or pinkish

Drainage Conditions:

Poor: grayish blue and gray or yellow mottled colors

Table E-10
Terms for Describing Mass Structure of Soils

Descriptive Term	Definition
Homogeneous	Uniform properties
Heterogeneous	Mixtures of soil types not in layers or lenses
Stratified	Alternate layers of different soils or colors
Laminated	Repeating alternate layers of different soils or colors 3 to 6 mm (1/8 to 1/4 in.) thick
Banded	Alternate layers in residual soils
Lensed	Inclusions of small pockets of different soils

Table E-11
Terms for Describing Mass Defects in Soil Structure

Descriptive Term	Definition
Slickensides	Fracture of failure planes (polished surfaces) seen in stiff clays
Root holes	Holes remaining after roots have decayed
Fissures	Cracks from shrinkage, frost, etc.; specimen breaks along a definite plane of fracture
Weathered, oxidized	Irregular discolorations
Concretions	Accumulations of carbonates or iron compounds
Blocky	Cohesive soil broken into small angular lumps which resist further breakdown

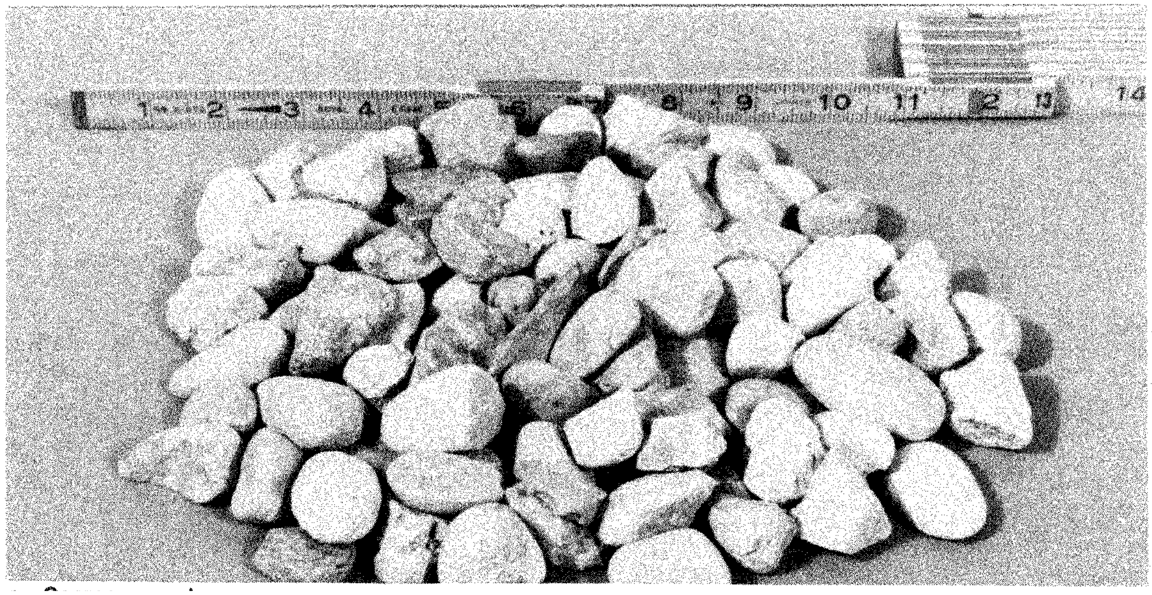
Table E-12
Commonly Used Descriptive Soil Names (Continued)

Common Name	Description
Adobe	Soils such as calcareous silts and sandy-silty clays which are found in semiarid regions of southwestern United States and North Africa.
Alluvium	Deposits of mud, silt, and other material commonly found on the flat lands along the lower courses of streams.
Argillaceous	Soils which abound in clays or clay-like materials.
Bentonite	A clay of high plasticity formed by the decomposition of volcanic ash.
Boulder clay (L)[1]	A name, used in Canada and England, for glacial till.
Buckshot (L)	Clays of southern and southwestern United States which crack into small, hard lumps of more or less uniform size upon drying.
Bull's liver (L)	The term name used in some sections of the United States to describe an inorganic silt of slight plasticity, which, when saturated, quakes like jelly from vibration or shock.
Calcareous	Soils which contain an appreciable amount of calcium carbonate, usually from limestone.
Caliche	The term which describes deposits of silt, clay, and sand cemented by calcium carbonate deposited by evaporation of groundwater; this material is found in France, North Africa, and southwestern United States.
Coquina (L)	Marine shells which are held together by a small amount of calcium carbonate to form a fairly hard rock.
Coral	Calcareous, rock-like material formed by secretions of corals and coralline algae.
Diatomaceous earth	A white or light gray, extremely porous, friable, siliceous material derived chiefly from diatom remains.
Dirty sand (L)	A slightly silty or clayey sand.
Disintegrated granite	Granular soil derived from decomposition and weathering of granite rock.
Fat clay (L)	Fine colloidal clay of high plasticity.
Fuller's earth	Highly plastic white to brown clays of sedimentary origin which are used commercially to absorb fats and dyes.
Gumbo (L)	Highly plastic silty and clayey soils which become impervious, sticky, and soapy or waxy when saturated.
Hardpan	A general term used to describe a hard, cemented soil layer which does not soften when wet.
Lateritic soils	Residual soils, usually red in color, which are found in tropical regions. In their natural state, these soils have a granular structure with low plasticity and exhibit good drainage characteristics; when remolded in the presence of water, they often become plastic and clayey.
Lean clay	Silty clays and clayey silts of low to medium plasticity.

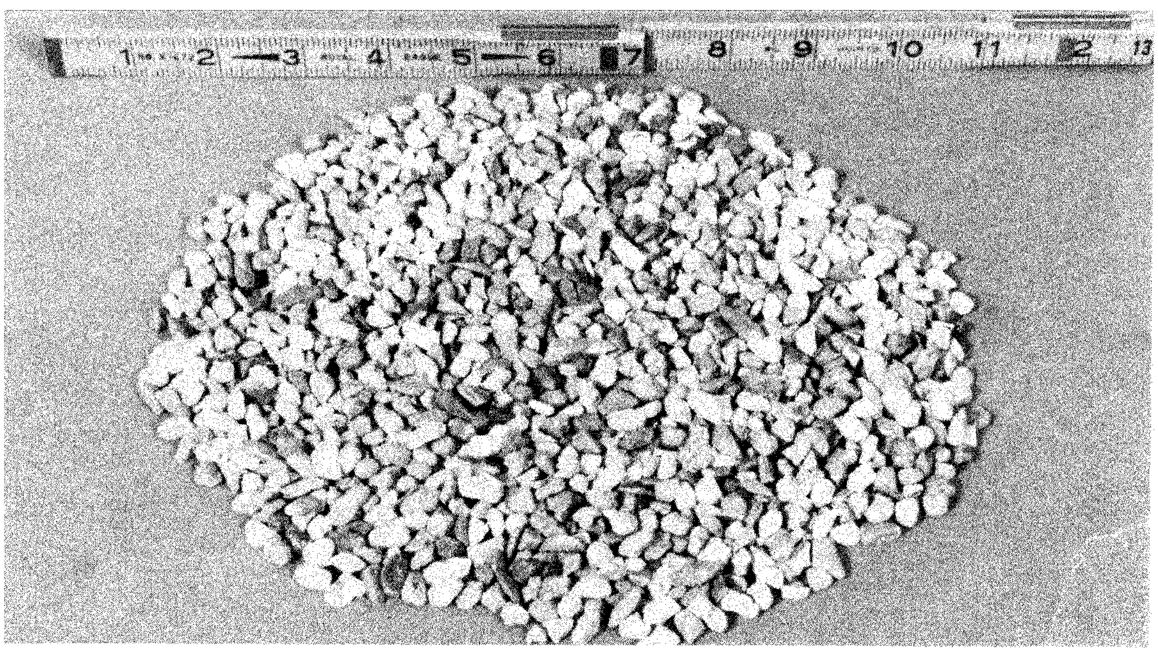
[1] (L) refers to a name which is used in local areas, only.

Table E-12 (Concluded)

Common Name	Description
Limerock (L)	A soft, friable, creamy white limestone found in the southeastern United States; it consists of marine remains which have disintegrated by weathering.
Loam	An agricultural term used to describe sandy-silty topsoils which contain a trace of clay, are easily worked, and are productive of plant life.
Loess	A silty soil of eolian origin characterized by a loose, porous structure, and a natural vertical slope; it covers extensive areas in North America, Europe, and Asia.
Marl	A soft, calcareous deposit mixed with clays, silts, and sands, and often contains shells or organic remains; it is common in the Gulf Coast area of the United States.
Micaceous soils	Soil which contains a sufficient amount of mica to give it distinctive appearance and characteristics.
Muck (mud)	Very soft, slimy silt which is found on lake or river bottoms.
Muskeg	Peat deposits found in northwestern Canada and Alaska.
Peat	Fibrous, partially decayed organic matter or a soil which contains a large proportion of such materials; it is extremely compressible and is found in many areas of the world.
Red dog (L)	The residue from burned coal dumps.
Rock flour	A low plasticity, sedimentary soil composed of silt-sized particles which may become quick at high moisture contents.
Shale	A thinly laminated rock-like material which has resulted from consolidation of clay under extreme pressure; some shales revert to clay on exposure to air and moisture.
Talus	A fan-shaped accumulation of fragments of rock that have fallen near the base of a cliff or steep mountainside as a result of weathering.
Topsoil	The top few inches of soil which contains considerable organic matter and is productive of plant life.
Tufa	A loose, porous deposit of calcium carbonate which usually contains organic remains.
Tuff	Stratified, compacted deposits of fine materials, such as cemented dust and cinders, ejected from volcanoes. Tuffs are prevalent in the Mediterranean area.
Varved clay	A sedimentary deposit which consists of alternate thin (less than 13 mm) layers of silt and clay.
Volcanic ash	Uncemented volcanic debris which consists of particles less than 3 mm diameter; upon weathering, a clay of high compressibility is formed.



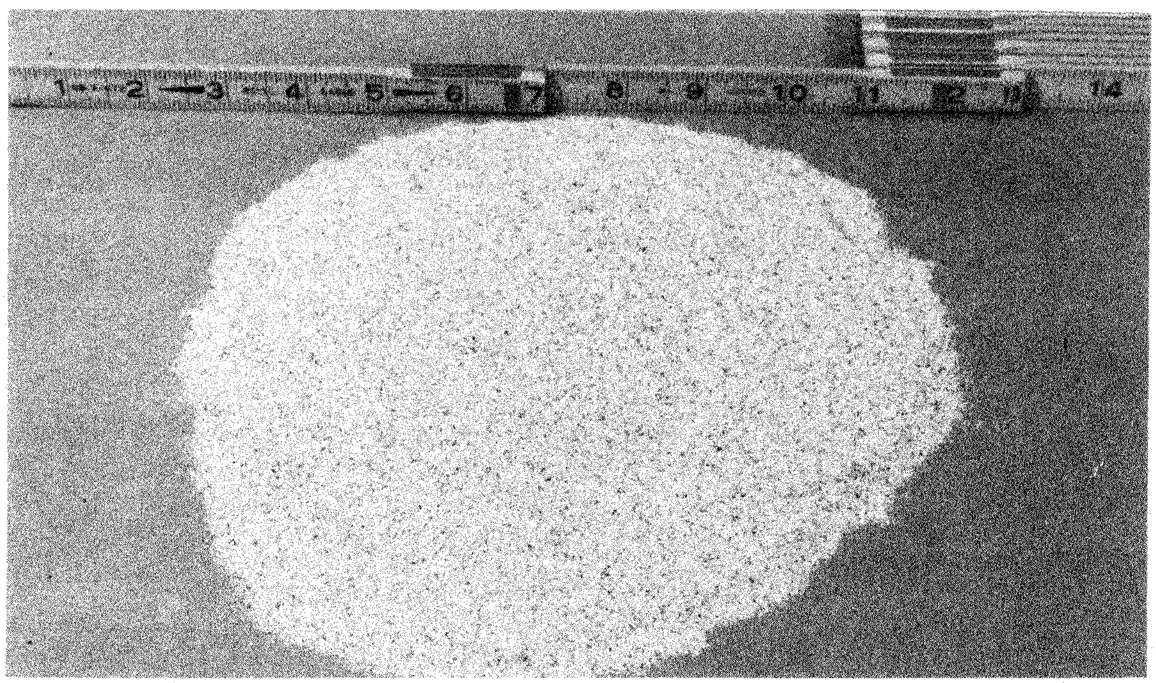
a. Coarse gravel



b. Fine gravel

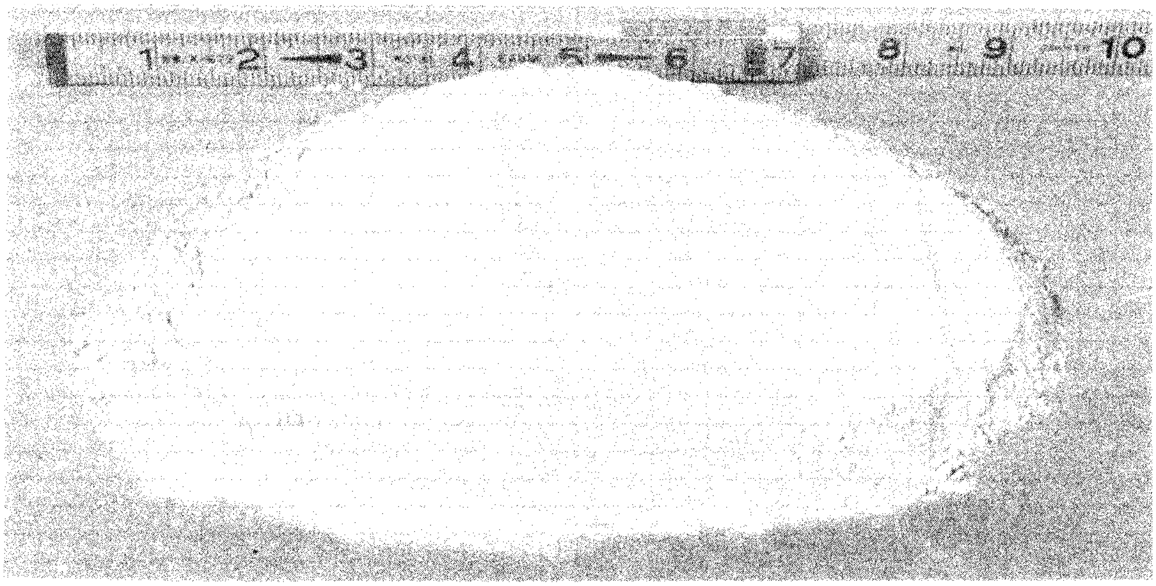
Figure E-1. Photographs of several soils to aid in selecting terms for describing the grain size of soil (Sheet 1 of 3)

c. Coarse sand



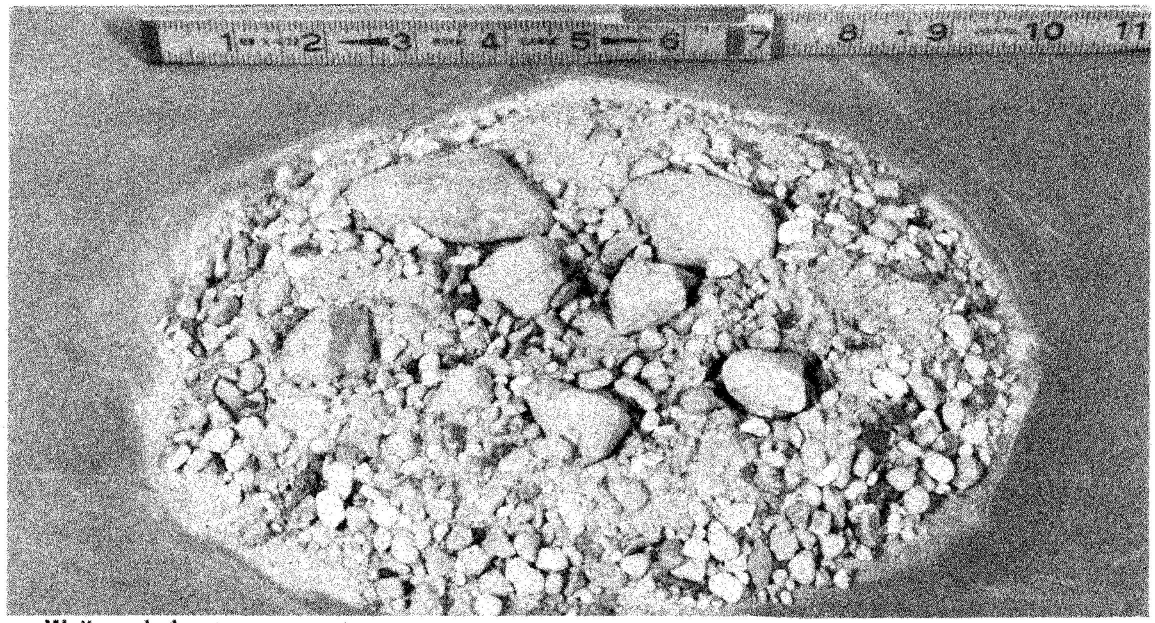
d. Fine sand

Figure E-1. (Sheet 2 of 3)



e. Silt or clay

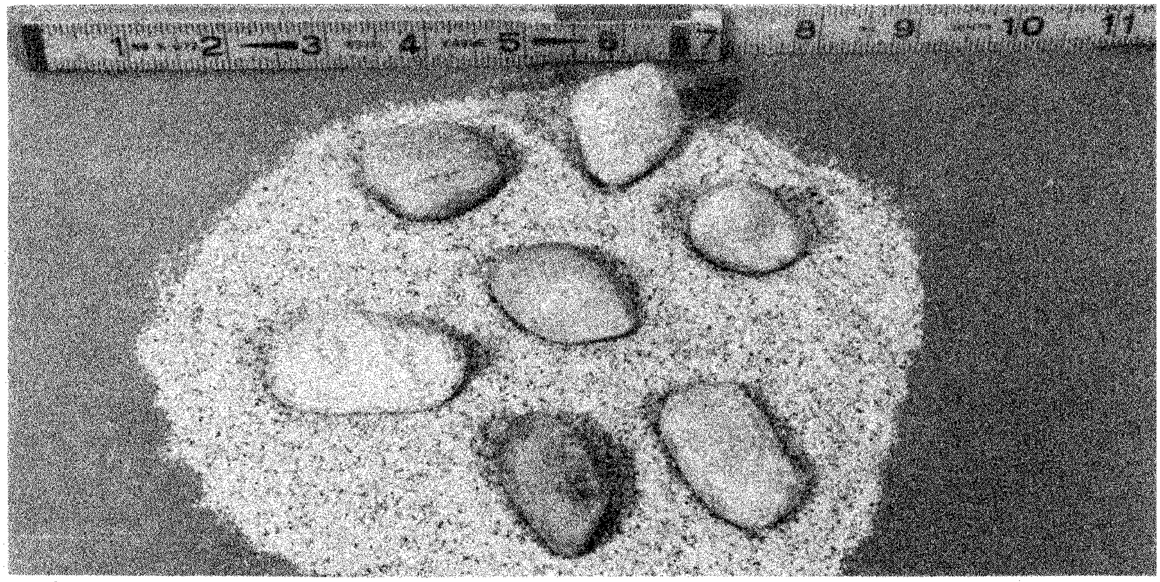
Figure E-2. Sheet 2 of 3)



a. Well graded

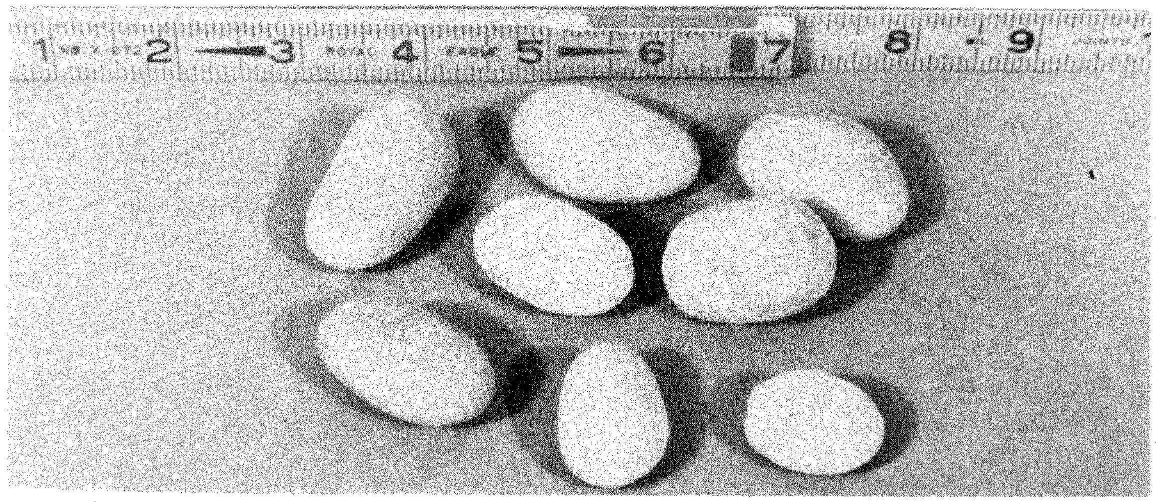
b. Uniformly or poorly graded

Figure E-2. Photographs of several soils to aid in selecting terms for describing the gradations of coarse-grained soils (Continued)

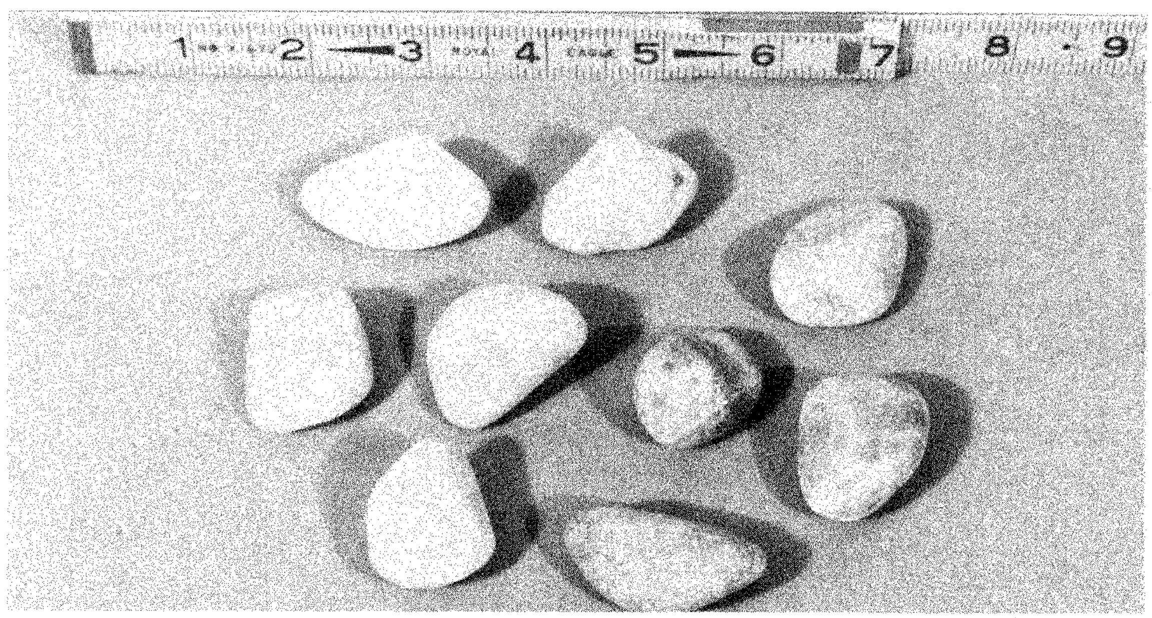


c. Gap graded

Figure E-2. (Concluded)

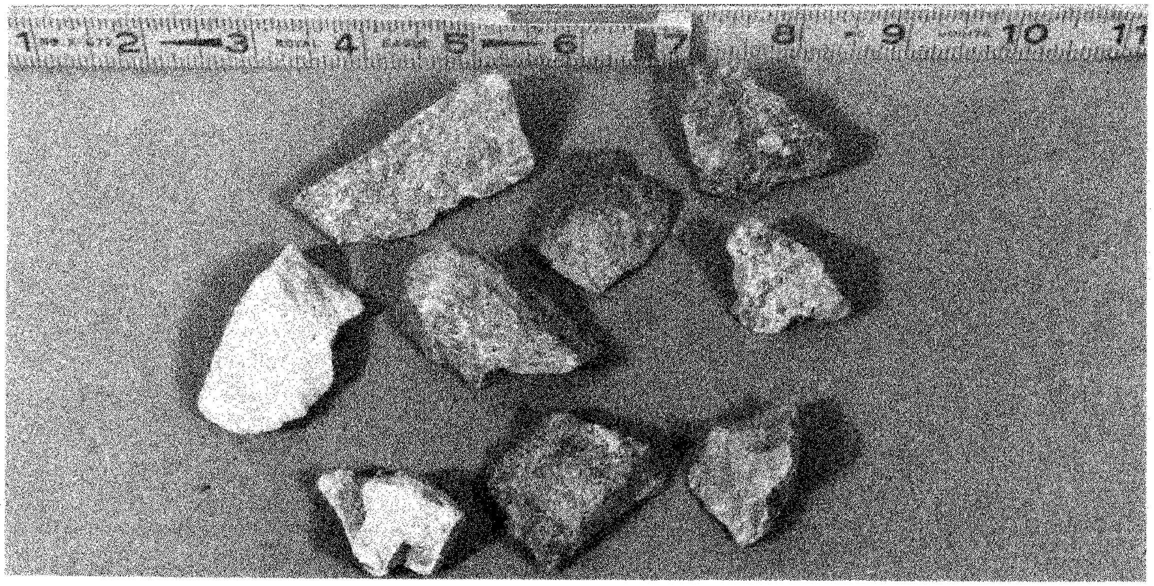


a. Rounded

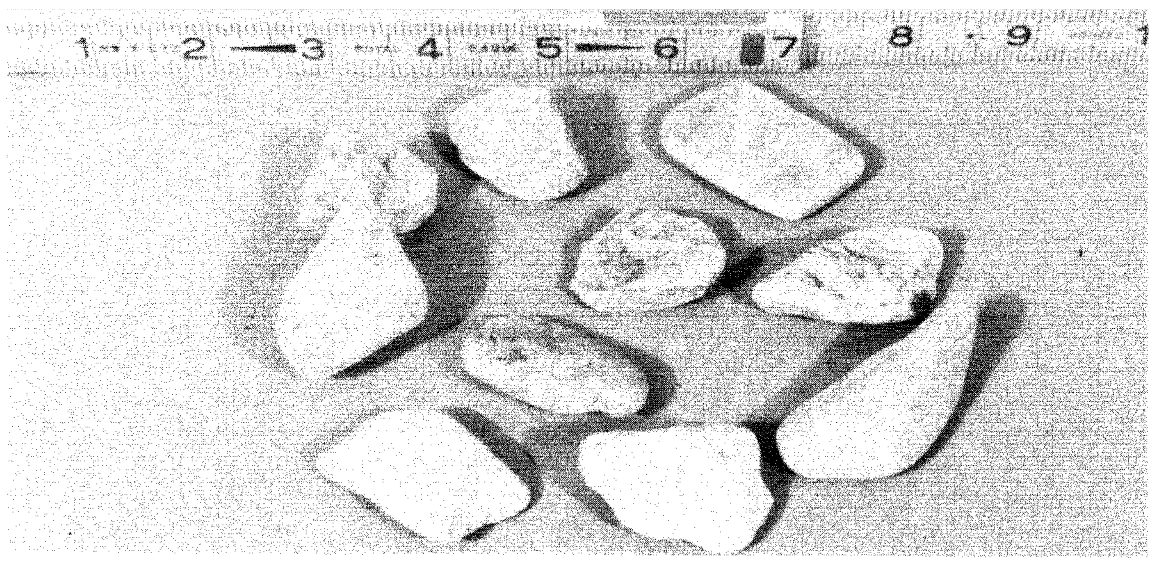


b. Subrounded

Figure E-3. Photographs of several soils to aid in selecting terms for describing the grain shape of coarse-grained soils (Continued)



c. Subangular



d. Angular

Figure E-3. (Concluded)

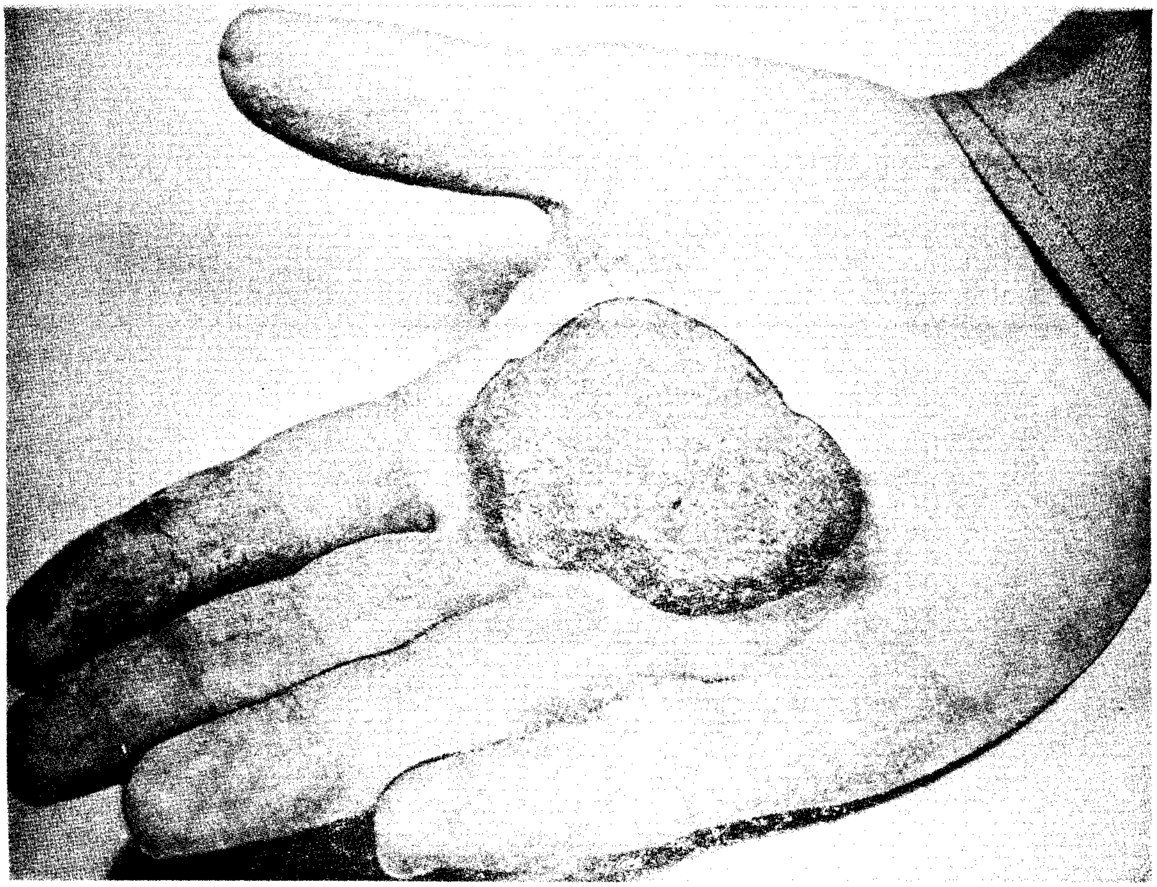


Figure E-4. Appearance of sample of moist, fine-grained soil prior to conducting the dilatancy test

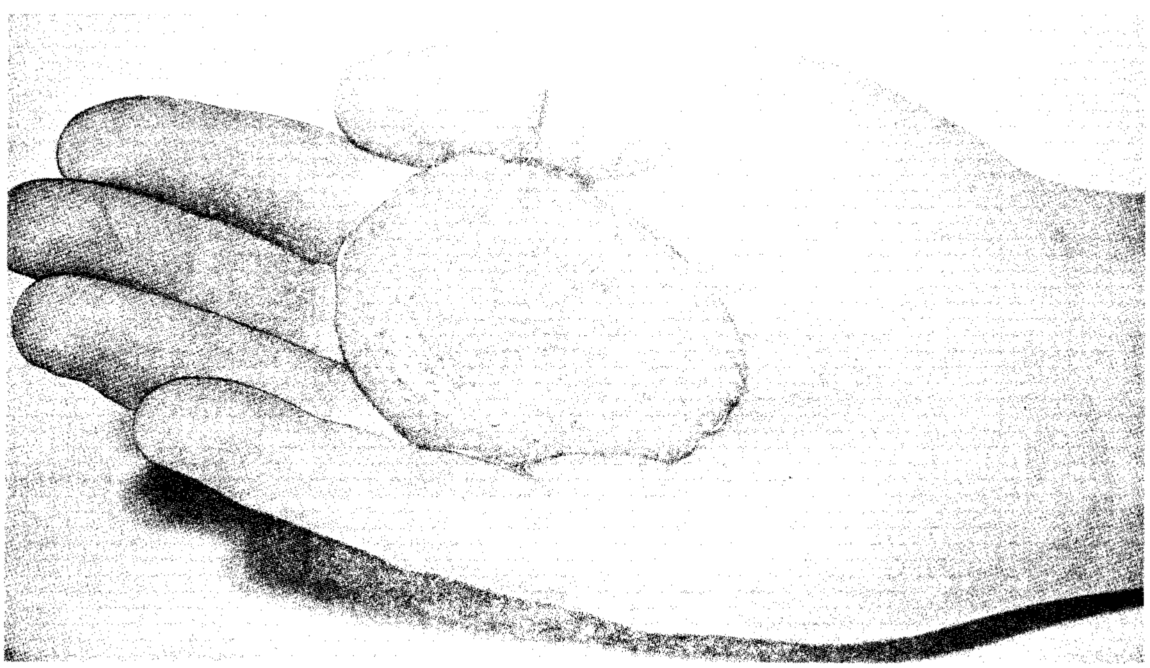


Figure E-5. Livery appearance of a sample of moist, fine-grained soil which occurred as a result of shaking during the dilatancy test

Figure E-6. Photograph of a sample of a moist, fine-grained soil being squeezed during the dilatancy test

Figure E-7. A sample of a moist, fine-grained soil cracking and crumbling during the dilatancy test

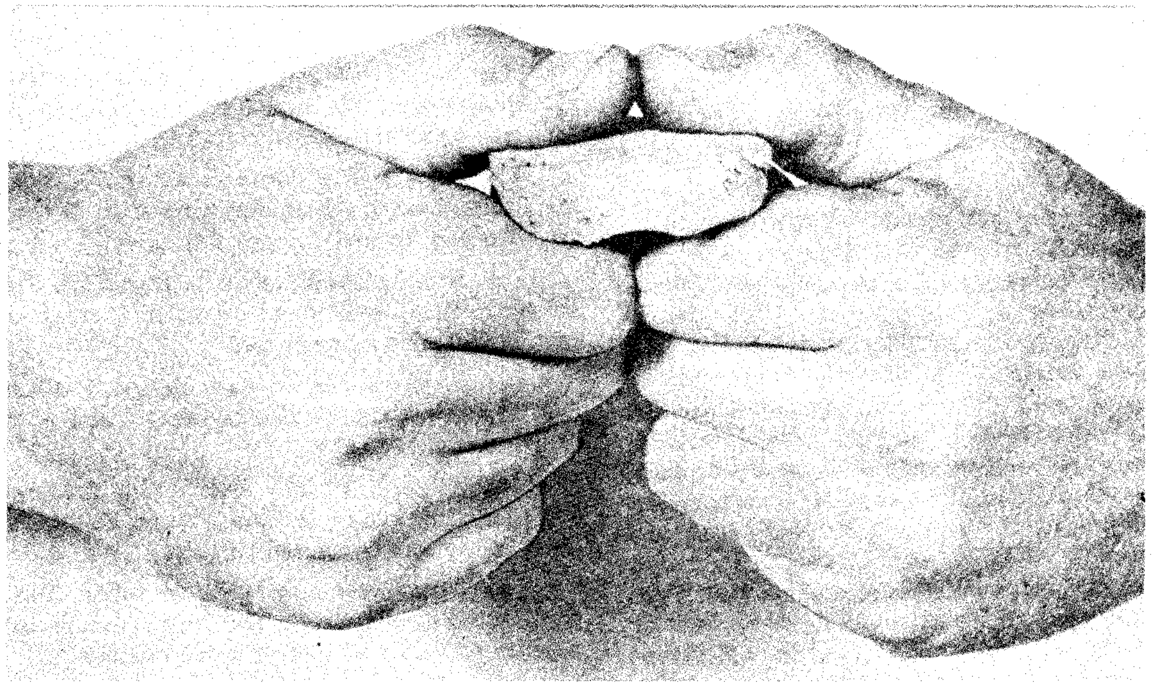


Figure E-8. A sample of a dry, fine-grained soil being broken to determine its dry strength

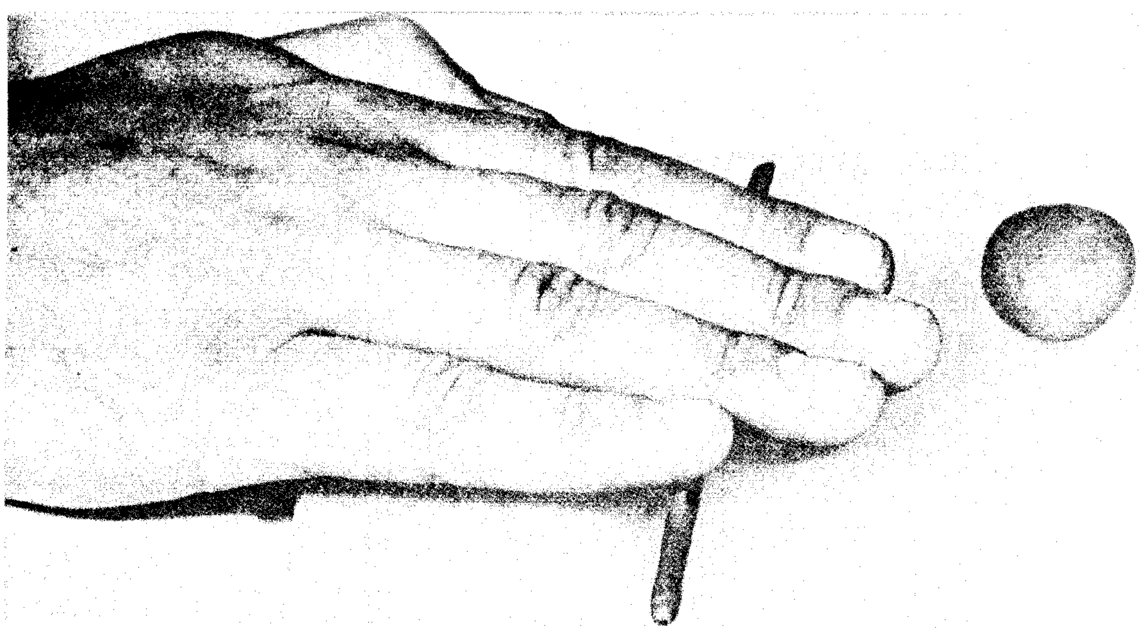


Figure E-9. A sample of a moist, fine-grained soil being rolled to a 3-mm- (1/8-in.-) diam thread to determine its toughness and plasticity

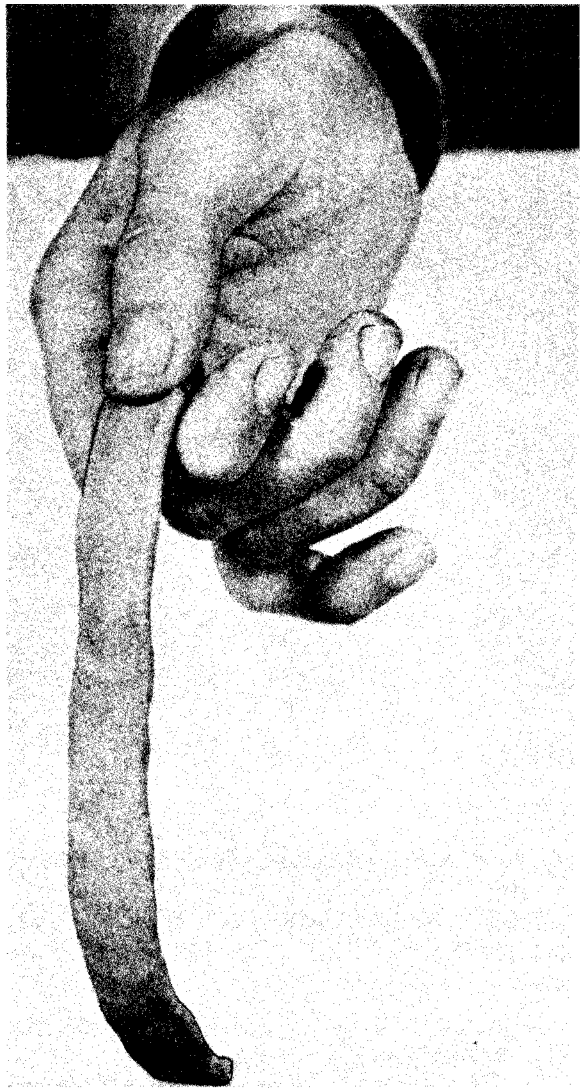


Figure E-10. Photograph of a sample of a moist, fine-grained soil being flattened to a ribbon about 3 to 6 mm (1/8 to 1/4 in.) thick to determine its toughness and plasticity