SLOPE STABILIZATION FOR LOCAL GOVERNMENT ENGINEERS IN MINNESOTA

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Mitchell Gregory Nelson

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Dr. David Saftner, Adviser

Dr. Carlos Carranza-Torres, Co-adviser

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ABSTRACT

Slope failures cause infrastructure damage, pose safety risks, and produce preventable maintenance costs. The purpose of this research was to recommend methods for stabilizing locally-maintained slopes requiring recurring maintenance in Minnesota. The author used input from county and municipal engineers to determine common techniques and identify slope failure examples. Site investigations helped develop case studies to analyze slope stabilization methods. Laboratory testing characterized representative soil strength properties. Additionally, the author developed Limit Equilibrium Method models for each slope to investigate different stabilization methods in a parametric study. Finally, modeling and analysis results were summarized in a guide for local government engineers. The target audience of the guide is county or local municipal engineers that do not have specialized geotechnical engineering experience. Additionally, modeling software enabled the development of slope stability charts specific to roadway embankments. The stability charts and guide will assist engineers in improving slope stability for roadway embankments.

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LIST OF ABBREVIATIONS AND SYMBOLS

c’	Effective cohesion
Co.	County
CSAH	County state aid highway
DCP	Dynamic cone penetrometer
DNR	Department of Natural Resources
EPS	Expanded polystyrene (geofoam)
eqn	Equation
FHWA	Federal Highway Administration
FS	Factor of safety
H	Slope height
kN	Kilonewton
kPa	Kilopascal
LEM	Limit Equilibrium Method
m	Meter
MN	Minnesota
MnDOT	Minnesota Department of Transportation
MSE	Mechanically stabilized earth
N_{60}	Field corrected blow count for penetration testing
pcf	Pounds per cubic foot
psf	Pounds per square foot
SPT	Standard penetration test
R	Radius of failure plane
$\overline{R}$	R normalized by slope height
S_u	Undrained shear strength
tsf	Tons per square foot
u	Pore water pressure
USCS	Unified Soil Classification System
USGS	United States Geological Survey
UTM	Universal Transverse Mercator coordinate system
x_c	Horizontal distance from slope toe to center of failure surface
$\overline{x}_c$	x_c normalized by slope height
y_c	Vertical distance from slope toe to center of failure surface
$\overline{y}_c$	y_c normalized by slope height
β	Slope inclination angle
γ	Soil unit weight
γ_w	Unit weight of water
σ’	Effective normal stress
τ	Shear stress
φ’	Effective friction angle
°	Degrees

1. INTRODUCTION

1.1. Slope Stability Overview

Slope stability is the ability of an inclined soil mass to resist lateral movement. Many slopes are designed to carry loads that are critical to infrastructure such as embankments for roadways and railroads, dams and large excavations. Slope stability analysis can range from detailed and technically sophisticated, such as a slope supporting a high-value structure, to non-technical and experience-based, such as routine maintenance on a rural county road embankment. Because of the large scope of slope failures and wide variety of stabilization methods, slope stability is a major topic in geotechnical engineering.

1.2. Need for Research

Slope failures on roadway embankments can damage pavement, pose safety hazards, and introduce preventable maintenance and repair costs. Figure 1 shows an example of slope failure on a county road causing substantial pavement damage and stranding a truck. The purpose of research for this thesis was to determine effective methods of stabilizing slopes along Minnesota's locally maintained roads and recommend stabilization methods for common site conditions. There is currently no guide to improve slopes typically seen along the state's locally-maintained roadways. While no single stabilization method is appropriate for all situations, several techniques including improving drainage, changing the geometry of the slope, and reinforcing the soil have proven effective. This study addresses the need to provide a consistent, logical approach to slope stabilization that is founded in geotechnical research and experience, and applies to common slope failures. Recommendations from this research will help engineers efficiently use their jurisdictions' resources to limit the negative effects of slope failure.

1.3. Scope and Summary of Work Performed

This research was closely related to a Minnesota Department of Transportation (MnDOT) project, *MnDOT Contract No. 99008, Work Order No. 190, Slope Stabilization and Repair Solutions for Local Government Engineers*, in which the author of this thesis was a key participant. The research recommended simple, effective methods of stabilizing locally maintained roadway embankments. The research did not address slope stability issues of the scale requiring local municipalities to hire geotechnical engineering consultants or specialty contractors. The target audience was county or local municipal engineers that do not have specialized geotechnical engineering experience.

The research identified and initially characterized slopes for further analysis via a survey sent to each county engineering department in the state. Respondents identified stabilization methods, and sites where field investigations could produce case studies. Next, the research identified and researched various stabilization methods in a literature review. Additionally, laboratory testing was conducted to more accurately characterize soil collected from slopes of interest. In a parametric study, Limit Equilibrium Method (LEM) models were developed to investigate the effect of various slope stabilization methods. The author used modeling software to develop dimensionless slope stability charts specific to roadway embankments. Finally, the research summarized the project's findings and presented recommendations in a slope stabilization guide for local government engineers. The stabilization guide – the main deliverable this research produced – will be available for engineers to improve the stability of the slopes they maintain. The author's team for site investigation, modeling, and chart development included two geotechnical engineering faculty advisers.

Figure 1: Example of damage from slope failure (from Houston Co. Highway Department)

2. BACKGROUND AND LITERATURE REVIEW

To provide slope stabilization recommendations, the author established background understanding of slope stability and stabilization methods. Identifying and understanding common stabilization approaches was necessary before developing LEM models.

2.1. Slope Stabilization Overview

Slope stability is typically quantified with a factor of safety (FS). The FS is the ratio of observed shear strength to the required shear strength for equilibrium at a specified location along a given potential failure surface. Equation 1 defines FS for slope stability.

$$\text{(Eqn. 1)} \qquad \text{Factor of safety} = \frac{\textit{in situ} \text{ shear strength of failure surface}}{\text{shear stress required for equilibrium of failure surface}}$$

There are several methods for determining a slope's FS. A common, state-of-practice analysis approach is the Method of Slices. The slope calculations involve dividing the slope into individual slices, and applying static equilibrium conditions to each. Figure 2 shows an example of the calculations for this method. The method analyzes a two-dimensional cross section of the slope. The Method of Slices was the approach used in slope stabilization analysis for this research.

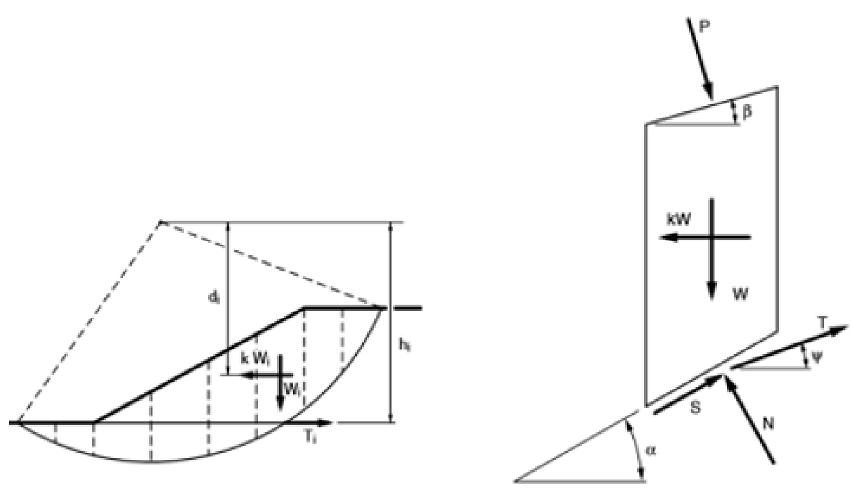


Figure 2: Example of the Method of Slices, from Duncan & Wright (2005)

Stabilizing a slope involves increasing the FS. Fundamentally, there are two ways to increase the FS and improve slope stability: introduce more stabilizing forces (increase capacity) or limit driving forces (decrease demand). Academic research and standard engineering practice have produced many slope stabilization methods; most fit into four categories: controlling groundwater and drainage, adding surface cover, excavating and regrading, and installing reinforcement and support structures.

Controlling water is important because groundwater can decrease the shear strength in a potential slide area. Surface cover can help protect the slope from water and erosion, and roots add stabilizing force to the soil. Excavation and re-grading decrease the forces that drive failure, increasing slope stability. Structural reinforcement directly adds supporting forces to the potential slide mass. Background research identified eleven general stabilization techniques. The literature review determined effective ways to stabilize the slopes.

2.2. Controlling Groundwater

A common cause of slope failure is the presence of groundwater in the soil. Water has a negative effect on soil ability to resist shearing, which leads to slope failure. An increase in pore pressure (due to groundwater) leads to a decrease in effective stress (σ'). Because σ' governs soil strength and deformation characteristics, the presence of water leads to decreased soil shear strength. Controlling groundwater in the slope area is a fundamental way to increase the resistance to shear failure.

Drainage decreases the amount of water in the slope material. Surface drains, trenches, horizontal drains, and drainage wells are methods to control water. Surface drains limit the amount of infiltration soil encounters. Trenches, drains, and wells divert water after infiltration; their construction varies by project type. Rahardjo et al. (2003) describes a study in which drainage features were used to increase slope stability. The project investigated placement of horizontal drains in areas with intense annual rainfall. The research concluded that placement of drainage features in the lower part of a slope can increase stability.

There are many installation methods and drainage configurations to consider. Site conditions and contractor experience often govern dimensions, spacing, and layout of drainage features. A general suggestion is to place drains close to the failure zone, and near the steepest angle exhibited by the slope (Stanic, 1984). In gravity-controlled horizontal drains, the most benefit is generally observed with placement near the toe. Local conditions typically control method selection.

Although they vary greatly in material, size and installation method, drainage pipes all have a common goal: remove water that has already infiltrated into the potential slide area.

Teams can place horizontal pipe drains into existing slopes or install them during construction. Installing the drains with a slight slope allows water to drain into a collection channel and away from the slope stabilization area. Figure 3 shows a simple example of drainage feature use. Horizontal pipe drains at the bottom of trenches can channel water away.

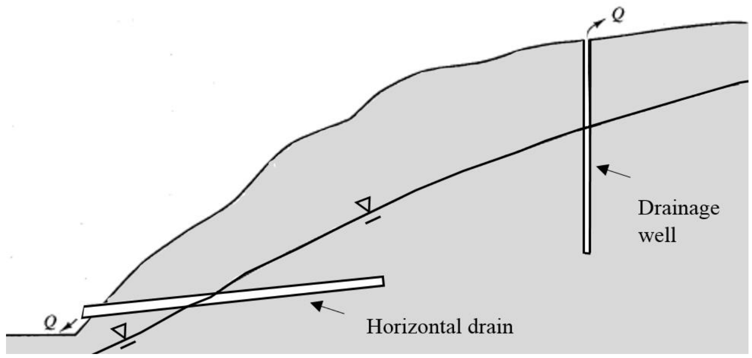


Figure 3: Example of common drainage features (from Coduto et al., 2011)

Drainage trench installation does not require specialty subcontractors. Digging an open channel to divert water can be a simple solution to excess groundwater concerns. Trenches can be good temporary solutions for drainage during construction. If the excavation is backfilled with a free-draining material, the trench can serve as a permanent drainage feature. Multiple drainage features used together can be most effective.

The main benefit of drainage is limiting pore pressure's effect on shear strength. When construction is necessary in areas that are affected by groundwater, a more involved technique is required. Dewatering effectively lowers the groundwater table, and is typically performed with a pump system. Dewatering can involve extensive work and cost. If required to perform construction below the groundwater table, dewatering pumps and drainage channels can increase shear strength and minimize related slope stability concerns due to water.

2.3. Surface Cover

Another method of stabilizing slopes is using surface cover to resist failure. Appropriate soil cover can increase stability by diverting water, limiting the effects of erosion, and providing stabilizing forces to the upper layer of a slope. Vegetative cover, rip-rap, replacement fill, and buttressing are common methods involving ground cover.

Using vegetation as ground cover is common and easily implemented. Vegetative cover helps protect slopes from erosion with aesthetic and environmental benefits. Grass and other vegetative cover protect the slope material from the impact of rainwater and surface runoff. Operstein & Frydman (2000) presented the effect of plant roots on soil shear strength. The research involved testing the mechanical properties of plant roots. Each vegetated soil type had a shear strength greater than that of the original soil. Plant roots remove water from the soil, limiting the effect of pore pressure on the slope material. Roots also provide mechanical reinforcement at the surface. Generally, vegetated slopes are more stable than slopes of the same soil type without vegetative cover.

Another advantage of vegetative cover is ease of installation. No specialty equipment is required. Grass seed can be easily placed at low labor cost, and material is readily available. Installing surface vegetation after a slope stabilization project is common practice. Figure 4 shows a site demonstrating steep slopes with vegetative cover.

Figure 4: Naturally vegetated slope in Lac Qui Parle Co., MN

Buttressing is placing a soil or rock mass against a slope face to add stabilizing force and decrease the overall slope height, as Figure 5 illustrates. Buttressing can be as simple as pushing material against the slope. Temporary buttresses can provide cover and stabilizing support for construction projects. For temporary slopes left steeper than ideal, buttressing is a way to add stability.

Figure 5: Simple example of buttressing

Placing coarse gravel or cobble rip-rap on the slope surface can provide protection from erosion. Figure 6 shows a large slope entirely covered with quarried cobble. Rip-rap placement is labor-intensive and generally has a higher cost than earthwork buttressing, but can provide erosion protection. Using high-flow concrete (shotcrete) is an option with similar cover advantages. Many cover methods are primarily erosion control; adding cover material can increase the weight of a slope and decrease global stability, depending on the material and site geometry.

Figure 6: Slope covered with rip-rap in St. Louis Co., MN

2.4. Excavating, Regrading and Decreasing Load

When possible, altering slope geometry can minimize the forces driving failure. If spatial concerns (i.e. jobsite and right-of-way boundaries) are not an issue, decreasing the slope inclination angle is an option. Another way to decrease driving forces is to remove any load or surcharge from the slope, limiting extra weight. For example, Cornforth (2005) describes a case study of slopes surrounding the Pelton Dam in central Oregon. Crews repaired the slopes with a lower inclination angle and observed significant decrease in slope failure.

Using lightweight fill can decrease slope weight and lower driving forces. Abramson et al. (2002) identifies expanded shale, shredded tires, encapsulated sawdust, seashells, and polystyrene foam as some examples of lightweight fill. Material choice depends greatly on local availability and transportation costs. Lightweight fill is a design consideration for new slope construction and a material option for slope repair.

Free draining compacted fill has ideal properties for slope material. Replacing slope soil with engineered fill (i.e. clean sand or tested borrow material) minimizes uncertainty in ground conditions and eliminates factors that lead to slope instability. Removing *in situ* soil and placing fill also allows the design team to control the geometry of the slope. However, space and budget considerations can make this method impractical. In cases where the remove-and-replace option is appropriate, proper fill selection improves *in situ* strength and drainage properties. Figure 7 shows one such case, where a convenient source of fill was located near the site, making the remove-and-replace method feasible.

Figure 7: Slope reconstruction project in Murray Co., MN (From Murray Co. Highway Department)

Benching, or excavating horizontal cutouts periodically along the slope, can stabilize temporary excavations and permanent embankments. Benching, also called terracing, allows the use of sections steeper than the overall slope. There are restrictions on benching geometry, especially in temporary construction excavations. Local building codes and safety committees are sources for benching dimension guidelines. The benches along an excavation provide a convenient flat surface for workers and equipment. Installing drainage features on benches increases long-term stability. Figure 8 shows an example of a benched embankment.

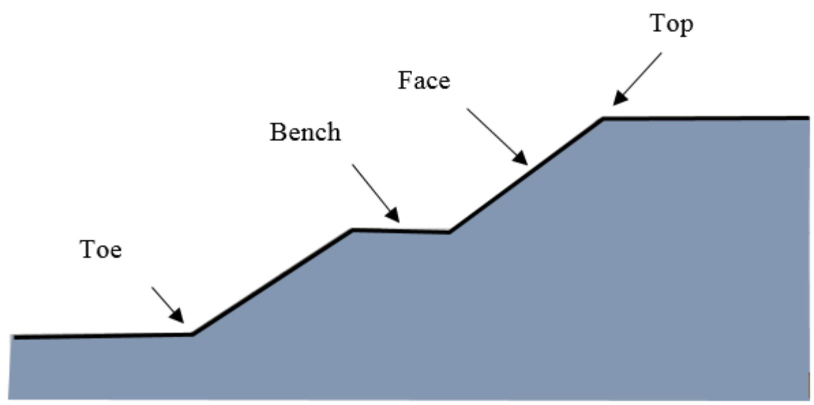


Figure 8: Example benched slope (adapted from Coduto, et al. 2011)

2.5. Reinforcing Structures

Installing reinforcing structures can increase resisting forces and improve FS. Retaining walls, soil nailing, ground anchors and mechanically stabilized earth (MSE) walls are examples of stabilizing structures. Most reinforcing structures require specialized experience and are likely more suitable solutions for large projects. There are standardized design approaches for many stabilizing structures. Although reinforcing structures are an expensive option, they are sometimes necessary to stabilize slopes.

Retaining walls are an option for projects in which space is an issue. A well-designed and constructed retaining wall enables design teams to work around severe grade changes in some highway projects. The Federal Highway Administration (FHWA) is a source for design guidelines for retaining walls, such as Christopher et al. (2009). Retaining walls can manage grade changes in roadway construction, keep salt, oil and other highway chemicals off the surrounding environment, and protect motorists from rocks, wildlife and other hazards that could enter the roadway. Figure 9 shows a simple retaining wall example.

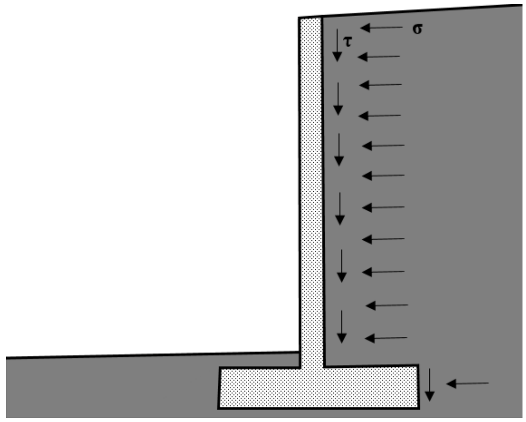


Figure 9: Retaining wall example with lateral earth pressures shown (from Coduto et al., 2011)

Soil nailing is another reinforcing structure method. Generally crews drive a pile, rod or pipe into soil to provide mechanical stabilization. This method is most effective when geotechnical modeling or analysis can approximate the failure surface, allowing the nails to reach stable soil. In some cases, placing cement grout anchors the structure. In rock slopes, drilling and grouting creates an anchor, or rock bolt. Figure 10 shows a sample design. Soil nails and rock bolts can be combined with other stabilization methods, such as surficial cover, as Figure 11 shows. Cables can also support retaining walls as tieback anchors. Soil nail, rock bolt, and tieback installation typically requires a specialty contractor.

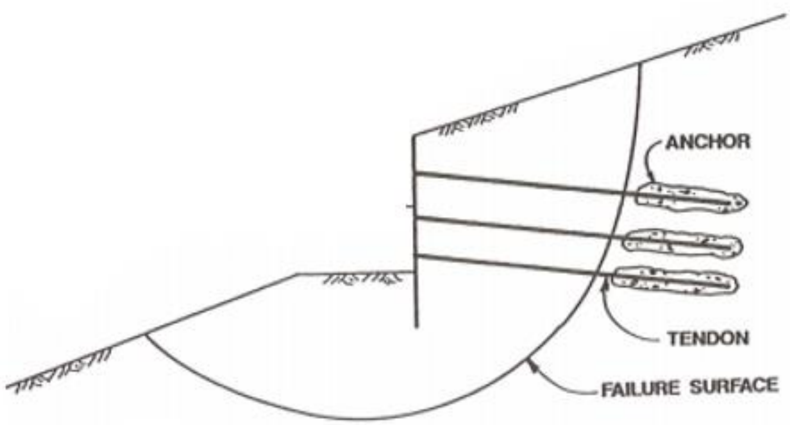


Figure 10: Structural reinforcement features extend past the failure surface to provide stability (from Abramson et al., 2002)

Figure 11: Rock bolts in combination with mesh cover in Washington Co., MN

Teams can drive structural reinforcement features like foundation piles, or pre-drill and grout them. Helical piles can be screwed into soil for placement. While helical pile installation is a specialty operation, it avoids the disruption and noise of driving nails.

Geosynthetic reinforcement is another stabilization option. The term “geotextile” describes a permeable fabric. The term “geogrid” typically refers to a lattice-pattern synthetic that is placed between layers of fill material. Figure 12 shows an example of geogrid. Westfall (2014) describes how geogrid was used in combination with other stabilization methods along U.S. 50 in Nevada, near Lake Tahoe. The Nevada Department of Transportation hired engineering consultants and used proprietary designs, indicating that this repair method is likely not an in-house stabilization method for county maintenance engineers. The geosynthetic material allowed the project to meet environmental and aesthetic requirements. Geosynthetics are often considered because of ease of installation, and work well in combination with other stabilization methods.

Figure 12: A worker anchors geogrid at the Lake Tahoe project (from Westfall, 2014)

Mechanically stabilized earth (MSE) embankments are simply a combination of several stabilization methods. Generally more common in new construction, an embankment is constructed using prescribed fill placed in compacted lifts with geosynthetic reinforcement between layers. Fill is typically free-draining borrow material, unless the site has adequate *in situ* drainage and strength. Teams may also install drainage features. This embankment type stabilizes slopes but is generally expensive. The FHWA is a good source for design guidelines and standards for MSE walls and other structural reinforcement methods (Berg et al., 2009).

2.6. Literature Review Summary

There are many options for slope stabilization and repair, and method selection is site-specific. The literature review provided background information on many common techniques. Managing groundwater and drainage typically increases shear strength. Surface cover helps protect slopes from erosion. Excavation and re-grading minimize forces that cause failure. Structural reinforcement adds direct supporting forces to slope material. Research produced an understanding of eleven general stabilization techniques. Table 1 summarizes the stabilization methods researched, along with a source of background information demonstrating each method's application.

Table 1: Slope stabilization methods researched

Stabilization Method	**Source of Background Information**
Drainage pipes, wells, and channels	Cornforth (2005) Ch. 17
Dewatering	Coduto et al. (2011) Ch. 11
Vegetation	Abramson et al. (2002) Ch. 7
Buttressing / rip-rip	Abramson et al. (2002) Ch. 7
Geosynthetics	Gee (2015)
Remove and replace	Duncan & Wright (2005) Ch. 16
Re-grading and benching	Cornforth (2005) Ch. 15
Lightweight fill	Abramson et al. (2002) Ch. 7
Retaining walls	Cornforth (2005) Ch. 19
Soil nails / rock bolts / tieback anchors	Abramson et al. (2002) Ch. 7
Mechanically stabilized earth embankments	Abramson et al. (2002) Ch. 7

3. METHODS AND APPROACH

The project intent was to create a guide focusing on locally-maintained slopes requiring recurring maintenance. A number of tasks including engineering research, material testing, computer modeling, and comparative analysis led to recommendations for the guide.

3.1. Summary of Research Tasks

First, case histories representative of the types of issues currently experienced by Minnesota county engineers were identified. Next, a literature review identified stabilization methods to apply to case studies for analysis. Laboratory testing provided representative soil strength properties for case study sites. LEM models quantified the effects of stabilization methods at each site and enabled development of slope stability charts for municipal engineers to use on roadway slope design projects. Producing the deliverable involved analysis of modeling results to determine the most appropriate stabilization methods for various conditions.

3.2. Initial Survey

Ensuring slope stabilization recommendations were practical for local government agencies was critical. Accordingly, a survey for Minnesota county engineers was developed to determine common slope stabilization methods. The survey asked engineers to identify slopes within their jurisdiction requiring recurring maintenance, and successful and unsuccessful methods of maintaining those slopes. Correspondence with survey respondents allowed scheduling of site visits and development of case studies. County maintenance engineers received surveys in September, 2015; fourteen engineering departments responded. Each site visit corresponded to a responding jurisdiction. Figure 13 shows the location of each site investigation.

3.3. Site Investigations

During site visits, the author's investigation team measured slope geometry with a field tape measure and surveying equipment (Figure 14). Soil type and strength properties were also evaluated. *In situ* investigation with a vane shear test and pocket penetrometer indicated soil's undrained shear strength (S_u) in tons per square foot (tsf). The pocket penetrometer is a tool that is inserted into soil with a spring that is calibrated to relate resistance to undrained shear strength. The vane shear test includes a probe with fan blades that is inserted into soil and twisted; the tool correlates torsional resistance to S_u.

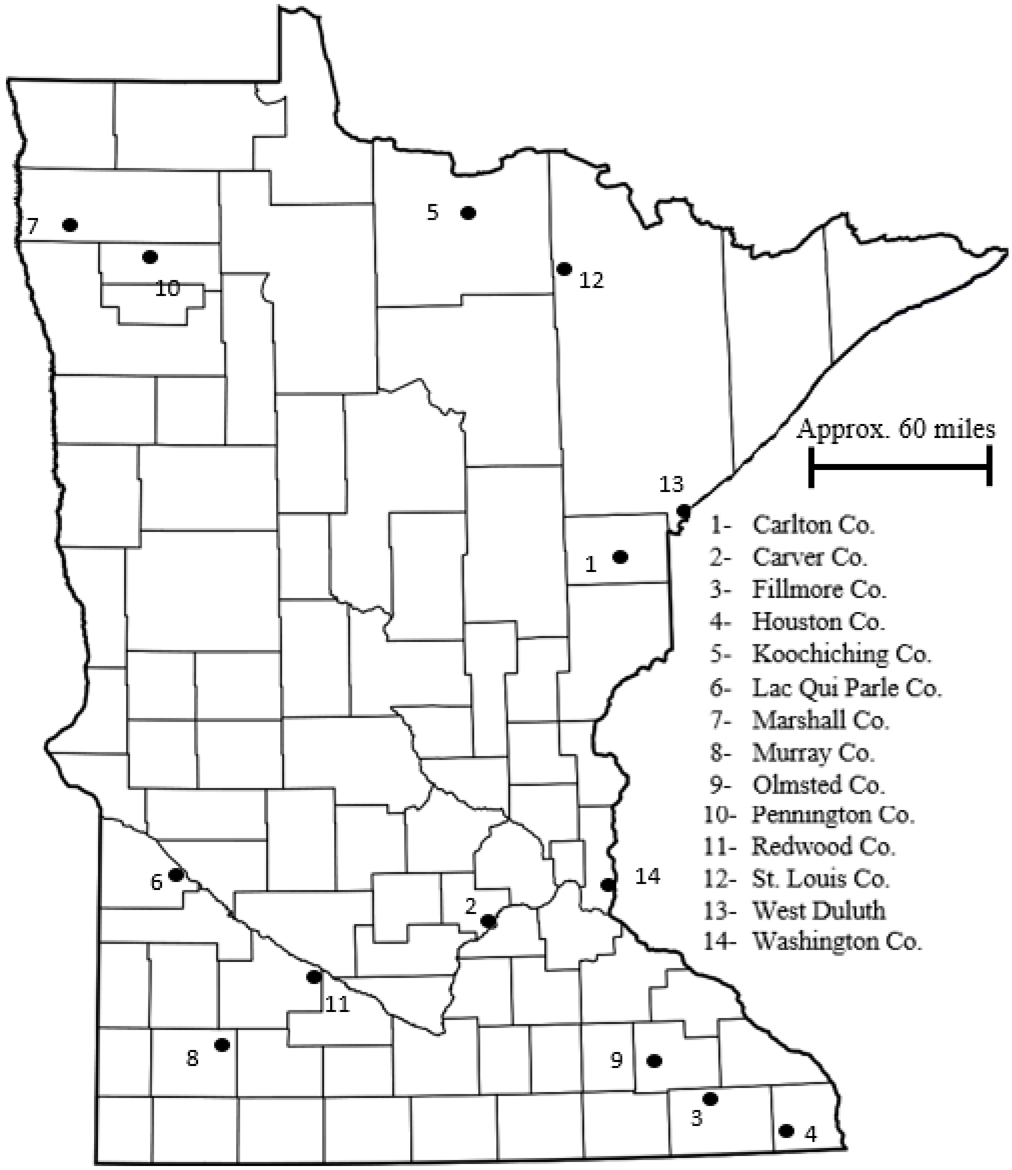


Figure 13: Approximate locations of Minnesota site visits for data and sample collection

Soil samples were collected with a hand auger. Soil was classified according to the Unified Soil Classification System (USCS). Where site investigation data was not available, *in situ* indications of soil density were collected using the dynamic cone penetrometer (DCP). The amount of blows to advance a probe struck with a standard weight a given distance was recorded.

Figure 14: Field survey equipment used to determine slope geometry

The DCP test is more easily implemented than full-scale geotechnical field testing. The test is the most convenient way to directly measure *in situ* resistance to penetration. Testing was conducted in general accordance with *ASTM D7380 – 15*. Figure 15 shows a positive correlation to standard penetration test (SPT) values, a more traditional geotechnical field parameter. The plot provides a 1:1 line for reference. The value of SPT blow counts is the field corrected SPT blow count, or N_{60}. Blow counts allowed estimation of the *in situ* density for replication in lab testing. Testing did not contribute to assessment of strength properties, but provided a relative comparison of soil density and an additional *in situ* test parameter to characterize sites. *In situ* tests such as the DCP and S_u tests were conducted several times at each site with the representative average range reported to characterize soil strength.

Photographs and field notes captured the site cover, erosion potential, drainage, and groundwater conditions. Ojakangas (2009) provided an overview of each site's geologic background, as did a geologic map from the Minnesota Geological Survey (Hobbs & Goebel, 1982). Depth to the groundwater table was available from Minnesota Department of Natural Resources (DNR) monitoring wells.

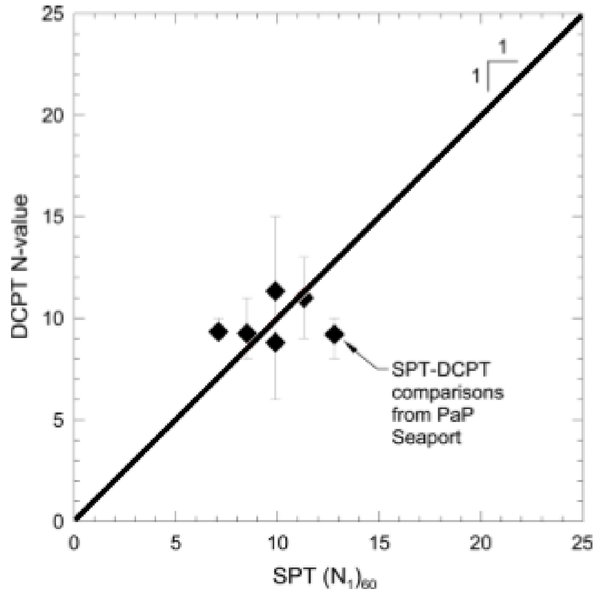


Figure 15: Relationship between DCP and SPT blow counts (from Green et al., 2011)

The presence or absence of visible failure planes was noted and failure types were classified following a FHWA design manual on soil embankments (Collin et al., 2001). Most commonly observed failure types were creep failures and rotational slide failures. Figure 16 shows examples of each. Creep slides are slow, surficial failures involving gradual downhill movement of material. Visible displacement and bio-indicators (i.e. trees that grow crooked) are characteristics of creep movements. Seasonal freeze-thaw cycles can cause creep failure in soils with inadequate shear strength properties. A circular failure plane in cross section is characteristic of rotational failure. This failure type typically leaves exposed soil, called scarps. In some soil types, cracking at the surface can indicate the slope is nearing a rotational failure.

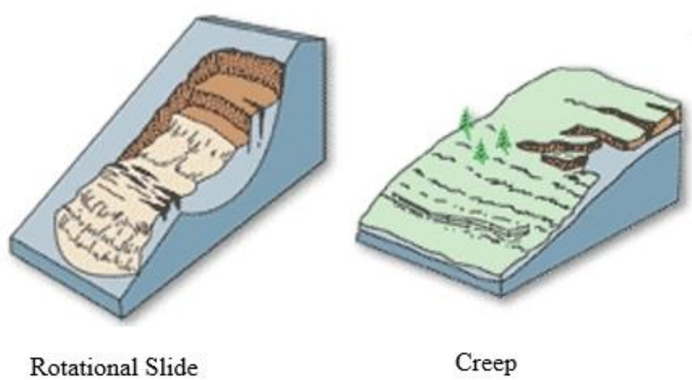


Figure 16: Examples of common slope failure types (from Varnes, 1978)

The goal of site investigations was to compile a representative set of case studies for analysis and modeling. Appendix A provides supplementary information for each site visit including soil properties and slope characteristics summaries.

3.4. Laboratory testing

The author determined soil strength properties from the direct shear test. Slope failures are examples of plane strain, and direct shear test specimens fail in the same way. This similarity in failure mechanism made slope stability and failure modeling a good application of direct shear testing. Tests were conducted in general accordance with *ASTM D3080-11*, with samples loaded into a mold and failed in shear, instrumentation to measure the force applied and sample displacement throughout the test, and graphical methods to determine output values. The outcome of direct shear testing was shear strength parameters representative of each soil, particularly effective friction angle (φ') and effective cohesion (c'). Use of saturated samples eliminated the effect of negative pore water pressure that may occur with partially-saturated samples.

Soil classification is a basic part of geotechnical investigations. Many stabilization and construction practices depend on soil type. The author conducted Atterberg Limit testing in general accordance with *ASTM D4318-10* to determine two qualitative soil properties of each sample: the plastic limit and liquid limit. *ASTM 2487-11* describes how to use these parameters to determine if fine soil samples classify as silt or clay. Sieve analysis helped classify soils containing granular soil. With gradation and behavior qualities, USCS soil classifications were assigned to each sample. Another property determined from lab testing was the moisture content of each site sample. This value helped determine how groundwater affected sites.

3.5. LEM Modeling

Slope stability modeling was conducted with the Rocscience program SLIDE (Rocscience, 2016). The baseline FS for each site was determined by executing LEM models with *in situ* shear strength properties, slope geometry, unit weight (γ), and groundwater conditions. Unit weight is a measure of soil's weight per unit volume, such as pounds per cubic foot (pcf).

The modeling program determined slope FS using the Method of Slices. The software generated output values using the Bishop method of stability analysis, a state-of-practice implementation of the method of slices. Abramson et al. (2002) provides a detailed description of different analysis types. The program conducted analysis following the setup outlined earlier in Figure 2. Combining the forces acting on each of the slices due to soil properties, geometry and external factors allowed the program to determine the overall slope FS. Since slope stability analysis involves performing calculations numerous times (i.e. considering a significant amount of trial failure surfaces to determine the critical failure surface), it was a good application of computer modeling. The output from executing each model was a rendering of the slope and site conditions, the lowest computed FS, and the critical failure surface. The calculations are based on static equilibrium, so the code defines failure as the force required to move the soil, and take the slope out of static equilibrium. An example is given in Figure 17, showing slices generated during computation. The default SLIDE failure mechanism is a circular surface.

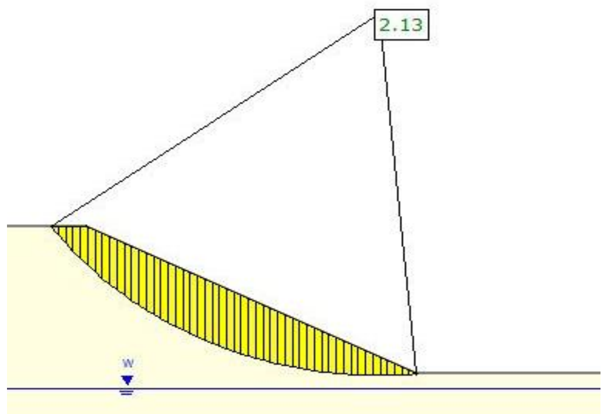


Figure 17: Example SLIDE slope stability model output

With the baseline FS for each site, the effect of various stabilization methods could be examined. By comparing the output FS before and after implementing a stabilization method, the author could quantify the effectiveness of the given approach. By modeling the same slope with a different stabilization technique, and using the same quantitative analysis, the most effective stabilization method was determined. Following this parametric study approach, the author developed a relative understanding of how much each technique improved slope stability. Analysis of all sites provided a case-by-case comparison. Generalizations of the type 'when *these* conditions are present, *this* appears to be the most effective stabilization option' helped outline the final deliverable.

3.6. Infinite Slope Analysis

The output from the LEM modeling program is the slope's critical circular failure surface. Therefore, at sites with creep failure, the modeling does not accurately represent the observed site. Infinite slope analysis uses a simple calculation that considers slope inclination angle (β), soil effective friction angle (φ'), soil unit weight (γ), and unit weight of water (γ_w). Figure 18 shows an infinite slope analysis example for a purely frictional soil (i.e. c' = 0); the equation for FS for dry slopes is shown in Equation 2, and FS for saturated slopes is shown in Equation 3 (Abramson et al., 2002).

(Eqn. 2) $$FS = \frac{\tan(\varphi')}{\tan(\beta)}$$

(Eqn. 3) $$FS = \left(\frac{\gamma - \gamma_w}{\gamma}\right)\frac{\tan(\varphi')}{\tan(\beta)}$$

The c' = 0 assumption simplifies the analysis to the provided equations. Since no sites were pure sand, all soil had some cohesion; therefore, the FS from infinite slope analysis was conservative for some sites. Sites composed primarily of sand exhibited high output FS from LEM modeling, and conservative FS from infinite slope analysis. SLIDE outputs of some sites showed circular failure planes with large radii, indicating that infinite slope analysis with the given assumptions would be a better way to asses these sites' stability. The author considered infinite slope analysis when it was more applicable to address a concern in the validity of LEM modeling.

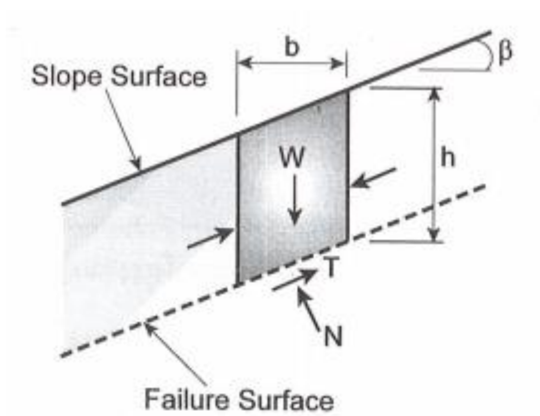


Figure 18: Example of Infinite Slope analysis, from Abramson et al. (2002)

4. RESULTS, ANALYSIS AND DISCUSSION

During analysis, slope properties were condensed to a number of given field scenarios, and effective stabilization methods were determined for each. By comparing field conditions to a given slope in their jurisdiction, end users can determine which scenario most closely represents the given site and view general recommendations.

4.1. Minnesota Site Visits and Case Studies

The investigation procedure was applied at the site specified by each survey respondent. The closest DNR groundwater depth monitoring well to each site is available in Appendix B.

4.1.1. Carlton County Site

The Carlton County site was located on CSAH 6, approximately 7 miles east of Barnum, MN, near County Road 103. The assistant county engineer described the project history. The failure was identified by pavement distress. The site cover was tall grass, as Figure 19 shows. *In situ* material was red to brown clay with sand seams. The sand was brown to light brown and fine to medium-grained. Field testing indicated an average undrained shear strength (S_u) of 1.25 to 1.5 tsf. DCP testing indicated N_{60} values were approximately 2 blows per foot. There was no observed groundwater during sampling. Ojakangas (2009) identified the site's surficial geology as glacial drift. The observed fine-grained soil is representative of glacial deposits. The geologic identification is consistent with Hobbs & Goebel (1982). There were no observed slope stabilization attempts.

4.1.2. Carver County Site

The Carver County site was located on County Road 40, at the Minnesota River north of Belle Plaine, MN. The county highway operations manager recommended the location in the initial survey. Minor pavement distress was evident at the site. The guardrails along the road were tilted slightly from vertical, indicating soil creep, as Figure 20 illustrates. Site cover was primarily tall grass.

Figure 19: Carlton County site slope

Figure 20: Carver County site slope, crooked guardrails indicate soil creep

Soil was brown to dark brown sand with silt. DCP testing indicated an increasing resistance to penetration with depth. Correlated N_{60} values ranged from 3 to 4 blows per foot. S_u values were between 0.5 and 0.75 tsf. The Minnesota River was near the toe of the slope, indicating possible groundwater concerns. Geologically, the site was made up of sediment from the Minnesota River (Ojakangas, 2009; Hobbs & Goebel, 1982), consistent with observed soil. No slope stabilization techniques were observed.

4.1.3. Fillmore County Site

The Fillmore County site was located on CSAH 5, approximately 5 miles southwest of Chatfield, MN. The county engineer identified the location in the initial survey. The site exhibited significant pavement distress, as Figure 21 illustrates. The Middle Branch Root River was near to toe of the slope. A large failure with clear scarp lines, characteristic of rotational slide failure (Figure 22 and Figure 23) was present. The site cover was tall grass. Topsoil extended 6 to 12 inches below the surface.

Figure 21: Pavement distress at Fillmore County Site

The observed soil was brown, fine-grained sand and silt. S_u values ranged from 1.25 to 1.75 tsf. DCP testing indicated soil's resistance to penetration increased with depth. The soil's correlated N_{60} values were approximately 4 to 5 blows per foot. The geology of the site was weathered material on bedrock (Ojakangas, 2009; Hobbs & Goebel, 1982), as supported by observed soil. With the toe of the slope near a stream, groundwater and drainage conditions were concern. As with several other slopes studied, the side of the road near a stream was failing, illustrating the importance of drainage. No slope stabilization attempts were evident during the investigation.

Figure 22: Clear rotational failure visible behind researcher at Fillmore County site

Figure 23: Scarp face at edge of failure at Fillmore County site

4.1.4. Houston County Site

The Houston County site was located on County Road 19, near Riceford Creek, approximately three miles northwest of Spring Grove, MN. The section of roadway was unpaved. A county engineering technician highlighted the project history and details onsite. The technician provided slope geometries from before the failure. The uphill side of the road exhibited no slope failure that affected the roadway. The downhill side of the slope was the subject of the repair and stabilization, shown in Figure 24. The slope's toe immediately bordered the creek. The technician indicated that large flooding events caused most of the county's slope stability problems. The slope failed after flooding in 2013.

Coarse rip rap covered the entire slope face, a common repair approach the county uses for slope failures. The plans specified a quarried rip rap and grass seeding. The county typically does not conduct a geotechnical investigation prior to implementing stabilization methods because maintenance teams have demonstrated success with the rip-rap cover method.

Figure 24: Houston County site slope

Soil samples were collected on the slope above the roadway, shown in Figure 25, which was original slope material. Soil was brown to dark brown silty sand. No DCP testing was conducted at this site. Surficial geology of the region was weathered material on Cambrian-Ordovician bedrock (Ojakangas, 2009; Hobbs & Goebel, 1982). The immediate proximity of the toe of the slope to a stream indicated groundwater concerns.

Figure 25: CSAH 19 (Houston County site) with steep uphill and downhill embankments

4.1.5. Koochiching County Site

The Koochiching County site was located on County Road 8, approximately ten miles southeast of Littlefork, MN. A highway maintenance supervisor highlighted three sites in the region. The first site, north of Littlefork, was a roadway exhibiting gradual creep failure, with a stream near the toe of the slope. The second site was on the Littlefork River where a culvert failure caused significant erosion and slope failure on the river bank.

The field investigation was performed at the third site (County Road 8), shown in Figure 26. Like several other slope failures in the region, slope stability issues were first noted 15 to 20 years earlier; maintenance teams repaired the pavement, but did not consider slope stabilization. The site is a good example of seasonal recurring slope issues. The slope featured multiple small failures, exhibiting visible creep failure and evidence of pavement repair.

Figure 26: Koochiching County site with visible soil creep

Soil encountered was dark brown clay with some sand and trace gravel. S_u values were approximately 1 to 1.5 tsf. Correlated N_{60} values were approximately 5 blows per foot. The Littlefork River was near the toe of the slope. Ojakangas (2009) and Hobbs & Goebel (1982) described the geology of the region as lakebed of Glacial Lake Agassiz. The cohesive material present was consistent with glacial lake sediment.

4.1.6. Lac qui Parle County Site

The Lac qui Parle County site was located on CSAH 20 between the Minnesota River and Lac qui Parle Village. The county engineer was onsite to describe erosion issues observed in the area. There was minimal evidence of slope failure affecting the roadway, as Figure 27 shows, but there were some failures at the edge of fields. The main concern was erosion of back slopes between planted fields and the ditch. Several examples of erosion were noted from field runoff causing washout similar to rotational slope failure, as Figure 28 demonstrates.

Figure 27: Steep backslopes at the Lac Qui Parle County site

Soil encountered was brown fine to medium-grained sand with trace gravel. The soil had correlated N_{60} values ranging from 5 to 7 blows per foot. *In situ* testing indicated S_u values of 1.25 to 2 tsf. Ojakangas (2009) and Hobbs & Goebel (1982) identified the geology of the region as sediment from the Minnesota River, consistent with granular soil observed. The county engineer provided project documents from recent roadwork (grading in 2013 and surfacing in 2014). Landowner property boundaries and right-of-way concerns caused geometry limitations next to planted fields. Areas with naturally forested uphill cover did not show evidence of failure. Erosion was the driving force of the failure.

Figure 28: Erosion at edge of planted field causes slope failure at the Lac Qui Parle Co. site

<u>4.1.7. Marshall County Site</u>

The Marshall County site was located on 280th Street NW, approximately 7 miles northwest of Warren, MN. The county engineer identified the location in the initial survey. The site exhibited multiple small failures down the slope (Figure 29). A ditch with standing water was located near the toe of the slope. The site cover was tall grass with topsoil extending 6 to 12 inches below the surface.

Figure 29: Marshall County site slope

Soil appeared to be gray to dark gray clay with trace sand. DCP results indicated N_{60} values of approximately 3 to 4 blows per foot. The geology of the site was sediment from Glacial Lake Agassiz (Ojakangas, 2009; Hobbs & Goebel, 1982); observed soil was consistent with glacial sediment. Given the toe of the slope was near standing water, groundwater and drainage conditions were a concern. As with other sites, the embankment near a water source was failing. The presence of groundwater, frost-susceptible soil, and cold weather made this site a good example of freeze-thaw cycle effect on slope stability and creep failure. No slope stabilization methods were evident during investigation.

4.1.8. Murray County Site

The Murray County site was located on CSAH 22 near Plum Creek south of Walnut Grove, MN. Evidence of pavement repair identified the site location. The county engineer and maintenance supervisor were present at the site. The slope had been repaired in 2014 using nearby fill, placed and compacted in lifts with geosynthetic reinforcement (shown earlier in Figure 7). Figure 30 shows the reconstructed slope.

The county engineering department provided slope geometry data, and samples of both the recent fill material and original soil (from the undisturbed slope) were collected. Observed fill was brown lean clay with trace sand and the slope had grass cover. The native soil was darker brown clay or silt. The correlated N_{60} value for the fill was approximately 2 blows per foot, and the native material had a correlated N_{60} value between 3 and 4 blows per foot. Ojakangas (2009) and Hobbs & Goebel (1982) described the geology of the region as the Altamont ground moraine. Glacial till (i.e. moraines) can deposit a variety of soil types. The nearby stream indicated probable groundwater near the toe. Slope steepness, landowner and right-of-way considerations, and groundwater were concerns on this site.

Figure 30: Murray County site slope after reconstruction with visible pavement repair

4.1.9. Olmsted County Site

The Olmsted County site (Figure 31) was located on CSAH 15 at County Road 117, west of Rochester, MN. The county engineer identified the location. The site demonstrated steep backslopes. The slope exhibited clear failure marks, shown in Figure 32. The failure in the backslope did not appear to affect the roadway. The site cover was grass and some small trees. Roots were evident in hand sampling, and topsoil extended to a depth of approximately one foot.

Figure 31: Olmsted County site slope

Observed soil was light brown silty sand. Results of DCP testing increasing resistance to penetration with depth. The soil had an average N_{60} value of 2 to 3 blows per foot. Ojakangas (2009) and Hobbs & Goebel (1982) described the geology of the region as glacial drift. No streams or other groundwater indicators were present. There were no stabilization attempts at this site.

Figure 32: Rotational failure scarp at Olmsted Co. site

4.1.10. Pennington County Site

The Pennington County site was located on MN-32, approximately 1 mile south of Thief River Falls, MN. The county engineer identified the site location in the initial survey. The site exhibited significant rotational failure, as Figure 33 illustrates. The Red Lake River was near the site and standing water was present at the toe of the slope. The failure left clear scarp lines 3 to 5 feet tall. The site cover was tall grass. Topsoil extended 6 to 12 inches below the surface.

Figure 33: Pennington County site slope, clear rotational failure

The site soil was brown clay with sand. *In situ* testing indicated S_u values of 0.25 to 0.75 tsf. DCP test results indicated N_{60} values of 3 to 4 blows per foot outside the failure zone. The DCP probe advanced under self-weight in the failed portion, indicating significantly low strength. Ojakangas (2009) and Hobbs & Goebel (1982) identified the geology of the site as sediment from Glacial Lake Agassiz, consistent with the cohesive material observed. Standing water near the toe of the slope indicated groundwater and drainage concerns. No slope stabilization attempts were noted.

4.1.11. Redwood County Site

The Redwood County site was located on CSAH 11 approximately one mile south of Franklin, MN. Pavement repair was evident at the site. The county engineer noted the site location in the initial survey. The failed side of the road was covered with coarse aggregate rip rap, shown in Figure 34. The opposite side (Figure 35), although apparently steeper, did not show signs of failure or stabilization attempts. Site surface cover was thick grass and rip rap.

Figure 34: Failed (east) side of Redwood County site slope and nearby stream

Figure 35: Opposite (west) slope, no observed failure, Redwood Co.

Soil observed was dark brown clay. Average S_u values were approximately 0.5 tsf. DCP testing indicated improving penetration resistance with depth. The average N_{60} values ranged from 4 to 5 blows per foot. Geologically, the site was composed of glacial till (Ojakangas, 2009; Hobbs & Goebel, 1982). A stream was present near the toe of the slope. Rip rap cover was evidence of slope stabilization attempts. Older rip rap indicated that the site has been previously repaired.

4.1.12. St. Louis County and West Duluth Sites

The site representing St. Louis County was located on County Road 535 near Greany, MN. The site featured severe embankment failure over a culvert and displacement of concrete roadway barriers. Slope damage was due to culvert failure and erosion causing the toe of the slope to fail. A county maintenance engineer and regional maintenance superintendent were onsite to discuss the project history. Figure 36 shows the site; cover was grass, brush and small trees.

Figure 36: St. Louis County slope and displaced concrete barriers

Soil encountered was gray to light brown clay with trace sand. Ojakangas (2009) and Hobbs & Goebel (1982) identified the geology of this region as sediment from Glacial Lake Agassiz, consistent with cohesive soil observed. Groundwater was present in the culvert at the toe of the slope.

A MnDOT engineer and project adviser identified another slope failure in St. Louis County. The site was located in West Duluth on Grand Avenue. The failure is shown in Figure 37. The slope was covered with grass and the top of the slope was wooded.

Figure 37: West Duluth site slope

4.1.13. Washington County Site

The Washington County site was located on CSAH 21, approximately one mile south of Afton, MN. The assistant county engineer identified the site in the initial survey. While some pavement repair was apparent at the site, the main site identification was wire mesh and rock anchors covering the slope. The slope was steep, and large portions of exposed rock were visible from the road, as shown in Figure 38. The county engineer provided two geotechnical reports from the slope project. The main face of the slope featured wire mesh and rock bolts, as pictured in Figure 39. This site provides examples of using surface cover (i.e. wire mesh) and reinforcing structures (i.e. rock bolts) to stabilize slopes.

Areas on the side and top of the slope were covered with grass and some small trees. DCP testing was not conducted at this site. Soil samples showed brown silty sand at the top and base of the slope. Ojakangas (2009) and Hobbs & Goebel (1982) described the geology of the region as the glacial till. No groundwater was present during sampling.

Figure 38: Washington County site slope

Figure 39: Wire mesh and rock bolt cover at Washington Co. site

4.1.14. Site Investigations summary

Fourteen sites were investigated and documented in counties bordering North Dakota, South Dakota, Iowa, Wisconsin and Ontario, and in the Red River Valley near Manitoba. Table 2 summarizes the site investigations. Of the documented sites, five had primarily sandy soil, eight had primarily fine-grained soil, and there was one rock slope. Slope failure was visible at nine sites, while four sites were already stabilized. The damaging effects of groundwater were evident at most site failures, indicating that controlling water is a valuable stabilization method. One site bridged a stream with a culvert that appeared to fail and cause slope damage, and three sites showed slope stability issues in back slopes.

Table 2: Field investigation site visits

Date	County Sites Investigated
Nov. 10, 2015	St. Louis
Nov. 12-13, 2015	Carver, Redwood, Murray, Lac qui Parle
Nov. 19, 2015	Carlton
Nov. 23-24, 2015	Washington, Houston, Fillmore, Olmsted, West Duluth
Dec. 3, 2015	Koochiching
Aug. 8, 2016	Marshall, Pennington

4.2. Soil Characterization and Strength Evaluation

Soil strength characterization involved laboratory testing. The author determined *in situ* moisture content of site samples in general accordance with *ASTM D2216 – 10.* Results from testing are available in Appendix A. Moisture content testing was conducted shortly after returning from field investigations to avoid loss of moisture in storage.

Soil classification was conducted following USCS specifications. Table 3 shows the USCS classification for each soil sample. Because samples were collected with a hand auger, characterization is based on the top several feet of slope material. USCS describes high plasticity clay as "fat clay" and low plasticity material as "lean clay."

Direct shear testing provided strength properties for each site; Table 4 displays the results. Outputs from testing are φ' in degrees and c' in stress units, i.e. pounds per square foot (psf). Direct shear test data is available in Appendix C. The scope of analysis was limited to slope failures that are common, recurring issues faced by county engineering teams. Due to extreme slope geometry and severe slope failure, the author excluded the sites in St. Louis and Washington Counties from lab and modeling analysis; a geotechnical engineering consultant, rock mechanics analysis, and complete site assessment would be necessary at these sites. Soil strength properties were critical to developing slope models representative of case study sites.

Table 3: Soil Classification for each site sample

Sample	**USCS Classification**
Carlton Co.	CL - Lean clay
Carver Co.	SP - Poorly-graded sand
Fillmore Co.	ML - Silt with sand
Houston Co.	SC - Clayey sand
Koochiching Co.	CL - Sandy lean clay
Lac Qui Parle Co.	SP-SM - Poorly-graded sand with silt
Marshall Co.	MH - Elastic silt
Murray Co. - Native	ML - Sandy silt
Murray Co. - Fill	SC - Clayey sand
Olmsted Co.	CL - Sandy lean clay
Pennington Co.	ML - Silt with sand
Redwood Co.	CH - Fat clay with sand

Table 4: Direct shear test results

Sample	**φ'**	**c'**
	(degrees)	(psf)
Carlton Co.	16	1220
Carver Co.	35	200
Fillmore Co.	35	150
Houston Co.	34	300
Koochiching Co.	24	400
Lac Qui Parle Co.	35	50
Marshall Co.	18	600
Murray Co. Fill	32	390
Murray Co. Native	22	900
Olmsted Co.	34	200
Pennington Co.	17	1275
Redwood Co.	21	750

4.3. Verification of Site Investigation Results

A second visit to a case study site was conducted for re-investigation and re-testing to verify the input parameters at the Koochiching County site in October 2016, involving reconducting field testing and collecting samples for lab test verification. Testing the same site (Figure 40) provided confidence in the accuracy of results from initial site visits, soil characterization and applicability of sample data to represent a given site.

Field testing was conducted at various locations along the cross section of the site, generally corresponding to the top, middle, and toe of the slope. Hand auger samples were collected at a depth of 2.5 to 3 feet at each location. Visual comparison identified samples as the same soil type. This indicated that the same soil layer was present at the top, middle, and bottom of the slope, as illustrated in Figure 41, and that field tests and samples collected from one location on a slope are representative of the given test depth along the slope.

Figure 40: Koochiching County site a) initial visit Dec. 2015 and b) revisit Oct. 2016

The author also verified lab results. Direct shear test results of samples from the second site visit yielded the following strength properties: $\varphi' = 24°$ and $c' = 550$ psf. These are comparable to the results from the original test in Table 4. The second site visit provided reasonable certainty that soil strength properties and site characteristics from investigations and lab testing were representative of each respective slope, allowing the project to confidently proceed to modeling and analysis.

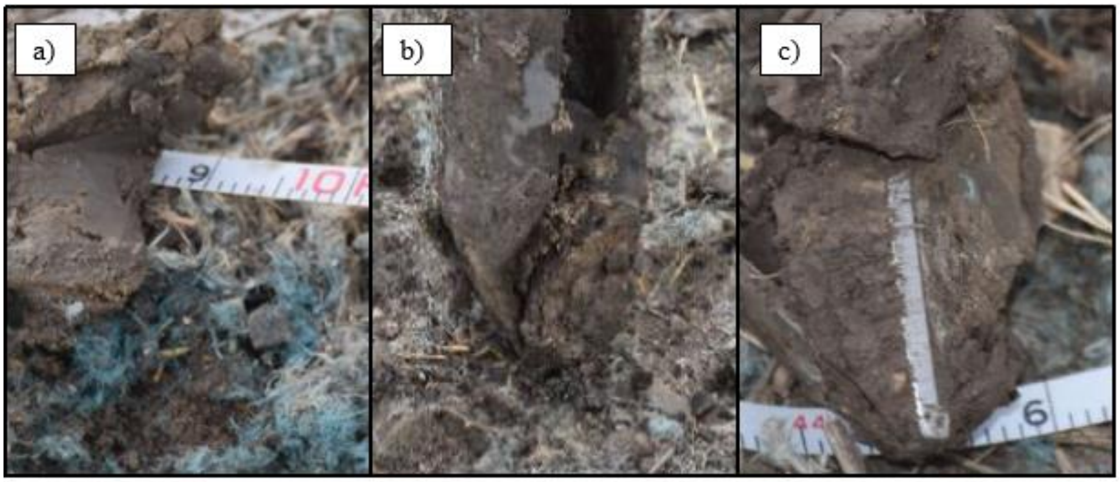


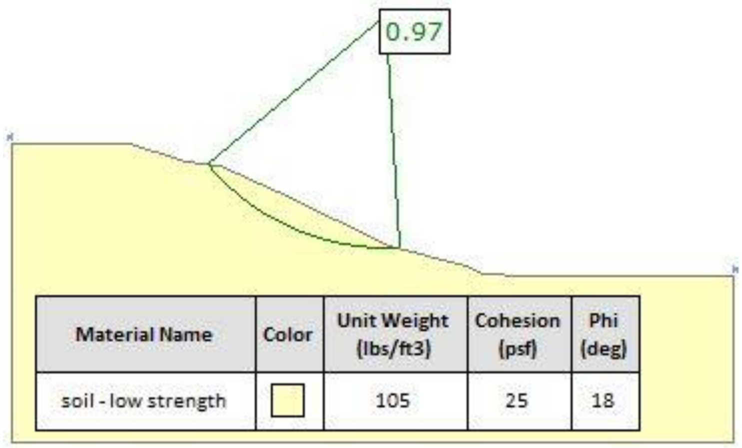
Figure 41: Samples from the a) top, b) middle, and c) toe of the same slope

4.4. Slope Stability Models

Modeling was intended to cover a wide spectrum of scenarios, allowing analysis for the best stabilization option for a given slope. Not all models necessarily represented the most practical or common methods (i.e. replacing an entire slope with fill), but their analysis provided FS outputs for comparative analysis. Modeling involves approximating some values, and experienced-based interpretation of results is ideal for LEM analysis, especially in practice. Modeling was used to conduct a comparative study of various stabilization alternatives, not necessarily to assess an actual slope's stability or approve a stabilization design.

4.4.1. Validating LEM Models

Site conditions were observed at sites with slope failure. When modeling *in situ* conditions of a given slope failure, the output FS was expected to be less than or equal to 1.0, confirming failure. LEM simulations of some failed slope situations (where FS should be less than 1.0) resulted in FS values greater than 1.0; this indicates that for some scenarios, input parameters were not representative of site conditions, and LEM modeling over-estimated the FS. To address this, the author adjusted input parameters for failed slopes, lowering the FS to confirm an observed failure. One method was placing the water table at the slope surface, decreasing the FS significantly. Poor compaction, freeze-thaw cycling, and undocumented fill also lower *in situ* soil strength. Therefore, for sites with observed failures, decreased strength values were used to represent these conditions with output FS values less than 1.0 and failure surfaces similar to conditions noted in the field. This provided a representative scenario for comparing improvements from stabilization. Figure 42 shows an example with decreased strength properties corresponding to poor compaction.



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
soil - low strength		105	25	18

Figure 42: Example failure confirmation, output matches observed slope failure

The example in Figure 42 shows the model of a site where a clear rotational slide was observed, and the model output confirms site observations. The failure is the Olmsted County site (Figure 32). Decreasing φ' and c' values caused the software to output a failure corresponding to conditions observed in the field.

4.4.2. Infinite Slope Analysis

Sites with observed creep failure were more difficult to validate, because the output from SLIDE identifies the circular plane with the lowest resistance to sliding. Therefore, models of sites exhibiting surficial creep were not representative of observed conditions. Model outputs of slopes with low values of c' exhibited shallow failure surfaces.

As mentioned earlier, the author represented some sites exhibiting creep failure with infinite slope analysis. Figure 43 shows the SLIDE output of a site with minimal cohesion; infinite slope analysis best represents the capacity of this site to resist creep failure, as indicated by the large failure radius.

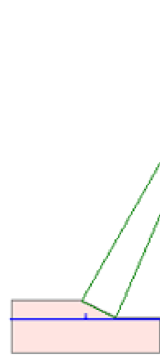


Figure 43: SLIDE output with low c', a site that is best represented by infinite slope analysis

Because LEM modeling can over-estimate stability for some conditions and the infinite slope analysis method used does not consider the benefit of soil c', the FS will be between results from LEM analysis and infinite slope analysis for some sites. The author considered results of infinite slope analysis for granular sites where the method was more appropriate than LEM modeling. For such sites, infinite slope FS was used as a baseline for comparing stabilization methods. Table 5 presents the results of infinite slope analysis for applicable sites. Only the more appropriate method was considered for each site's baseline in parametric analysis.

Table 5: Infinite Slope Analysis results for appropriate sites

Site	**FS**		**Slope Angle**	**Soil Unit Weight**	**Soil Friction Angle**
	Dry	**Saturated**	β (deg)	γ (pcf)	φ' (deg)
Carver	**2.16**	**1.08**	18	125	35
Fillmore	**2.16**	**0.93**	18	110	35
Houston	**1.32**	**0.69**	27	130	34
Koochiching	**3.17**	**1.59**	8	125	24
Lac Qui Parle	**0.78***	**0.42***	42	135	35
Murray Fill	**1.23**	**0.61**	27	125	32
Olmsted	**1.85**	**0.96**	20	130	34
* Failed due to erosion damage and surface washout					

4.4.3. Modeling Groundwater and Drainage Effects

Groundwater has a negative effect on soil strength. This is a fundamental concept in soil mechanics; as pore water pressure (u) increases, effective normal stress (σ') decreases. Because σ' governs soil strength, an increase in pore water pressure decreases soil strength. The author modeled the effect of groundwater by considering a steady-state, worst-case scenario. Assuming no drainage, the groundwater table would rise to the ground surface, as shown in Figure 44.

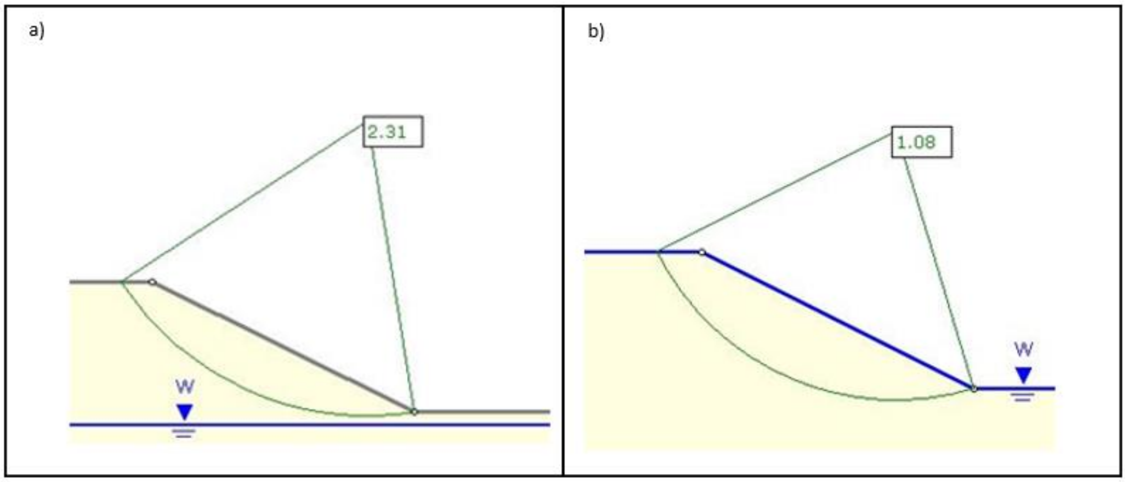


Figure 44: Groundwater effects modeled by a) *in situ* and b) worst-case conditions

Installing drainage features returned the site to hydrostatic conditions with a higher FS. The author did not consider transient groundwater analysis to represent a situation between those shown in Figure 44. This would have required a more sophisticated finite element analysis. Construction below the water table depth should involve detailed geotechnical analysis. These scenarios are outside the scope of research.

Both LEM models and infinite slope analysis were considered in studying groundwater effects. For the infinite slope analysis, noting the difference between dry FS and saturated FS (from Equations 2 and 3) quantified the effect of groundwater. Table 6 shows modeling results of sites with both *in situ* and worst-case groundwater conditions. Sites with higher φ' values were more sensitive to the presence of groundwater. Sites with a higher percentage FS change corresponded to higher values of φ', indicating that sandy sites experience a higher drop in FS than cohesive materials.

Table 6: Output results from modeling *in situ* and worst-case water table depths

Site	Analysis Method	FS		% change	φ' (deg)
		***in situ* Water Table**	**Worst Case Water Table**		
Carlton Co.	LEM	9.63*	8.30*	13.8	16
Carver Co.	Infinite Slope	2.16	1.08	50.0	35
Fillmore Co.	Infinite Slope	2.16	0.93	56.9	35
Houston Co.	Infinite Slope	1.32	0.69	47.7	34
Koochiching Co.	Infinite Slope	3.17	1.59	49.8	24
Lac Qui Parle Co.	Infinite Slope	0.78	0.42	46.2	35
Marshall Co.	LEM	4.21*	3.76*	10.7	18
Murray Co. Fill	Infinite Slope	1.23	0.61	50.4	32
Murray Co. Native	LEM	2.21	1.57	29.1	22
Olmsted Co.	Infinite Slope	1.85	0.96	48.1	34
Pennington Co.	LEM	10.95*	8.95*	18.2	17
Redwood Co.	LEM	6.25~	4.32~	30.8	25

* Slope did not fail, or model is not of failed portion of slope

~ Slope had been repaired

4.4.4. Modeling Surface Cover

Surficial failure and creep was observed at several sites. Stabilizing the uppermost soil layer minimizes the effect of soil creep and limits pavement damage. For modeling, a one foot-thick-layer of fill material covering *in situ* soil was considered, as shown in Figure 45. Models were executed with properties for coarse gravel and cobble rip rap. Representative strength properties for common borrow rock were (Attia et al., 2009): φ' = 45°, c' = 5 psf and γ = 120 pcf.

Surface cover does not typically increase the FS; in the case of rip rap cover, the method can increase weight and driving forces, which decreases the FS. Increasing strength properties of the cover material requires compaction, which can be difficult to achieve on the surface of a failing slope. The significant benefit is erosion protection, which cannot be easily quantified with LEM modeling.

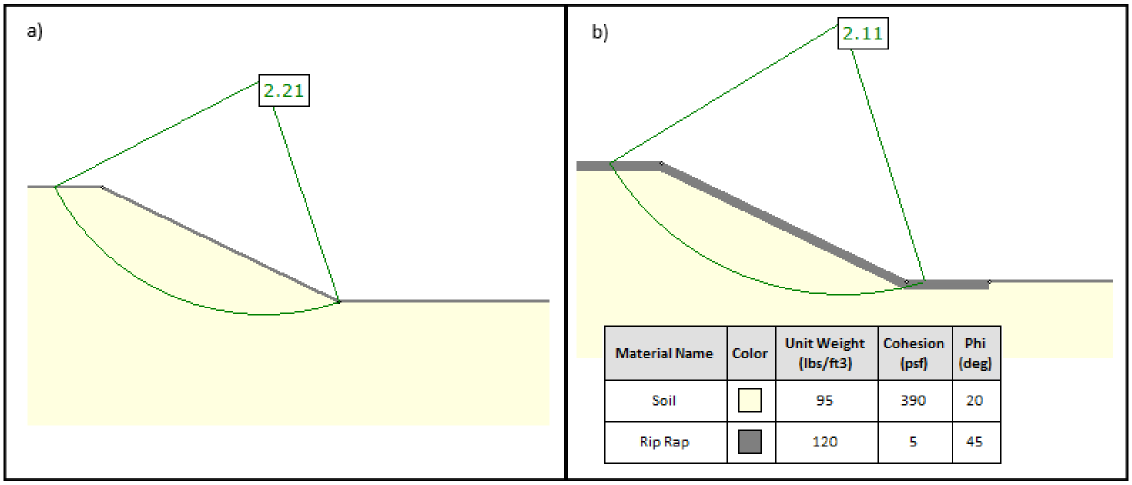


Figure 45: Example model a) with no surface cover, and b) with rip rap surface cover

Vegetative cover is another stabilization method. Operstein & Frydman (2000) concluded that vegetation increases shear strength, and that roots improve strength by increasing overall c'. Observations from modeling indicate that c' governs the depth of the failure surface; a soil with higher c' will have a deeper circular failure. Because plant roots have a proven impact on c', using vegetative cover on slopes is recommended to increase surficial stability.

<u>4.4.5. Modeling Buttressing</u>

The advantage of buttressing is that no slope reconstruction is necessary. Maintenance teams can simply place fill material against the slope toe. The author considered the same common borrow rip rap from surface cover as buttress material, with the same material properties. LEM tests were performed on baseline slopes with buttresses extending various heights above the toe of the slope. Figure 46 provides an example.

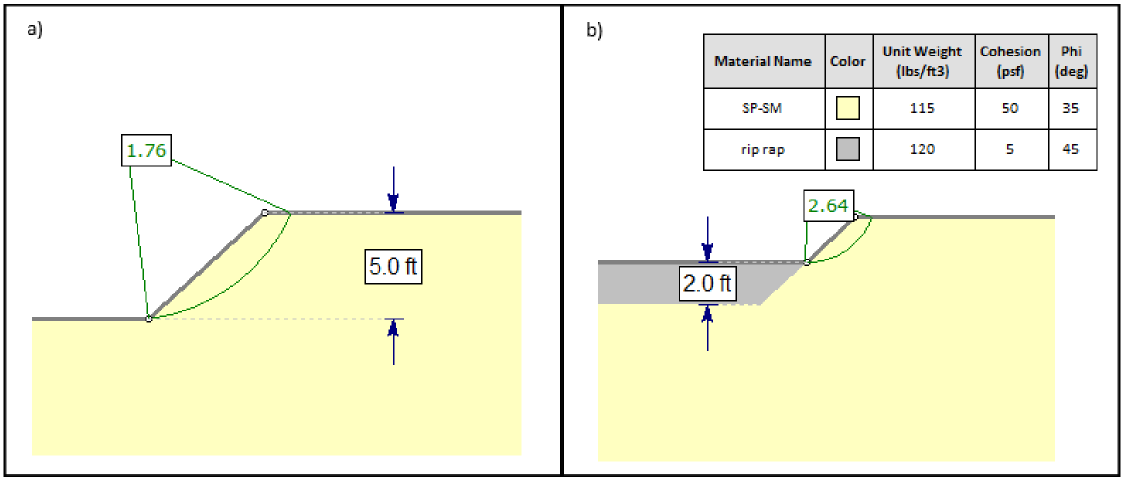


Figure 46: Example model a) without and b) with buttressing

As the example illustrates, an aggregate buttress affects the failure surface. The buttress material has higher strength properties, so failure occurs in the soil above the buttress, and the FS rises, indicating an increase in global stability. The most benefit was noted in small slopes.

4.4.6. Modeling Regrading

Decreasing slope angles can reduce driving forces. If there is room in the right-of-way, shallow slopes (i.e. lower inclination angle) will be more stable. Regrading, even when not changing the overall slope angle, can increase global FS. The standard practice of re-compacting soil in layers and finishing the slope to a specified grade generally adds stability; fundamentally, soil with higher density will exhibit higher strength values. In some cases, geometric inconsistencies can cause local instability. Regrading is a way of 'smoothing out' irregularities. Simply regrading a failed slope to the same overall angle noticeably improved the overall FS at the Olmsted County failure site, as Figure 47 shows.

While the Olmsted County site is an extreme example, regrading typically improves stability. For every method that required excavation, modeling reflected the benefit of regrading, proper construction, and re-compacting. Similar to new construction, for reconstruction and excavation projects, the slope is finished at a specified grade. This 'straight-line' slope face avoids geometric inconsistencies encountered *in situ* and exhibits a higher FS, and compacted soil has higher soil strength properties.

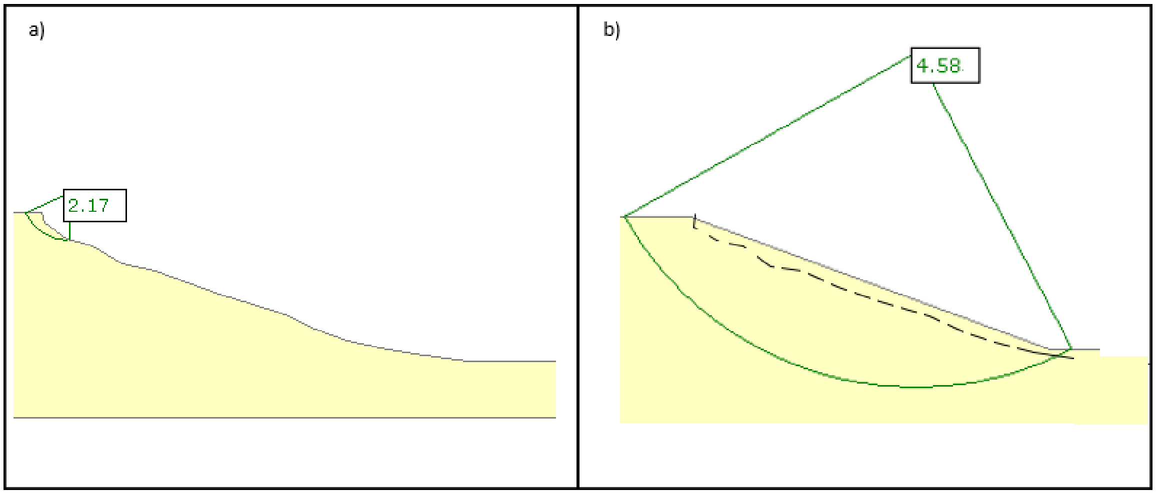


Figure 47: Model of regrading and 'filling in' geometric inconsistencies; a) original and b) regraded

4.4.7. Modeling Soil Replacement

Clean, free-draining sand is ideal for roadway embankments. Replacing *in situ* material with more suitable fill is a stabilization option, although typically more expensive than other methods. Direct shear testing was conducted on coarse, clean sand, and replacement material with the following properties was considered: $\varphi' = 35°$, $c' = 100$ psf and $\gamma = 120$ pcf. The remove-and-replace method requires excavation, but likely not a specialty contractor. The author modeled three scenarios for soil replacement: replacing the top 5 feet of a slope with sand fill, replacing the top 10 feet, and replacing the entire slope. These extreme scenarios, although expensive and rarely practical, provided a relative understanding of the effectiveness of the method.

An important benefit of using sand fill to stabilize slopes is improving drainage properties. The author considered worst-case drainage and adequate *in situ* drainage conditions for each site. The worst-case drainage scenarios were modeled by assuming the native material had poor or no drainage capability and placing the water table immediately at the bottom of the fill layer. The adequate *in situ* drainage situations were modeled with the water table at its baseline depth, representing a native material with good drainage properties, or poor drainage and the use of additional drainage features. The example below shows the two scenarios: replacing the top 10 feet of the same slope with sand. Figure 48 shows the worst-case drainage scenario, and Figure 49 shows the model assuming adequate drainage for the same site.

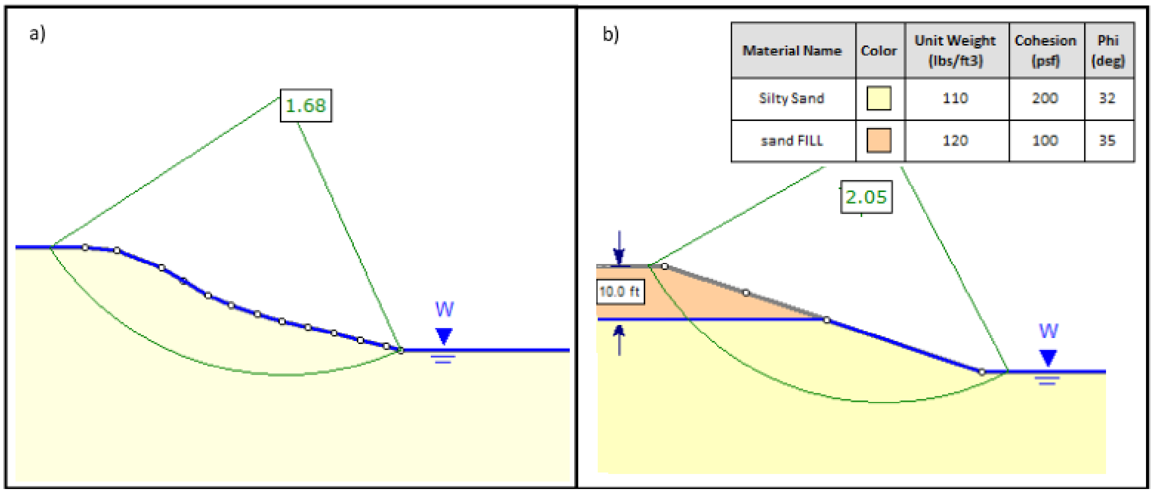


Figure 48: Example model of replacement with sand, worst-case drainage scenario; a) before and b) after replacement

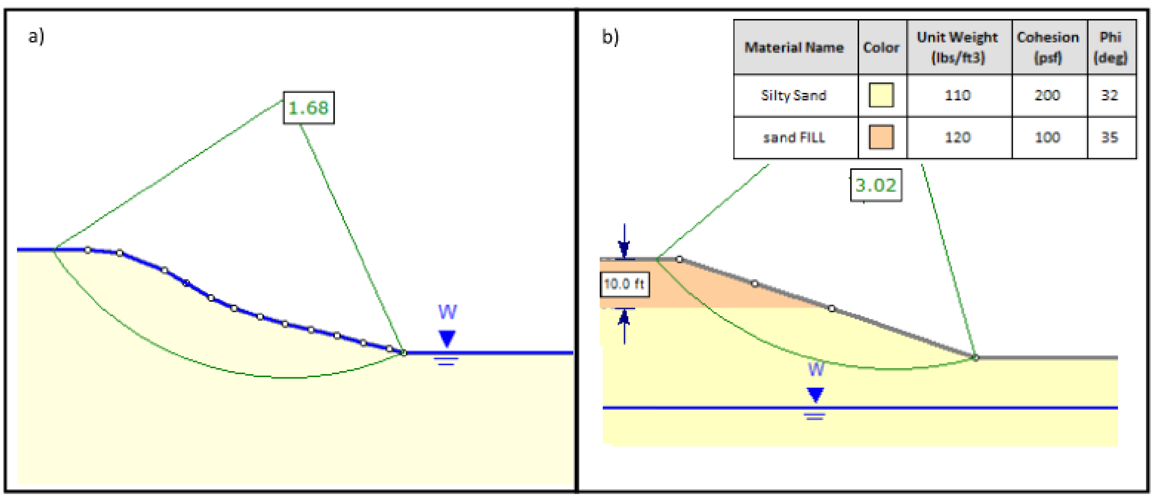


Figure 49: Example model of replacement with sand, adequate drainage scenario; a) before and b) after replacement and drainage feature installation

The worst-case scenario in Figure 48 simulated poor drainage conditions, and only replacing the top portion of the slope with free-draining fill. The adequate drainage scenario in Figure 49 represents choosing to install drainage features; this will have a higher cost, but appears to be more effective if *in situ* drainage is poor. Feasibility for the remove-and-replace method typically depends on availability of fill material. Maintenance teams should cover sand after placement to prevent erosion.

Another replacement fill option is expanded polystyrene (EPS) foam blocks, commonly called geofoam. The blocks can be easily placed in an excavation, and dramatically decrease the weight of the slope. The author modeled the effects of EPS geofoam by treating the blocks as a new material layer. Direct shear testing on EPS geofoam (Padade & Mandal, 2014) led to the following strength properties: $\varphi' = 6°$, $c' = 1250$ psf and $\gamma = 2$ pcf. A scenario replacing the entire slope depth with EPS geofoam was modeled to note the maximum difference in FS. Figure 50 shows an example of soil replacement with geofoam.

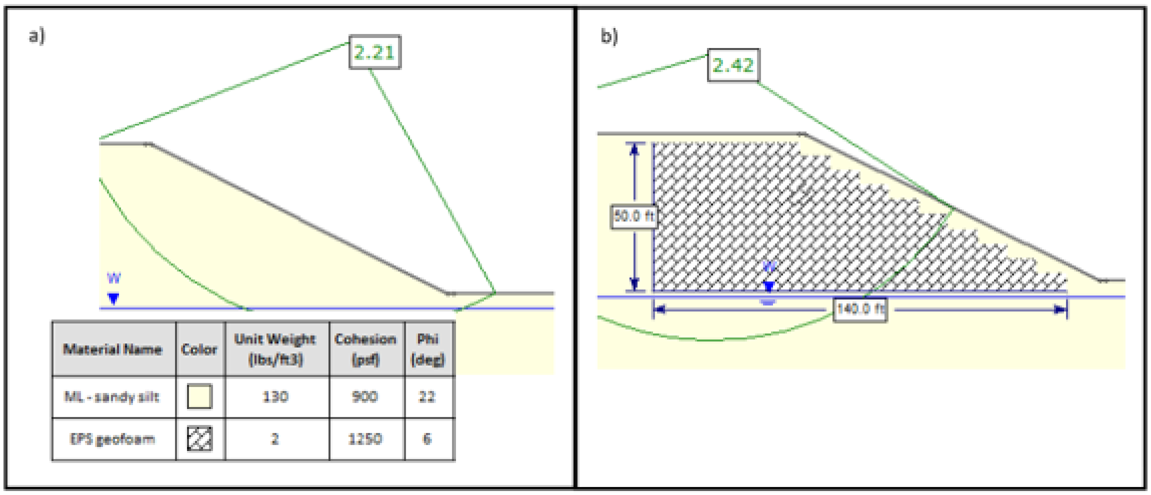


Figure 50: Example model simulating a) before and b) after implementing EPS geofoam

Little benefit from geofoam fill was noted on most slopes. The greatest benefit was on the largest slopes, where excavation would cost the most. This method requires extra consideration in areas with environmental sensitivity concerns. Due to material buoyancy, geofoam should not be used in flood plains. The use of EPS geofoam also requires excavation, and is likely a better consideration during the design of new slopes. Geofoam blocks have common applicability in other areas of infrastructure construction, such as bridge abutments and culvert fill.

4.4.8. Geosynthetic and Structural Reinforcement

Geosynthetic reinforcement can increase slope stability and decrease the effect of erosion. Installing geosynthetic material requires excavation. The author considered several scenarios involving geogrid and found a variety of application methods and material properties. Strength properties vary for each manufacturer, and a geotechnical design process is necessary for each installation. Geogrid is an option for increasing shear strength, but not an easy stabilization method for recurring slope maintenance. Therefore, geosynthetic material is outside the scope of this research.

The literature review identified several stabilization methods that increase slope stability, including retaining walls, soil nailing, and MSE walls. These structural reinforcement techniques are effective stabilization approaches when applied correctly. In some cases it is appropriate to conduct a geotechnical investigation and hire a specialty contractor to consider such stabilization procedures. These solutions, however, are beyond the scope of this research.

4.5. Development of Deliverable

Modeling results were analyzed to develop recommendations for the project deliverable. The guide starts with common slope failure types and site conditions. Users will find the set of conditions that most closely match a given slope stabilization site, and the guide will recommend stabilization approaches.

The guide includes site characteristics that future users are most likely to encounter. The tool uses distinctions in three site conditions to characterize any given slope project:

- Failure type (i.e. surficial soil creep or rotational slide)
- Soil type (i.e. cohesive or granular soil)
- Drainage condition (i.e. presence or absence of groundwater indication)

These distinctions most clearly categorize the site conditions observed during site investigations. The tool provides examples of the type ‘if you see *this*, consider …’ and suggest stabilization approaches for each situation based on modeling conclusions. Users can follow the guide like a flowchart to arrive at the combination of site conditions that most closely matches the observed slope.

4.5.1. Site Distinctions Based on Failure Type

Failure type has significant impact on which stabilization methods are appropriate. The distinction for failure type is surficial soil creep vs. rotational failure; Figure 16 showed examples. If users observe circular rotational failure, excavation and slope reconstruction will likely be necessary, so maintenance engineers can consider more involved stabilization methods. Creep failure often indicates the need for surface stabilization.

4.5.2. Site Distinctions Based on Soil Type

The broadest distinction in soil type is cohesive vs. granular (i.e. clay vs. sand). Visual inspection may distinguish between the two types, but sometimes laboratory testing is necessary. The author followed lab techniques to determine USCS classification for soil type. Soil strength parameters, especially c’, control the depth of the failure surface. Sand typically has higher values of φ’ and lower values of c’. Slopes made of cohesive material will have more drainage concerns, and are usually more susceptible to seasonal frost heave. Slopes made of exposed sand typically have potential for surface erosion.

<u>4.5.3. Site Distinctions Based on Drainage Concerns</u>

Streams and standing water can indicate that the steady-state water table is near the ground surface. Undesirable drainage conditions caused many failures observed in site visits. In modeling, proper drainage was the most beneficial stabilization method. A site has poor drainage if groundwater lowers soil strength and leads to failure. Cohesive soils typically have poor drainage properties. The author modeled the effect of groundwater by assessing the FS with *in situ* water table conditions, and worst-case water table conditions, as shown earlier in Figure 44. Table 6 showed results from modeling both *in situ* and worst-case groundwater conditions.

4.6. Final Deliverable Layout and Scenario Descriptions

Combining the three site condition distinctions results in eight possible scenarios. Table 7 shows the end results of following the tool.

Table 7: Overview of scenarios outlining the final deliverable

Name	Failure Type	Soil Type	Groundwater Concerns?
Scenario #1	Rotational Slide	Cohesive	Yes
Scenario #2	Rotational Slide	Cohesive	No
Scenario #3	Rotational Slide	Granular	Yes
Scenario #4	Rotational Slide	Granular	No
Scenario #5	Surficial Creep	Cohesive	Yes
Scenario #6	Surficial Creep	Cohesive	No
Scenario #7	Surficial Creep	Granular	Yes
Scenario #8	Surficial Creep	Granular	No

<u>4.6.1: Scenario #1, rotational failure, cohesive soil, drainage concerns</u>

After a rotational failure at least part of the slope will require reconstruction, so excavation is necessary. If strength properties of the *in situ* material are unknown, the author recommends soil testing. If soil has poor strength properties, regrading with engineered sand fill is the best option. After excavation, the new slope surface should have ground cover. When groundwater concerns are present, installing drainage features is recommended. Site investigations included several examples of this combination of site conditions (Figure 51).

Figure 51: Example of Scenario #1 in Pennington Co., MN

At the Pennington County site, a small stream is located at the toe of the slope and a clear rotational failure is visible. To repair this slope, reconstruction and regrading will be necessary. Maintenance teams should consider either remove-and-replace or regrading with *in situ* soil, adding drainage features, and vegetative cover.

Figure 52 shows a modeling example of the recommended stabilization methods. The model represents a rotational failure site with the remove and replace method and additional drainage features.

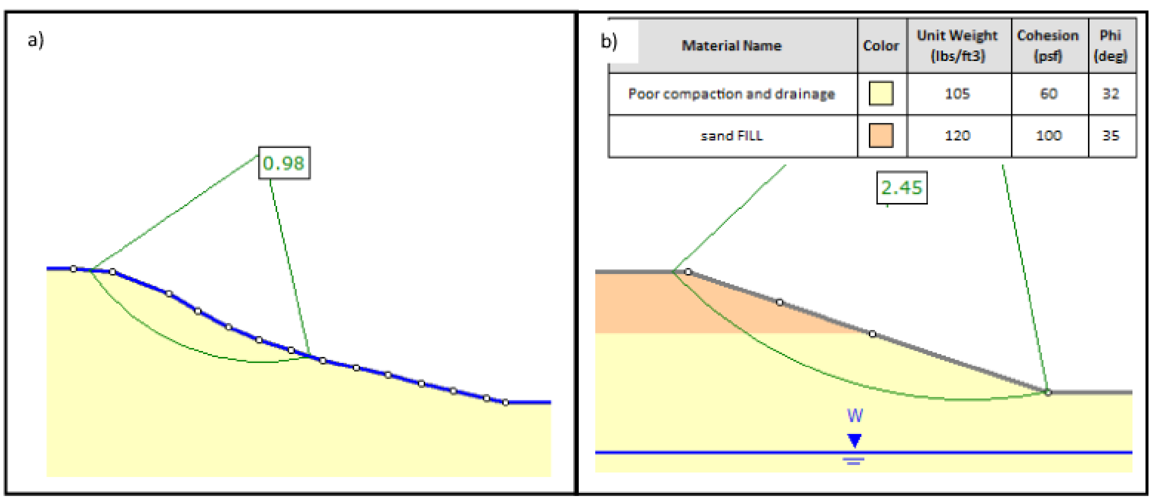


Figure 52: Model representing Scenario #1 a) with observed failure characteristics, and b) after implementing stabilization methods

Figure 52a shows the validated site conditions from the observed failure. The program outputs the observed rotational failure. Drainage features lower the water table; soil replacement and regrading add stability.

4.6.2. Scenario #2: rotational failure, cohesive soil, no drainage concerns

In this situation, the only difference from Scenario #1 is adequate drainage with no indication of groundwater concerns. A lack of obvious groundwater indicator (i.e. a stream or pond) does not necessarily indicate a lack of drainage or groundwater concerns. The author recommends installing a standpipe or other simple groundwater monitoring system before distinguishing between Scenario #2 and Scenario #1. If groundwater is not a concern, the maintenance team can disregard drainage features. Reconstructing the slope is still necessary. Olmsted County is an example of Scenario #2 conditions; Figure 53 shows the failure.

Figure 53: Example of Scenario #2 in Olmsted Co., MN

Recalling soil strength fundamentals, poor compaction causes low soil strength. Cohesive materials are more susceptible to frost heave than granular soils, and freeze-thaw fatigue may also cause loss in strength. For repair, maintenance teams should consider either remove-and-replace or regrading and compacting with *in situ* soil. Assessing the native soil's strength properties is recommended. Repair should include adequate cover (i.e. local vegetation). Figure 54 shows the SLIDE model simulating Scenario #2 and an appropriate repair approach.

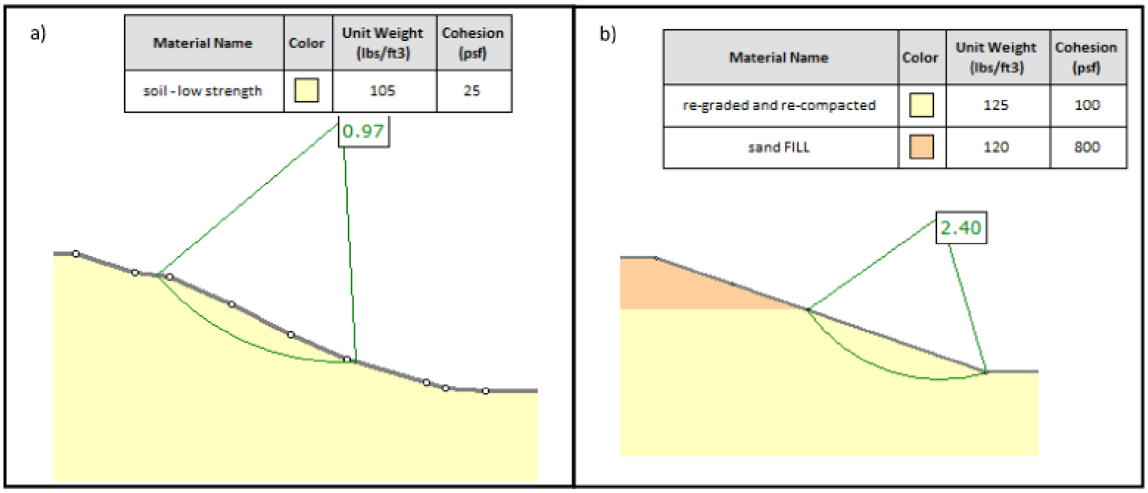


Figure 54: Model representing Scenario #2 a) with observed failure characteristics, and b) after implementing recommended stabilization methods

The demonstrated stabilization method is a partial soil replacement; the model shows a stability analysis of the slope regraded, and the top 10 feet replaced with sand fill. Replacing the failed material with fill of adequate strength properties improves the FS. The author simulated re-compacting material with the partial restoration of *in situ* strength properties. Regrading and re-compacting, when properly executed, increases soil φ’ and c’, leading to the increased FS.

4.6.3. Scenario #3: rotational failure, granular soil, drainage concerns

The next distinction is granular soil. Surface cover is important for slopes with granular soil due to erosion. Surface erosion can cause geometric inconsistencies that impact slope FS, as Figure 47 demonstrated. No observed sites matched Scenario #3 in site investigations. As with other rotational failures, excavation and reconstruction are necessary on such sites. Because groundwater is a concern, the author recommends drainage features to remove groundwater and increase shearing resistance. Maintenance teams should consider regrading, or if necessary, replacement with engineered fill. Cover options that protect the slope from erosion, such as vegetation, are recommended.

4.6.4. Scenario #4: rotational failure, granular soil, no drainage concerns

The distinction between Scenario #3 and Scenario #4 is the lack of groundwater concerns. The site in Lac Qui Parle County was an example where field runoff caused damage at the surface, leading to instability and failure, as shown in Figure 55. The observed failure appeared to be a washout from surface water.

Figure 55: Example of Scenario #4 in Lac Qui Parle Co., MN

Although the example in Figure 55 was caused by surface washout, the stabilization and repair are the same as other rotational failures. While this example does not appear to impact the roadway, reconstruction is necessary for slopes that do. As groundwater is not the primary reason for failure, users must identify and mitigate the main source of strength loss. If erosion is the primary driving force, a more rigorous surface cover (i.e. rip rap or shotcrete) is the recommendation. Slope angle may also be a concern, as in the Lac Qui Parle Co. site. If possible, decreasing the overall slope grade will increase stability. The author recommends regrading and compacting with *in situ* material, and also suggests extra consideration of adequate ground cover to protect the slope from erosion damage.

4.6.5. Scenario #5: surficial creep, cohesive soil, drainage concerns

Sites exhibiting creep failure can require recurring slope maintenance. A site will be more likely to have drainage concerns if cohesive material is present. In Scenario #5, groundwater and drainage concerns lead to creep failure. Figure 56 shows an example of this situation.

Figure 56: Example of Scenario #5 in Koochiching Co., MN

The observed failure in Figure 56 had a nearby indication of groundwater and exhibited clear soil creep. The county representative mentioned that slopes exhibited new failures each spring. With groundwater present, and *in situ* material being frost-susceptible cohesive soil, seasonal frost heave is a possible cause of soil movement. Drainage features are the main recommendation for slope stabilization. If creep is at the top of the slope, maintenance crews can also consider replacing the top portion of the slope with free-draining fill. This option would require excavation. If the failure is near the bottom of the slope, a buttress can be an effective stabilization method, depending on material availability.

4.6.6. Scenario #6: surficial creep, cohesive soil, no drainage concerns

At sites where groundwater was not a concern, soil creep was more commonly noted than rotational failure. The author recommends replacing the failed portion of the slope for increasing sliding resistance. Figure 57 shows an example of Scenario #6, where creep appeared at the top of a slope.

Figure 57: Example of Scenario #6 in Murray Co., MN

The Murray County Site is a good example of how soil creep at the top of a slope can lead to pavement damage. In the absence of groundwater, poor compaction decreases the soil's shear strength. The author recommends ensuring proper compaction in replacement or *in situ* material to stop soil creep.

4.6.7. Scenario #7: surficial creep, granular soil, drainage concerns

Adequate ground cover is essential to prevent erosion in slopes with sand. If proper ground cover is present, the slope's failure behavior can be similar to Scenario #5. The Carver County site is an example of Scenario #7 (Figure 58).

Figure 58: Example of Scenario #7 in Carver Co., MN

The bent guardrail is evidence of soil creep. This example does not appear to be severely impacting the roadway. Proper drainage can remove groundwater from the area, increasing resistance to soil creep. Crews should install drainage features and replace failed soil with properly-compacted fill, or re-compact *in situ* material.

4.6.8. Scenario #8: surficial creep, granular soil, no drainage concerns

With no groundwater to lower soil strength and cause global failure, surficial erosion is a concern. Scenario #8 describes more of a surface washout; this failure type can undermine roadways and cause pavement damage. Erosion damage and soil creep have similar movement type and stabilization methods. No examples of Scenario #8 were observed in field investigations. Ensuring adequate ground cover is important when repairing surficial damage in slopes with granular fill. Damage at the top of the slope is best repaired by regrading. Maintenance teams can consider using a buttress at sites with damage in the lower part of the slope.

4.7. Slope Stabilization Guide for Local Government Engineers

End users are expected to compare any given slope stabilization site to the scenarios in the guide layout. The input for analysis came from case studies that local government engineers identified to provide representative examples of slopes in Minnesota. The guide contains a flowchart for users to determine which scenario to study. This research developed scenarios based on analysis of modeling results. The parametric study led to the recommendations in the deliverable, which is available in Appendix D.

5. DIMENSIONLESS SLOPE STABILITY CHARTS

Dimensionless slope stability charts can approximate a slope factor of safety and critical failure surface given soil strength parameters, inclination angle, and height. The author produced charts to aid in assessing the stability of roadway embankments. The stability charts were developed to help local government engineers in preliminary design and stabilization method selection.

5.1. Overview of Slope Stability Charts

Many researchers have developed and published charts since the 1940's (see, for example, Abramson et al., 2002). Stability charts are alternative solutions to computer modeling, particularly for slopes of simple geometry, because it takes less time to use them to determine slope factor of safety. With stability charts, users can perform stability analyses without Limit Equilibrium Method (LEM) software, rule out a stabilization method without performing detailed analysis, or determine optimum slope geometries during preliminary design. The typical chart layout is a plot on which users compute a combination of input parameters on one axis and use a curve to find the corresponding point on the other axis to determine the output stability parameter.

5.2. Chart Development

State-of-practice LEM modeling with Rocscience SLIDE v7 helped generate new charts specific to roadway embankments. The author used case study inputs to determine typical ranges of parameters for slope stability analysis including soil strength properties, material density, and slope geometry. Representative input parameters were used to develop a chart relevant to typical roadway construction and repair projects. Figure 59 shows an existing dimensionless chart (from Bell, 1966) that plots a function of FS on the vertical axis, and a combination of effective friction angle (φ'), effective cohesion (c'), soil unit weight (γ), and slope height (H) on the horizontal axis. There is a separate curve for each slope inclination angle (β), and interpolation between curves makes the chart usable for any combination of input values.

In Figure 59, the horizontal value (N^*) is a dimensionless combination of input values. The vertical value (F^*) is the ratio of factor of safety and tangent of φ'. Bell (1966) shows the formulas for F^* and N^* in the framed insert on the chart. The chart was developed using the Ordinary Method of Slices, a formulation that has rarely been used since the development of the Bishop Method of Slices, which is the standard state-of-practice analysis method. The Bishop Method involves more rigorous slice boundary equilibrium calculations, providing a more accurate estimate of overall stability. The chart developed by Bell (1966) does not consider pore water pressure (u). Since φ' is a necessary input, F* values directly relate to FS.

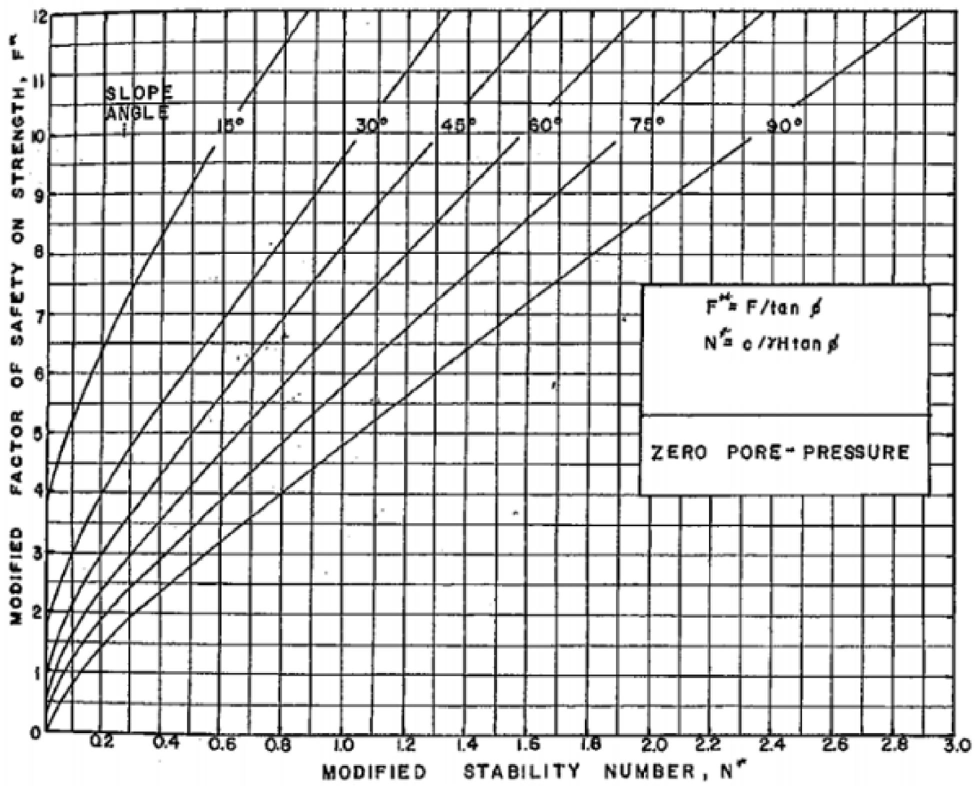


Figure 59: Example dimensionless slope stability chart (from Bell, 1966)

The author refined the range of inputs using representative values of field conditions for roadway embankments. Table 8 shows the representative ranges for input parameters. Charts were generated using SLIDE Version 7 with the Bishop Method of Slices and Auto-Refine Circular Search Method (program default parameters).

Table 8: Representative values for input parameters in slope stability development

Height, H	1 to 25 m	~ 3 to 80 ft.
Effective Cohesion, c'	0 to 60 kPa	~ 0 to 1250 psf
Unit Weight, γ	12.5 to 23 kN/m^3	~ 80 to 147 pcf
Friction Angle, φ'	15 to 45º	-
Slope Inclination Angle, β	15 to 60º	-

For a given slope inclination angle, Equation 4 describes the horizontal axis values (X) on the developed curve, and Equation 5 describes the vertical axis values (Y). These equations are input values, combinations of known or assumed parameters. Instead of entering known inputs into a modeling program, chart users calculate the axis values with the following equations, allowing utilization of the chart to obtain an output value.

(Eqn. 4) $$X = \frac{\gamma\ H \tan(\varphi')}{c'}$$ (Eqn. 5) $$Y = \frac{FS}{\tan(\varphi')}$$

A combination of programs helped develop the roadway-specific slope stability charts. For each inclination angle, a series of Microsoft Visual Basic Script (VBscript) routines generated one hundred random combinations (3 models per combination = 300 SLIDE v7 models) of the input parameters within the specified ranges. The VBscript routines allowed automated batch computation to open SLIDE, populate a model with the first random combination of input variables, and run the model. The VBscript routines repeated this process for all remaining random combinations of input parameters, and exported the results to a text (ASCII) file. Importing the results file to PTC Mathcad allowed the author to view and analyze the modeling results. The output factor of safety provided vertical axis values for each point, according to Equation 5. Each deterministic stability output corresponded to a combination of input parameters, providing the horizontal value for that combination (Equation 4). Each random case provided one point on the plot; repeating the process populated the plot with points to produce a curve. Following this process for β values equal to 15º, 30º, 45º, and 60º covered the representative range of inclination angles. This produced the factor of safety stability chart, available in Appendix E (Figure E1).

Output data was also used to analyze geometric properties of the failure surface corresponding to each set of input variables. The author produced similar charts, plotting the same horizontal axis value (X – the combination of input parameters in Equation 4) against failure surface radius (R) normalized by slope height (see Figure 60). The author developed two more charts involving the distance from the toe of the slope to the center of the output circular failure surface. The horizontal distance (x_c) and vertical distance (y_c), both normalized by slope height, were respectively plotted against X (Equation 4) – see Figure 60. The additional three charts allow users to predict failure surface depth and location in addition to overall FS. These dimensionless charts are presented in Appendix E. Presently, these types of prediction tools do not exist for typical roadway embankment design and analysis.

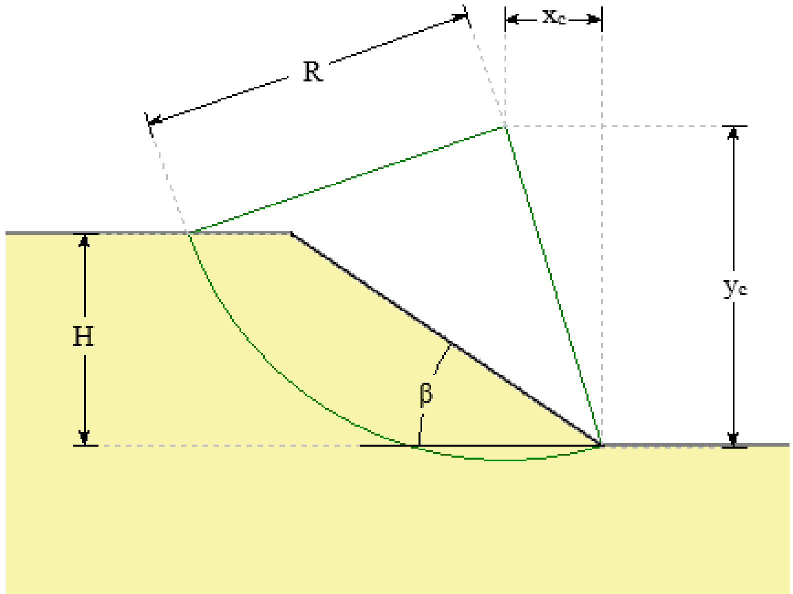


Figure 60: Geometric parameters of circular slope failure

The author used Mathcad to develop the stability output curves and regression functions to fit third order inverse polynomial equations to the curves. Regression analysis provided a general approximate equation for Y (Equation 5) in terms of X (Equation 4) and β. Because of the intrinsic errors associated with standard regression analysis, the curve fitting and general equation are not as precise as LEM modeling. Figure 61 shows the factor of safety chart and the third order inverse polynomial approximation formula for each inclination angle curve.

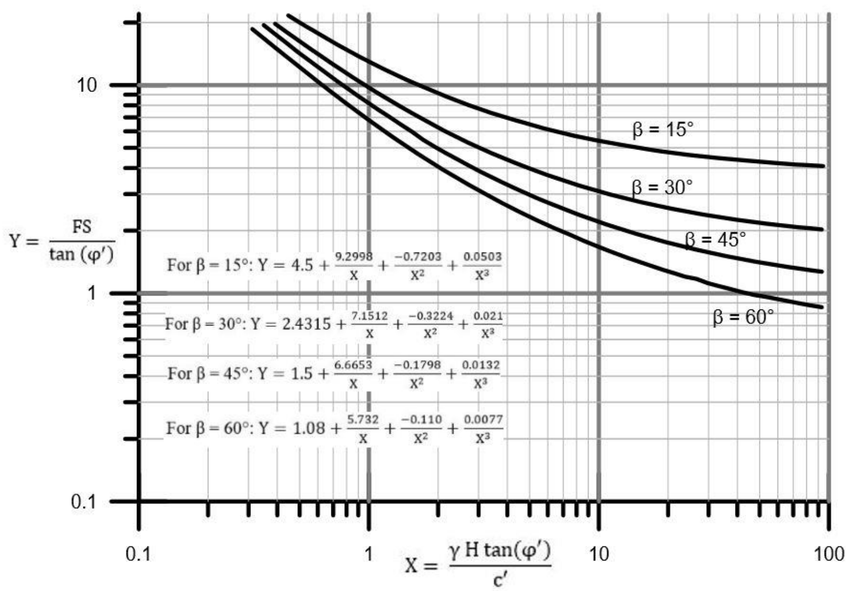


Figure 61: Factor of safety slope stability chart with individual curve equations

Appendix E shows the curve fitting and regression analysis, along with the full-size FS chart. Equation 6 provides the general approximate equation for FS.

$$\frac{FS}{\tan(\varphi')} = Y = 75.548(\beta)^{-1.03} + \frac{-0.075(\beta) + 10.01}{X} + \frac{0.041(\beta) - 1.237}{X^2} + \frac{1.83(\beta)^{-1.32}}{X^3} \quad \text{(Eqn. 6)}$$

5.3. Example of Chart Use

The following example illustrates the applicability and accuracy of the charts, the approximation and general equation. Consider a slope with the following input parameters: H = 7 m, c' = 26 kPa, γ = 17.5 kN/m^3, φ' = 27º, β = 45º. From Equation 4, the calculation for the horizontal value yields: X = 2.4. From the chart, users can find the vertical value Y = 4.4, and calculate a FS of approximately 2.24, as Figure 62 shows.

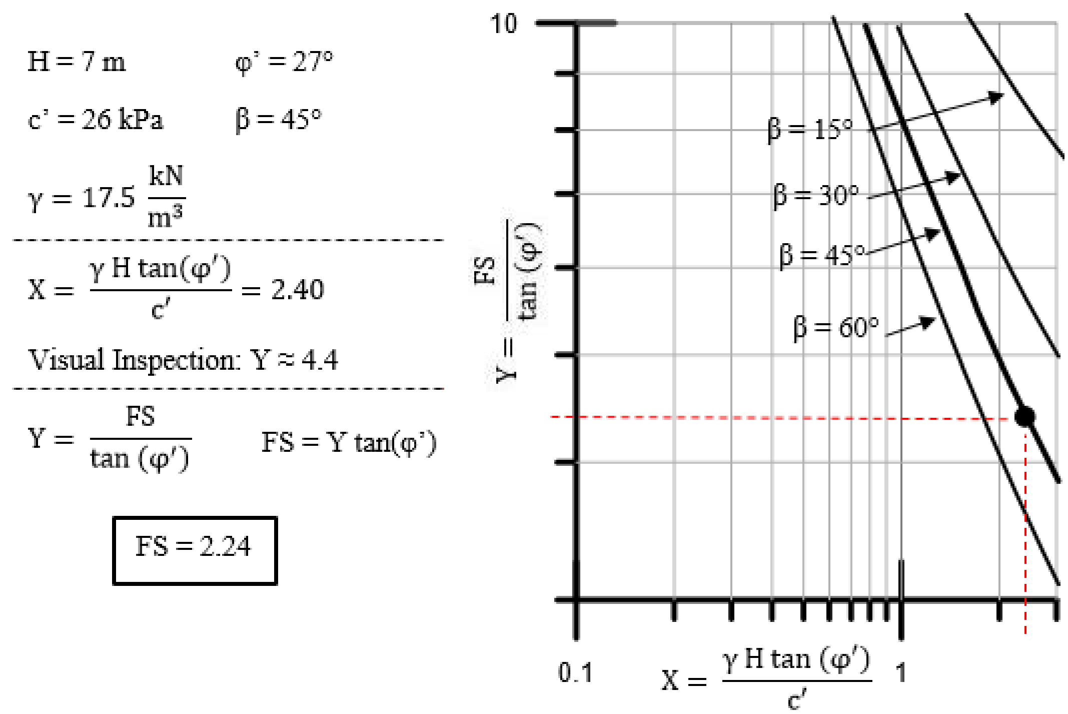


Figure 62: Example stability analysis of using the slope stability chart

To verify the answer, the author analyzed the same model with SLIDE (Figure 63). The output FS is 2.25, which is basically identical to the result from the stability chart analysis. The stability chart appears to be a reliable estimate of slope FS.

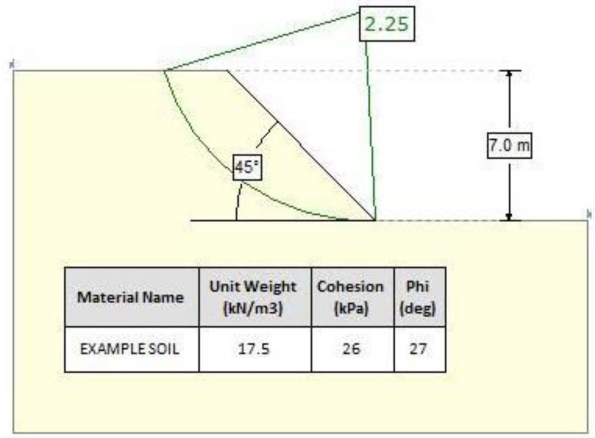
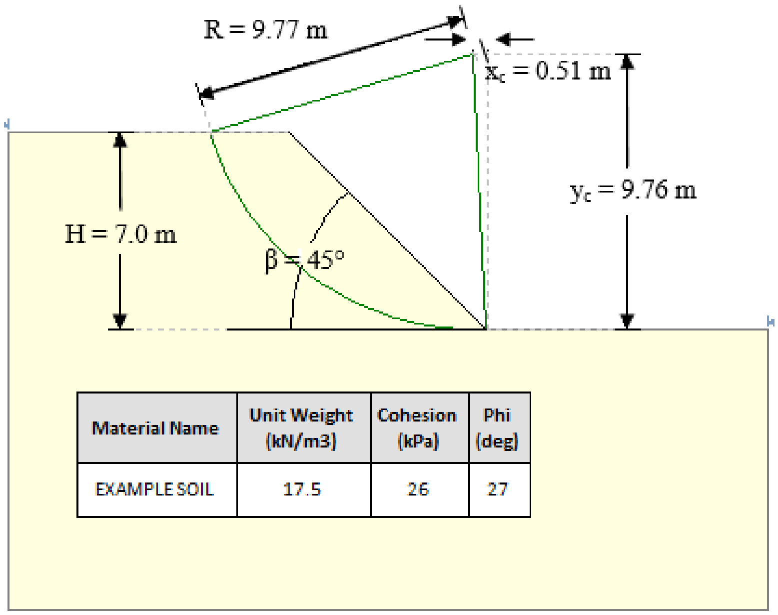


Figure 63: SLIDE model of example, verifying slope stability chart

Entering values from the example (Figure 63) into Equation 6 (i.e. $\beta = 45°$ and $X = 2.4$) yields the output $Y = 4.37$ and a corresponding FS = 2.23. Compared to the actual example value $Y = 4.4$ and the LEM output FS = 2.25, the general equation, for this particular example, provides the same results with low approximation error.

A similar comparative analysis of the same slope uses stability charts to find the location and size of the circular failure surface. Figure 64 shows the example output (the same example slope as Figure 63) values of R, x_c, and y_c computed by SLIDE.



Material Name	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
EXAMPLE SOIL	17.5	26	27

Figure 64: SLIDE output of example showing slope failure geometry

Figure 65 shows stability analysis using the chart for circular failure radius normalized by slope height ($\overline{R}$). With the chart, users predict a failure radius of 9.8 m; the SLIDE output indicates R = 9.77 m.

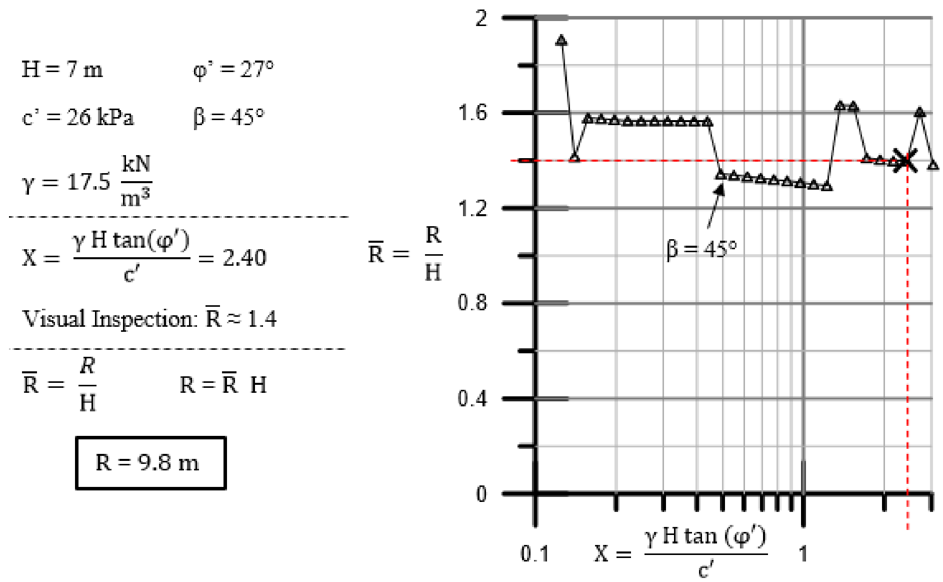


Figure 65: Example of stability analysis using the $\overline{R}$ Chart

Figure 66 illustrates chart analysis for the normalized horizontal distance from the toe to center of the circular failure ($\overline{x}_c$).

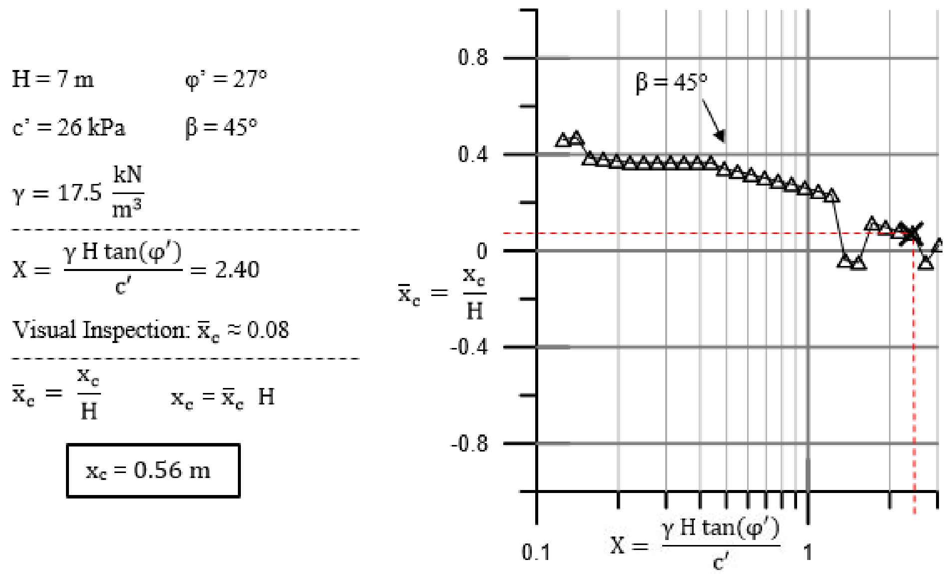


Figure 66: Example of stability analysis using the $\overline{\mathrm{x}}_{\mathrm{c}}$ Chart

The chart predicts x_c = 0.56 m, compared to the SLIDE output x_c = 0.51 m. The normalized height of the failure center can be estimated the same way. From the $\overline{\mathrm{y}}_{\mathrm{c}}$ chart (Figure 67), users find y_c = 9.8, compared to the SLIDE output y_c = 9.76.

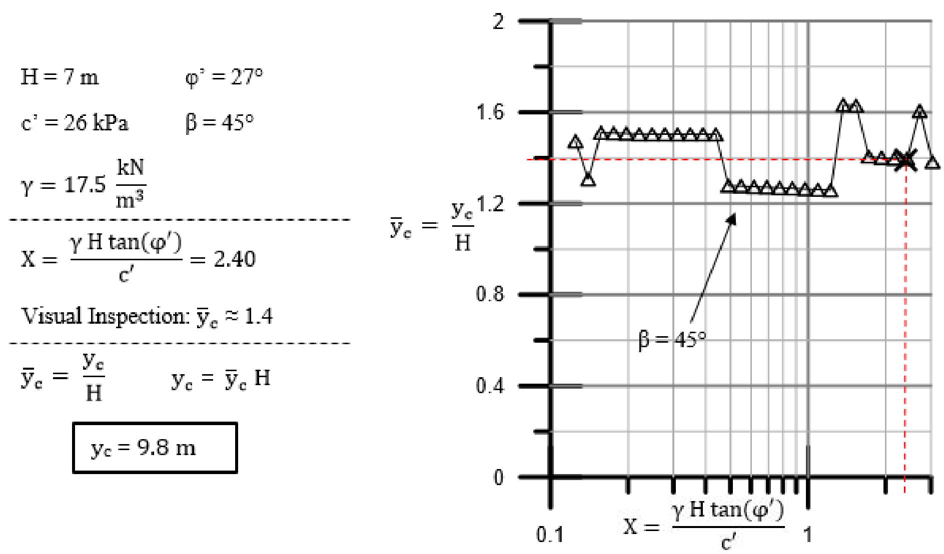


Figure 67: Example of stability analysis using the $\overline{\mathrm{y}}_{\mathrm{c}}$ Chart

Each of the circular failure geometry charts appears to predict parameters with small errors compared to LEM outputs. Users can therefore obtain a quick and reasonably accurate output parameter estimate from the developed slope stability charts.

5.4. Applicability of Charts

Because the stability chart vertical value – Y – is normalized by the tangent of φ’, multiple slopes can have the same Y value (and corresponding X value) and have different FS outputs. Figure 68 shows two more slopes that match the example’s point on the chart (i.e. β = 45º, X = 2.4, Y = 4.4. Although the sites have different FS outputs, they will have the same $\overline{R}$, $\overline{x}_c$, and $\overline{y}_c$, indicating the same failure surface size and relative position (normalized by height), as well as the same normalized stability output parameter (Y).

Figure 68a shows a slope with a higher factor of safety than the first example, while the factor of safety in Figure 68b is lower. Rearranging Equation 5 allows users to directly compute FS. In the case of Example Soil 2 in Figure 68a, 4.4 tan (30°) = FS = 2.54, effectively the same as the LEM output FS = 2.55. For Example Soil 3, 4.4 tan (22°) = FS = 1.78, exactly the same value from Figure 68b.

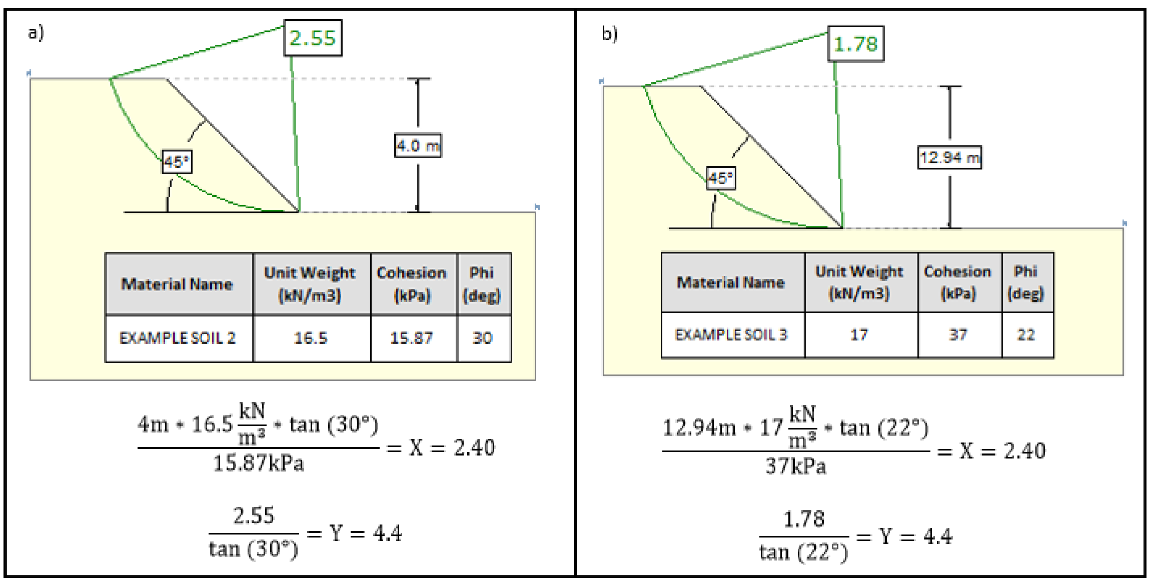


Figure 68: Example different slopes whose inputs provide the same stability chart point (X = 2.4, Y = 4.4)

The examples illustrate that although input parameters for various slopes may be different, the parameters X and Y (from Equations 4 and 5) govern the similarity of the way in which slopes are expected to fail. Given a representative range of values was used to develop charts, end users can perform calculations on any given roadway embankment encountered.

5.5. Summary

The slope stability charts in this chapter can serve as tools for engineers in roadway embankment design and maintenance. The factor of safety chart can help users conduct preliminary slope stability analysis without LEM modeling software. The charts for R, x_c, and y_c can also assist users, and are available in Appendix E. The charts predicting geometric characteristics of the failure surface appear to have discontinuities in the curves, possibly reflecting different failure modes (i.e. failure at the toe vs. a global rotational failure affecting the entire slope). Stability modeling used default program parameters to determine failure characteristics; modifying program inputs to explore failure modes and minimize discontinuities in the charts is a possible area of future research.

Knowing the location of the circular failure surface and radius can be valuable in the preliminary design of stabilization methods. For designs requiring soil nailing, the failure geometry is important to ensure stabilizing structures are placed beyond the failure surface. The author illustrated use of the factor of safety charts and general equation with examples; the deliverable tool provides a reliable stability estimate with low error.

The development of the slope stability charts can serve as an example for future research. Following the process of parametric analysis and steps used to develop the stability charts can assist future researchers in producing similar dimensionless charts that include other commonly encountered conditions, such as location of the groundwater table, and multiple soil types present.

6. SUMMARY AND CONCLUSIONS

In this thesis, the author studied locally-maintained slopes requiring recurring maintenance. A survey of Minnesota county engineers identified case histories representative of the project scope. Respondents identified stabilization methods, and sites at which field investigations could be conducted. The author researched various stabilization methods in a literature review. Additionally, laboratory testing characterized soil properties and provided representative strength parameters from site samples. Slope stability models were used to investigate the effect of various slope stabilization methods. A parametric study of each stabilization method and each site model identified common site scenarios, and led to conclusions for stabilizing each. A slope stabilization guide for Minnesota county and local government engineers summarized the project's findings and presented recommendations. The author developed and verified slope stability charts specific to roadway embankments. The charts provide a previously unavailable tool for engineers to conduct preliminary slope stability analysis on roadway-specific problems; the charts are presented in Appendix E.

During analysis, some common recommendations were developed. Controlling water is a critical stabilization method, and sensitivity to groundwater directly relates to the friction angle of the soil. Slope surfaces, especially in sand materials, should be covered to protect the embankment from erosion damage. The project deliverable aims to provide individual solutions for each problem users expect to encounter. The guide is presented in Appendix D.

The guide was organized into eight scenarios, because a major challenge in slope stabilization is the variety of site conditions. Providing representative scenarios for comparison of observed conditions will help users obtain general recommendations from the deliverable. Often engineering experience, availability of material, and budget concerns govern repair method selection for county engineers. The author used resources not always available to county engineering and maintenance departments, such as lab testing, slope stability modeling software and geotechnical analysis. The stabilization guide will assist local government engineers in effectively using budget and time resources.

This project provides an example of a parametric study for future engineering research. Recommendations for local government engineers can improve the stability of roadway embankments, minimize slope failure and associated damage, and decrease avoidable maintenance cost with efficient stabilization methods.

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APPENDIX A: CASE STUDY SITE SUPPLEMENTARY INFORMATION

Following is supplementary information for each site, including location descriptions, aerial images, slope geometry profiles, and topographic maps from the United States Geological Survey (USGS). Tables describe the soil and slope characteristics from each site. Location descriptions are approximate and relative, with map overviews at varying scales.

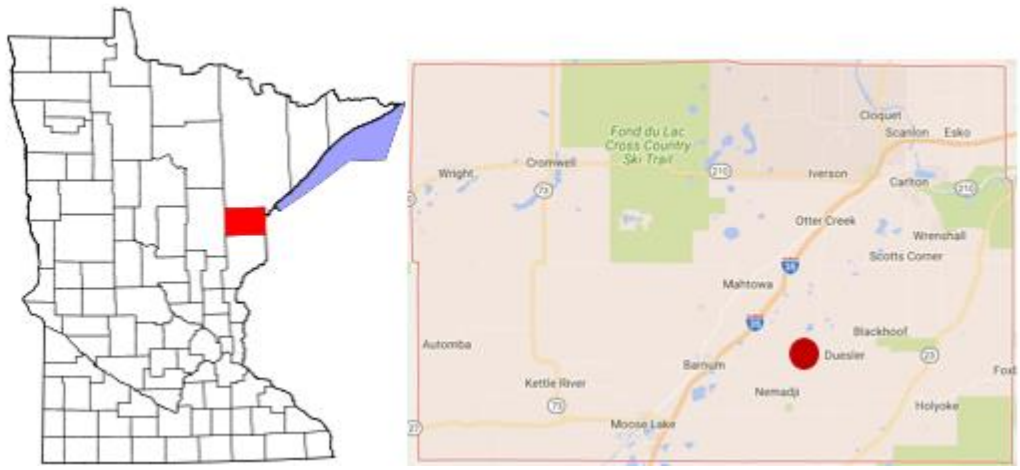


Figure A1.1: Carlton County site location

Figure A1.2: Carlton County site image

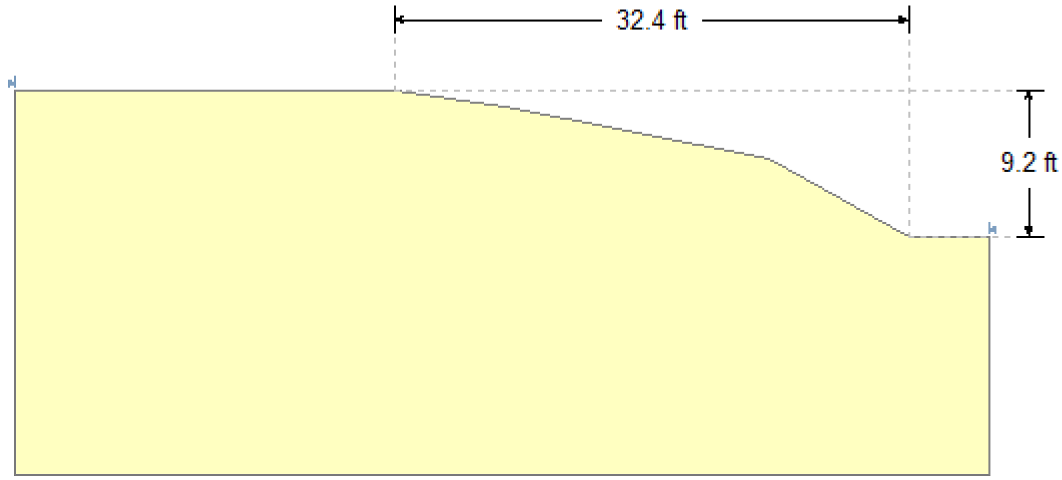


Figure A1.3: Carlton County slope profile

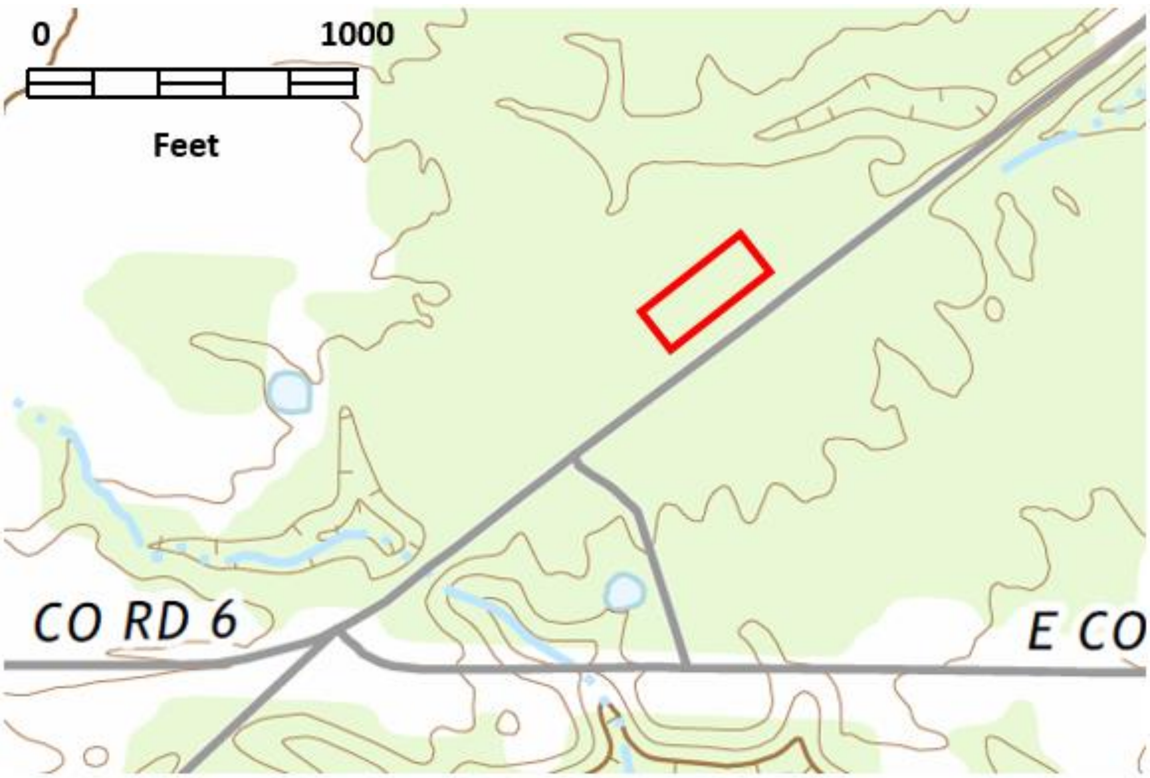


Figure A1.4: Carlton County topography, from USGS Wrenshall Quadrangle, MN 7.5 Minute Map (2016)

Table A1.1: Carlton County Site Soil Characteristics

USCS Classification	CL - Lean clay
SPT Correlation, N_{60} (blows / ft)	2
Moisture Content, w (%)	31.1
Undrained Shear Strength, S_u (tsf)	1.25 to 1.5
Effective Cohesion, c' (psf)	1220
Effective Friction Angle, φ' (deg)	16

Table A1.2: Carlton County Slope Characteristics Summary

Slope failure observed?	No
Failure type	N/A
Evidence / indication of failure	N/A
Water present near toe?	No
Above / below roadway?	Below
Approximate steepness	3.5H : 1V
Observed Stabilization methods	N/A
Topsoil depth	0.5 to 1 ft
Approximate site UTM Coordinates	15T N 5152500 E 539800

Figure A2.1: Carver County site location

Figure A2.2: Carver County site image

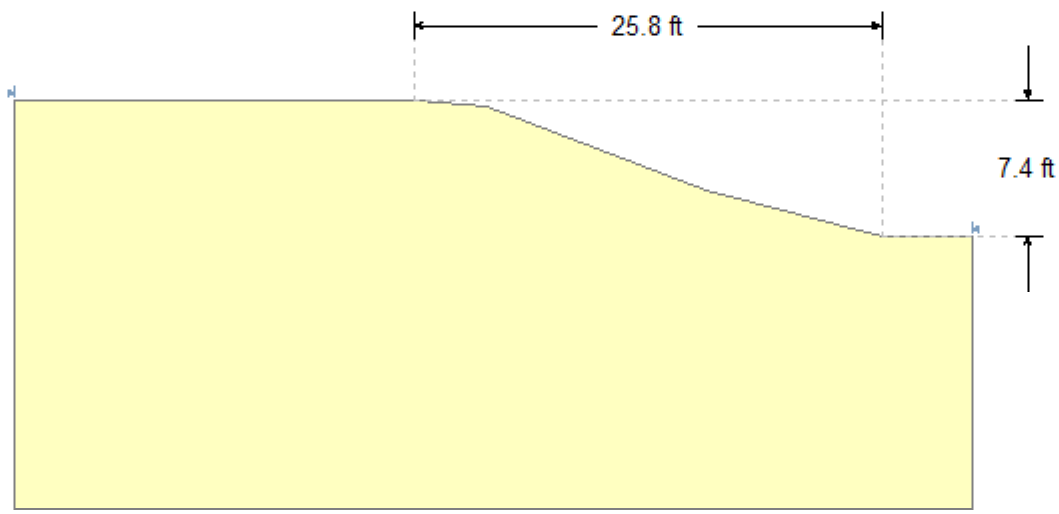


Figure A2.3: Carver County slope profile

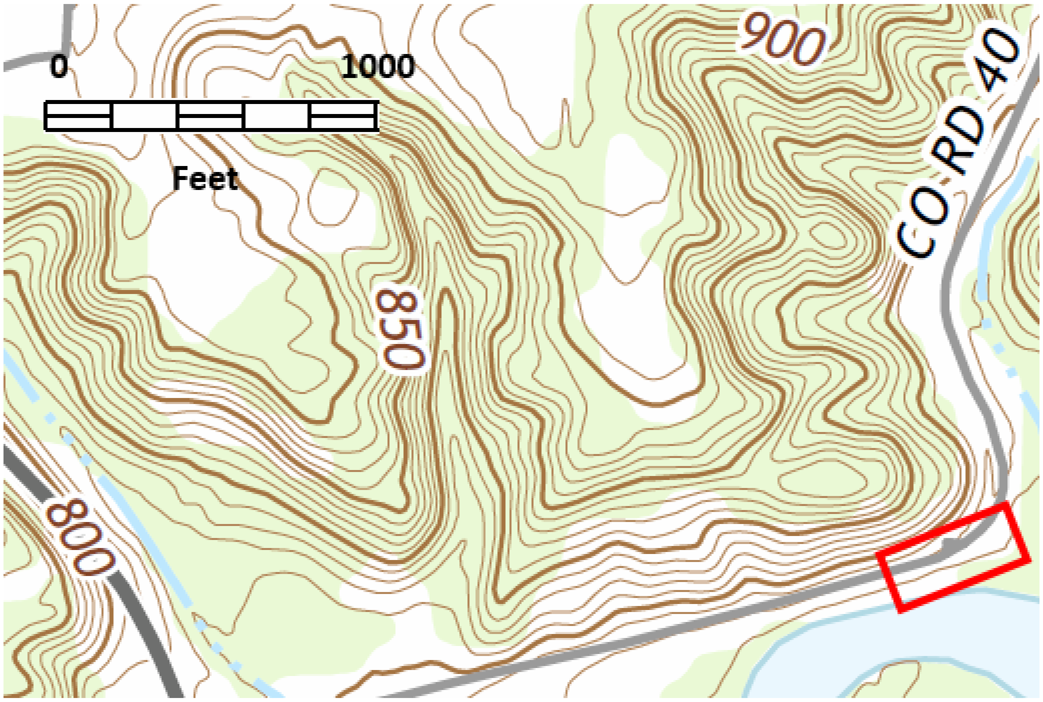


Figure A2.4: Carver County site topography from USGS Belle Plaine North Quadrangle, MN 7.5 Minute Map (2016)

Table A2.1: Carver County Site Soil Characteristics

USCS Classification	SP - Poorly-graded sand
SPT Correlation, N_{60} (blows / ft)	3 to 4
Moisture Content, w (%)	19.5
Undrained Shear Strength, S_u (tsf)	0.5 to 0.75
Effective Cohesion, c' (psf)	200
Effective Friction Angle, φ' (deg)	35

Table A2.2: Carver County Site Slope Characteristics

Slope failure observed?	Yes, no visible failure surface
Failure type	Creep
Evidence / indication of failure	Tilted guardrail posts
Water present near toe?	Yes - Minnesota River
Above / below roadway?	Below
Approximate steepness	3.5 H : 1V
Observed stabilization methods	N/A
Topsoil depth	0.5 to 1 ft
Approximate site UTM coordinates	15T N 4943700 E 439900

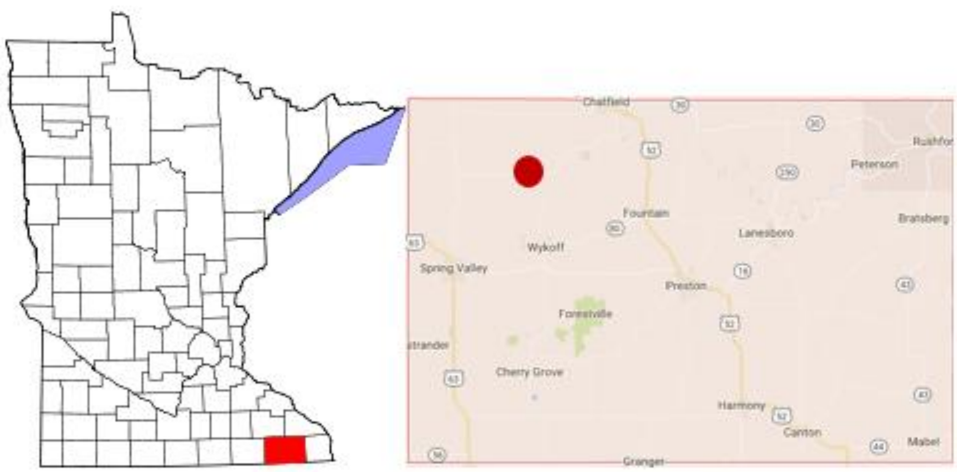


Figure A3.1: Fillmore County site location

Figure A3.2: Fillmore County site image

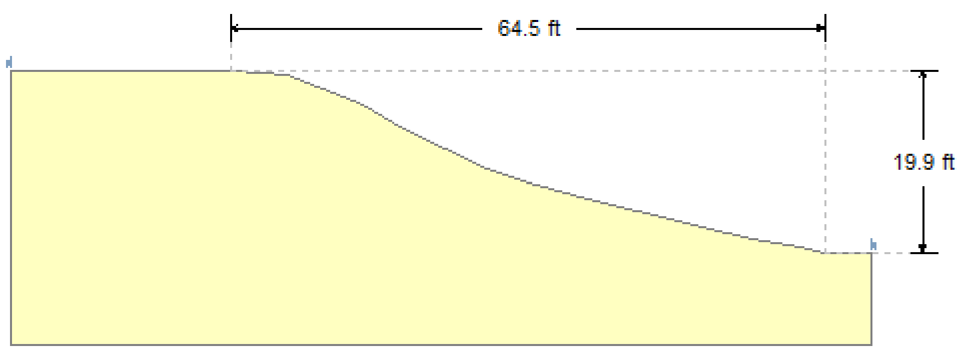


Figure A3.3: Fillmore County slope profile

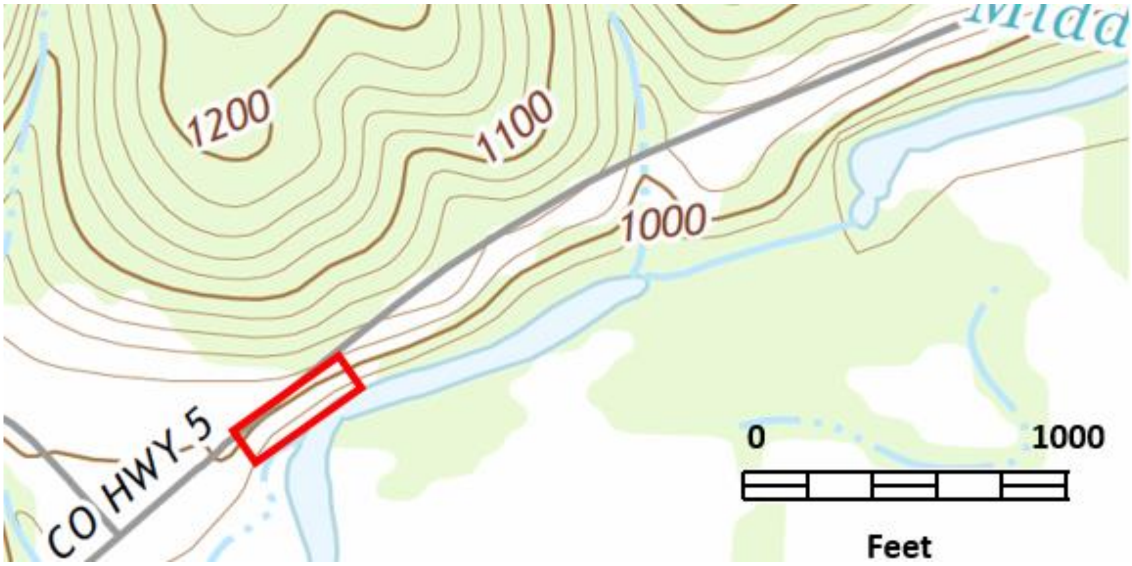


Figure A3.4: Fillmore County site topography from USGS Chatfield Quadrangle, MN 7.5 Minute Map (2016)

Table A3.1: Fillmore County Site Soil Characteristics

USCS Classification	ML – Silt with sand
SPT Correlation, N_{60} (blows / ft)	4 to 5
Moisture Content, w (%)	21.4
Undrained Shear Strength, S_u (tsf)	1.25 to 1.75
Effective Cohesion, c' (psf)	150
Effective Friction Angle, φ' (deg)	35

Table A3.2: Fillmore County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Rotational
Evidence / indication of failure	Pavement failure, visible scarp
Water present near toe?	Yes- Middle Branch Root River
Above / below roadway?	Below
Approximate steepness	3.5H : 1V
Observed Stabilization methods	None
Topsoil depth	0.5 ft
Approximate site UTM coordinates	15T N 4849700 E 561700

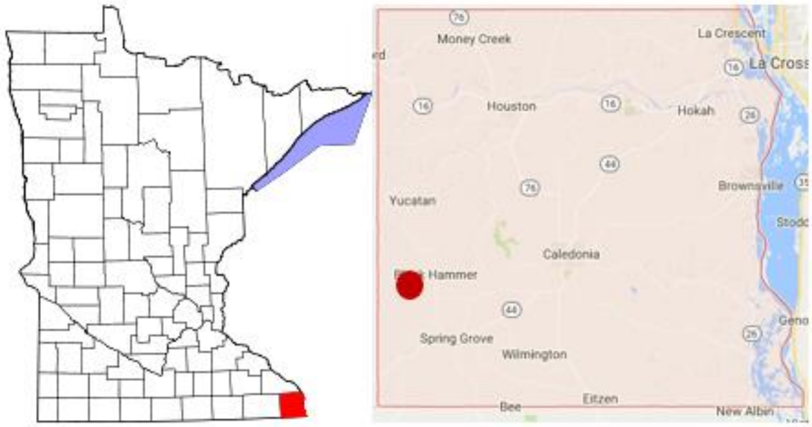


Figure A4.1: Houston County site location

Figure A4.2: Houston County site image

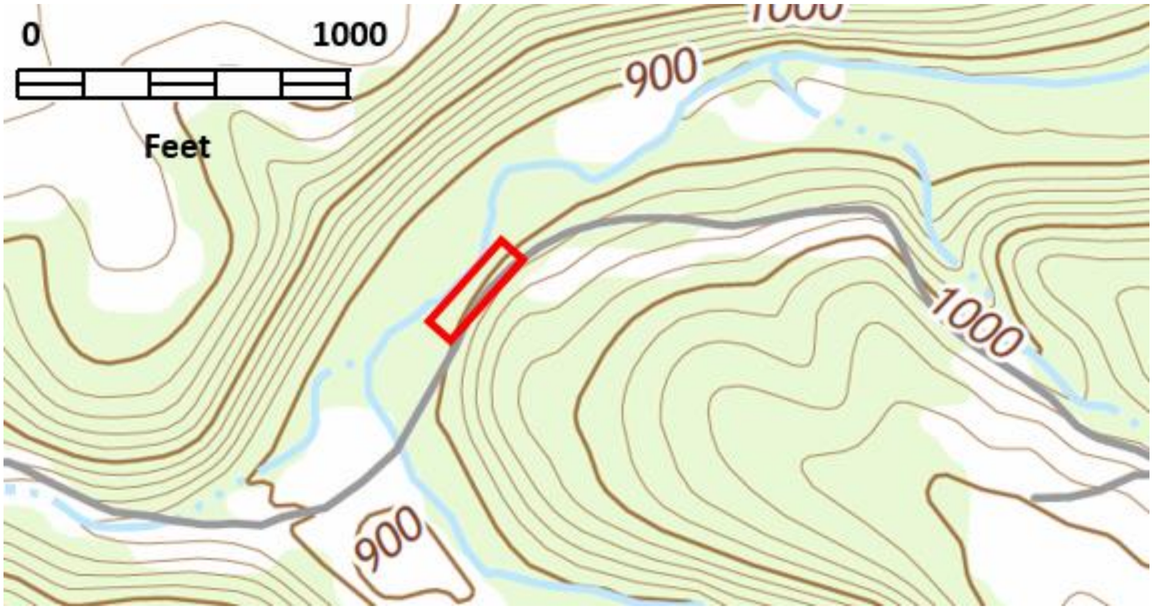


Figure A4.3: Houston County site topography from USGS Spring Grove Quadrangle, MN 7.5 Minute Map (2016)

Table A4.1: Houston County Site Soil Characteristics

USCS Classification	SC – Clayey Sand
SPT Correlation, N_{60} (blows / ft)	Not tested
Moisture Content, w (%)	19.9
Undrained Shear Strength, S_u (tsf)	Not tested
Effective Cohesion, c' (psf)	300
Effective Friction Angle, φ' (deg)	34

Table A4.2: Houston County Slope Characteristics Summary

Slope failure observed?	No (repaired)
Failure type	Rotational
Evidence / indication of failure	Failure across road (repaired)
Water present near toe?	Yes - Riceford Creek
Above / below roadway?	Below
Approximate steepness	2H : 1V (repaired)
Observed Stabilization methods	Rip Rap cover
Topsoil depth	not measured
Approximate site UTM coordinates	15T N 4629000 E 604000

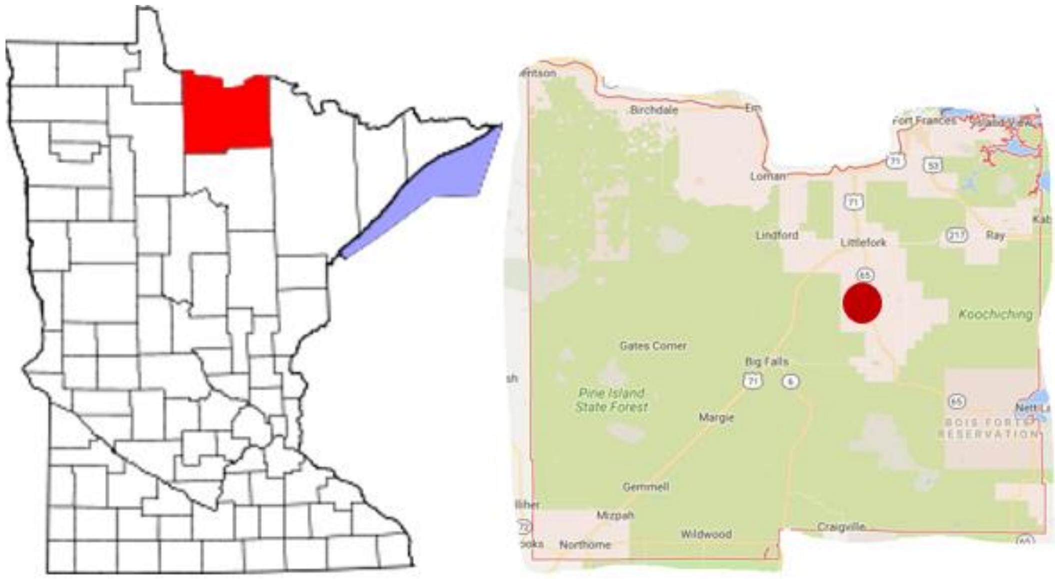


Figure A5.1: Koochiching County site location

Figure A5.2: Koochiching County site image

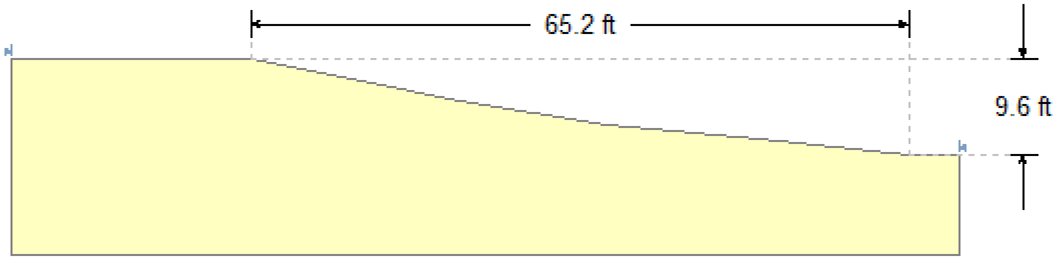


Figure A5.3: Koochiching County slope profile

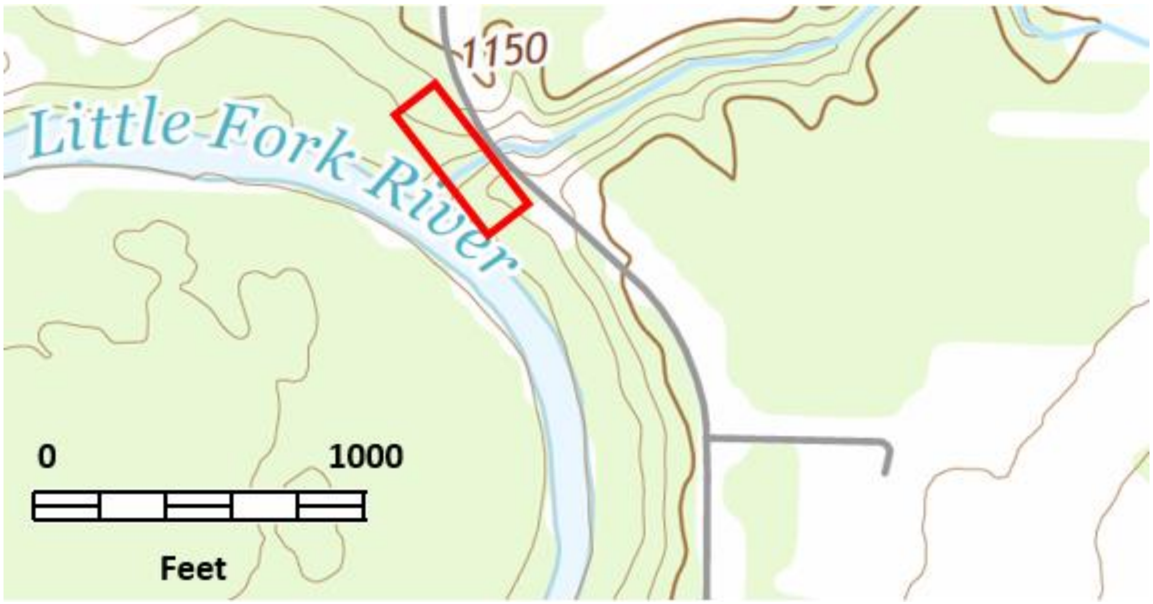


Figure A5.4: Koochiching County site topography from USGS Ericsburg SW Quadrangle, MN 7.5 Minute Map (2016)

Table A5.1: Koochiching County Site Soil Characteristics

USCS Classification	CL - Sandy lean clay
SPT Correlation, N_{60} (blows / ft)	5
Moisture Content, w (%)	26.4
Undrained Shear Strength, S_u (tsf)	1 to 1.5
Effective Cohesion, c' (psf)	400
Effective Friction Angle, φ' (deg)	24

Table A5.2: Koochiching County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Creep
Evidence / indication of failure	Visible Scarp on face
Water present near toe?	Yes - Littlefork River
Above / below roadway?	Below
Approximate steepness	6.5H : 1V
Observed Stabilization methods	None
Topsoil depth	0.5 to 1 ft
Approximate site UTM coordinates	15T N 5349250 E 467400

Figure A6.1: Lac Qui Parle County site location

Figure A6.2: Lac Qui Parle County site image

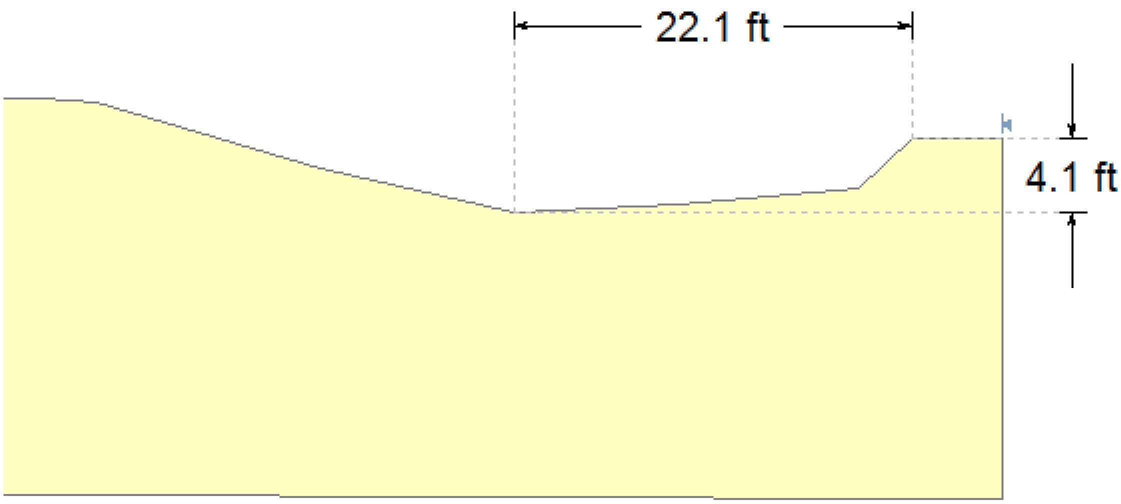


Figure A6.3: Lac Qui Parle County slope profile

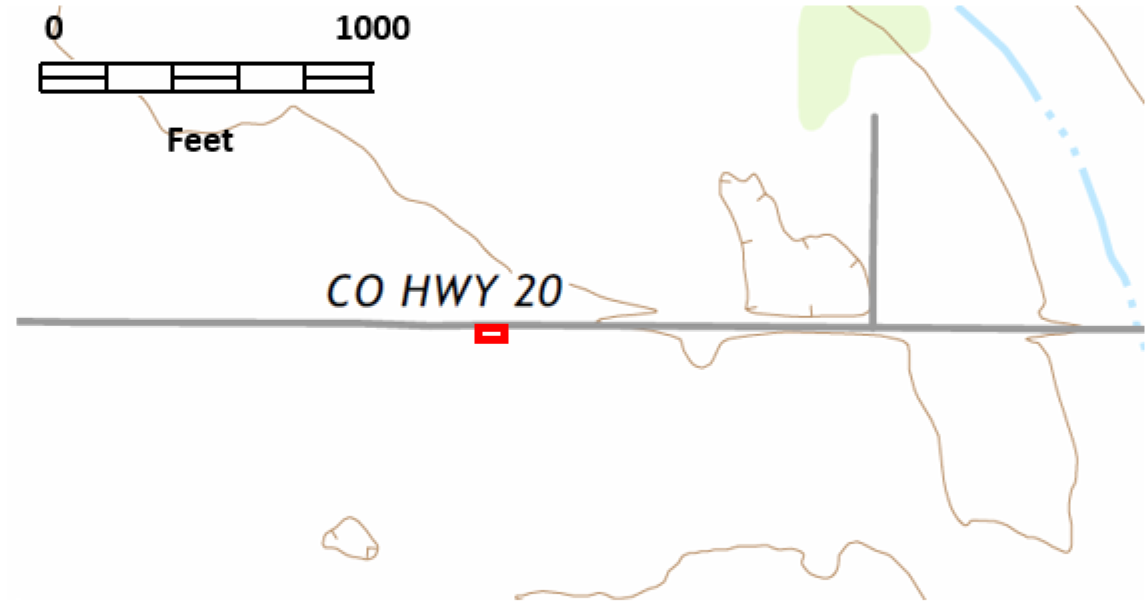


Figure A6.4: Lac Qui Parle County site topography from USGS Clarkfield NE Quadrangle, MN 7.5 Minute Map (2016)

Table A6.1: Lac Qui Parle County Site Soil Characteristics

USCS Classification	SP-SM - Poorly-graded sand with silt
SPT Correlation, N_{60} (blows / ft)	5 to 7
Moisture Content, w (%)	18.7
Undrained Shear Strength, S_u (tsf)	1.25 to 2
Effective Cohesion, c' (psf)	50
Effective Friction Angle, φ' (deg)	35

Table A6.2: Lac Qui Parle County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Rotational (erosion)
Evidence / indication of failure	Visible washout failures
Water present near toe?	No
Above / below roadway?	Above
Approximate steepness	1.5H : 1V (backslope)
Observed Stabilization methods	None
Topsoil depth	1 ft
Approximate site UTM coordinates	15T N 4986100 E 274500

Figure A7.1: Marshall County site location

Figure A7.2: Marshall County site image

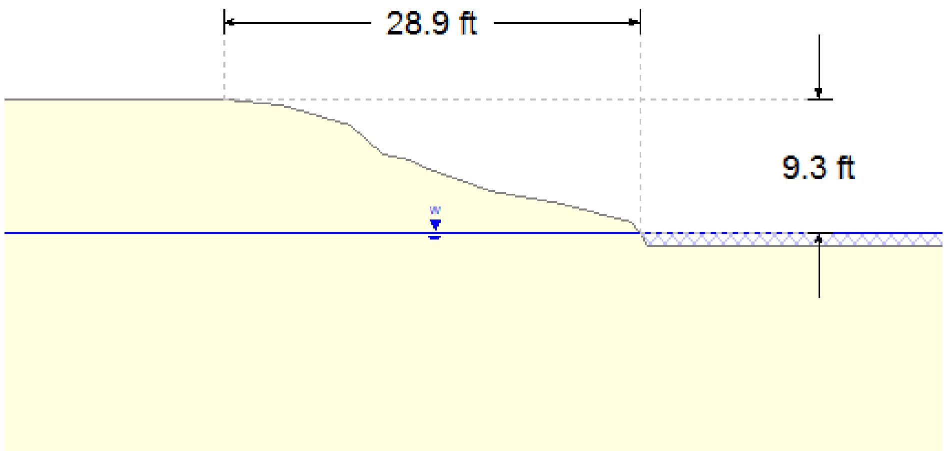


Figure A7.3: Marshall County slope profile

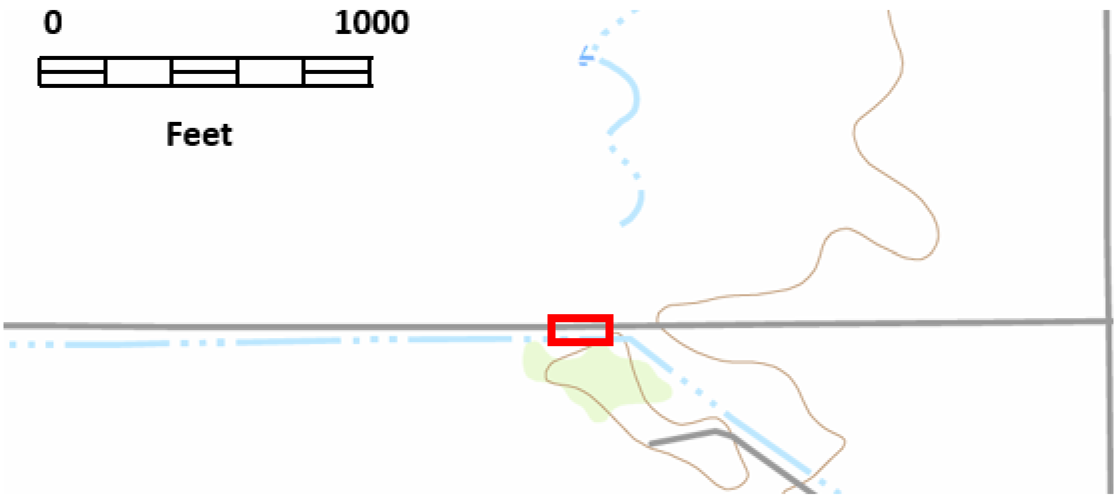


Figure A7.4: Marshall County site topography from USGS Argyle Quadrangle, MN 7.5 Minute Map (2016)

Table A7.1: Marshall County Soil Characteristics

USCS Classification	MH - Elastic Silt
SPT Correlation, N_{60} (blows / ft)	3 to 4
Moisture Content, w (%)	21.8
Undrained Shear Strength, S_u (tsf)	1.25 to 1.75
Effective Cohesion, c' (psf)	600
Effective Friction Angle, φ' (deg)	18

Table A7.2: Marshall County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Creep
Evidence / indication of failure	Visible soil movement
Water present near toe?	yes
Above / below roadway?	below
Approximate steepness	2.5H:1V
Observed Stabilization methods	N/A
Topsoil depth	1 ft
Approximate site UTM coordinates	14U N 5349900 E 660500

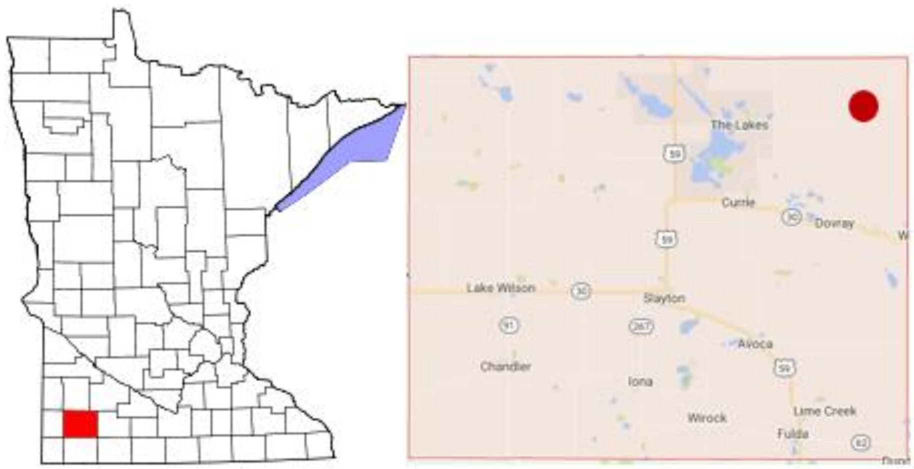


Figure A8.1: Murray County site location

Figure A8.2: Murray County site image

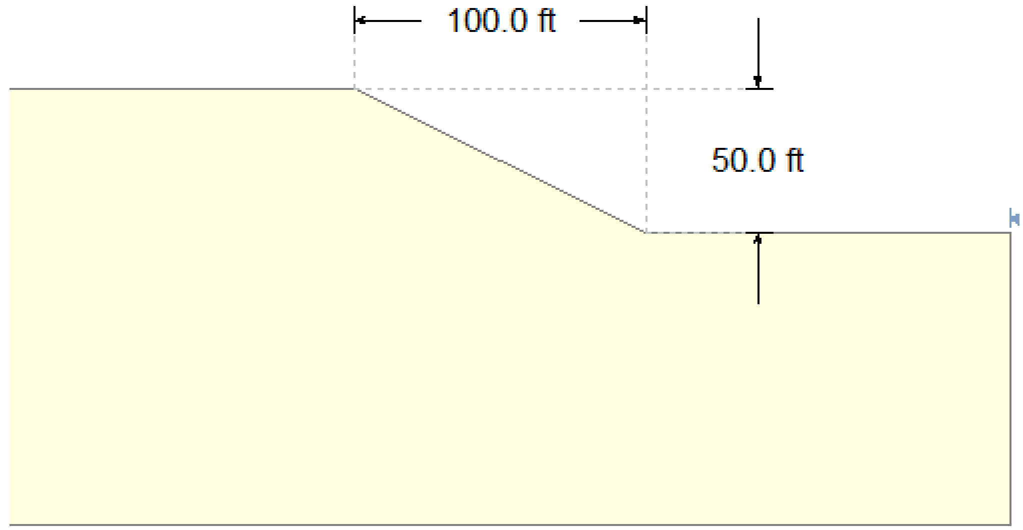


Figure A8.3: Murray County slope profile

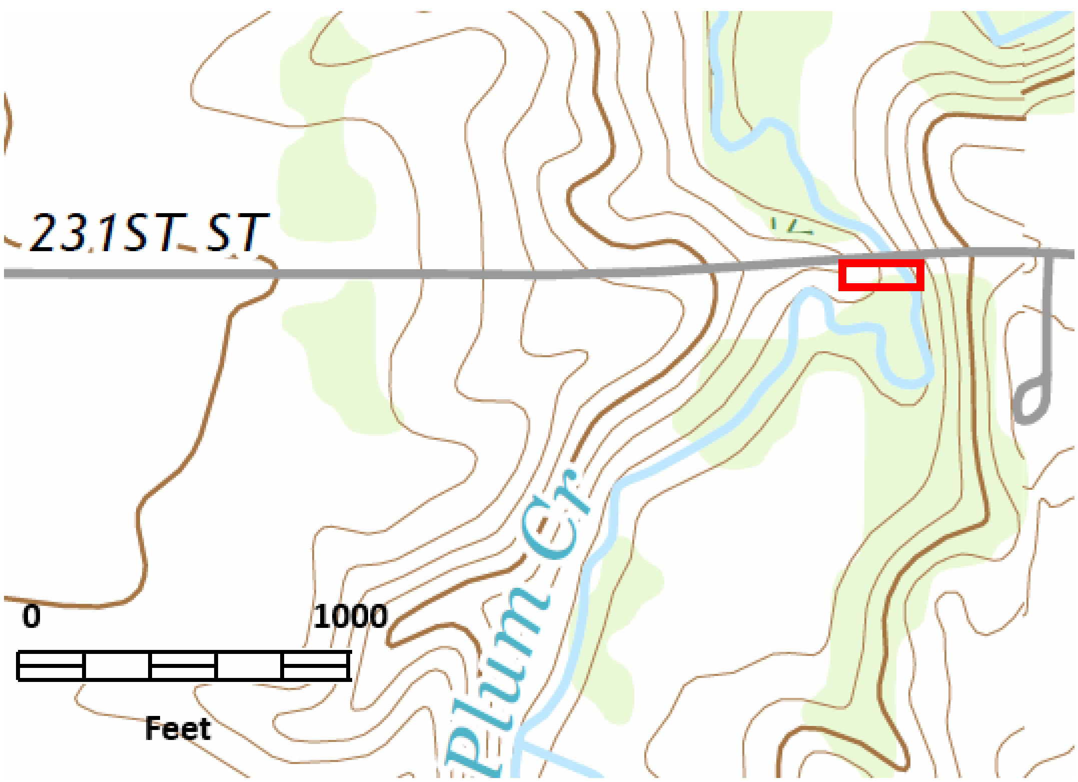


Figure A8.4: Murray County site topography from USGS Tracy East Quadrangle, MN 7.5 Minute Map (2016)

Table A8.1: Murray County Site Native Soil Characteristics

USCS Classification	ML - Sandy silt
SPT Correlation, N_{60} (blows / ft)	3 to 4
Moisture Content, w (%)	30.6
Undrained Shear Strength, S_u (tsf)	not tested
Effective Cohesion, c' (psf)	900
Effective Friction Angle, φ' (deg)	22

Table A8.2: Murray County Site Fill Soil Characteristics

USCS Classification	SC - Clayey Sand
SPT Correlation, N_{60} (blows / ft)	3 to 4
Moisture Content, w (%)	30.6
Undrained Shear Strength, S_u (tsf)	not tested
Effective Cohesion, c' (psf)	390
Effective Friction Angle, φ' (deg)	32

Table A8.3: Murray County Slope Characteristics Summary

Slope failure observed?	Yes (repaired)
Failure type	Creep at top of slope
Evidence / indication of failure	Pavement distress
Water present near toe?	Yes (Plumb Creek)
Above / below roadway?	Below
Approximate steepness	2H : 1V
Observed Stabilization methods	Remove and Replace
Topsoil depth	0.5 to 1 ft
Approximate site UTM coordinates	15T N 4895100 E 300050

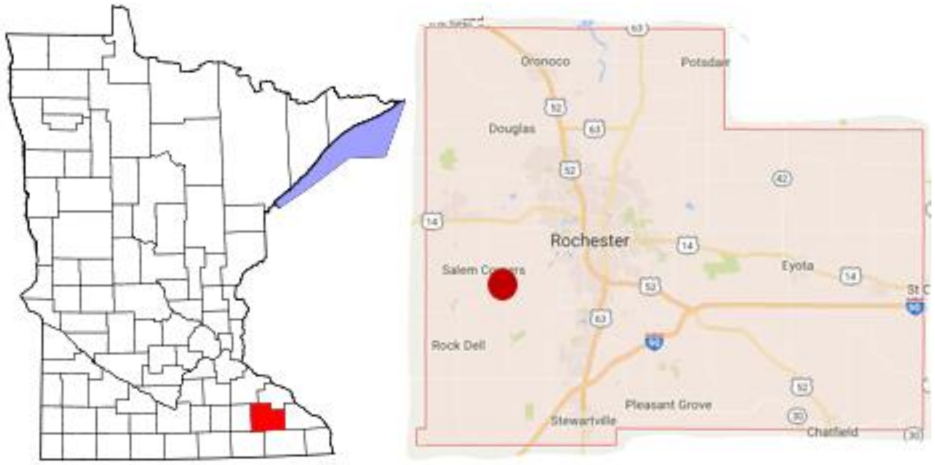


Figure A9.1: Olmsted County site location

Figure A9.2: Olmsted County site image

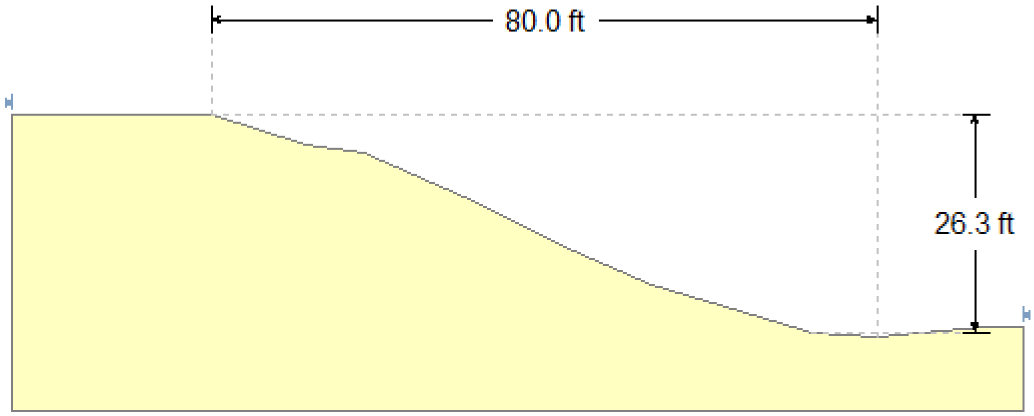


Figure A9.3: Olmsted County slope profile

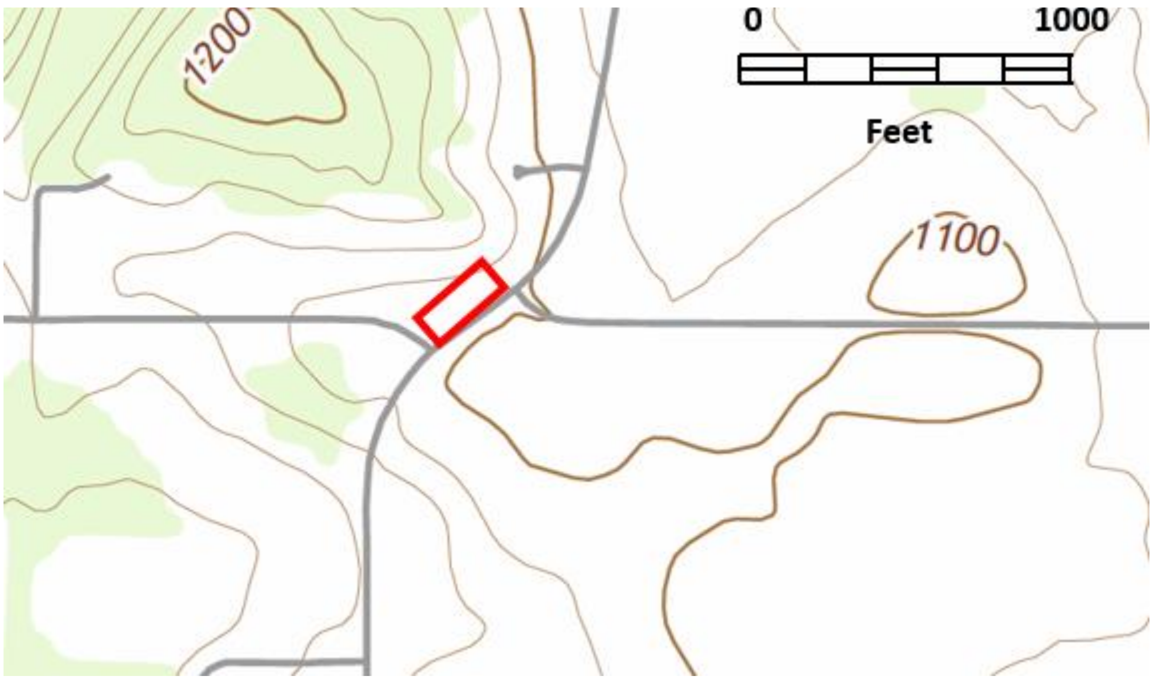


Figure A9.4: Olmsted County site topography from USGS Salem Corners Quadrangle, MN 7.5 Minute Map (2016)

Table A9.1: Olmsted County Site Soil Characteristics

USCS Classification	CL - Sandy lean clay
SPT Correlation, N_{60} (blows / ft)	3
Moisture Content, w (%)	16.8
Undrained Shear Strength, S_u (tsf)	0.25 to 0.5
Effective Cohesion, c' (psf)	200
Effective Friction Angle, φ' (deg)	34

Table A9.2: Olmsted County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Rotational and creep
Evidence / indication of failure	Visible scarp on face
Water present near toe?	No
Above / below roadway?	Above
Approximate steepness	3H : 1V
Observed Stabilization methods	None
Topsoil depth	0.5 to 1 ft
Approximate site UTM coordinates	15T N 4867200 E 533950

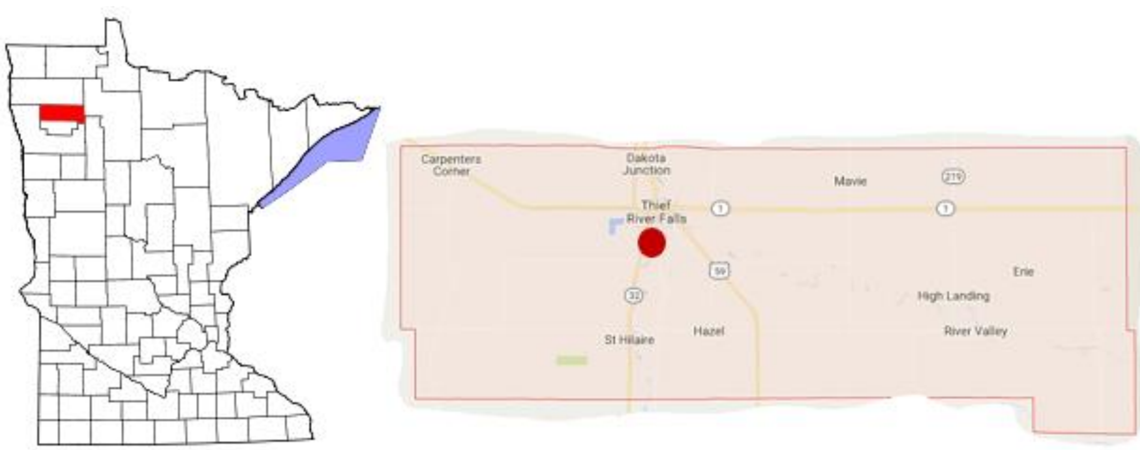


Figure A10.1: Pennington County site location

Figure A10.2: Pennington County site image

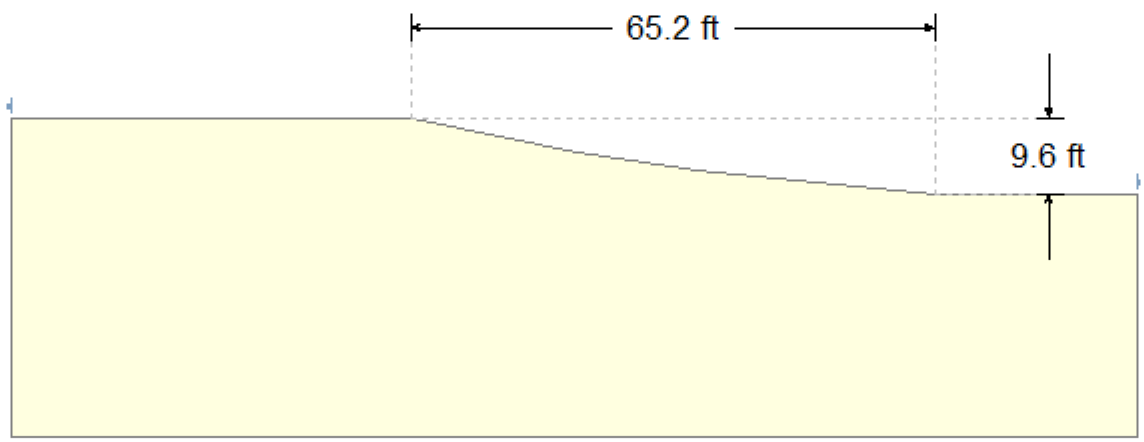


Figure A10.3: Pennington County slope profile

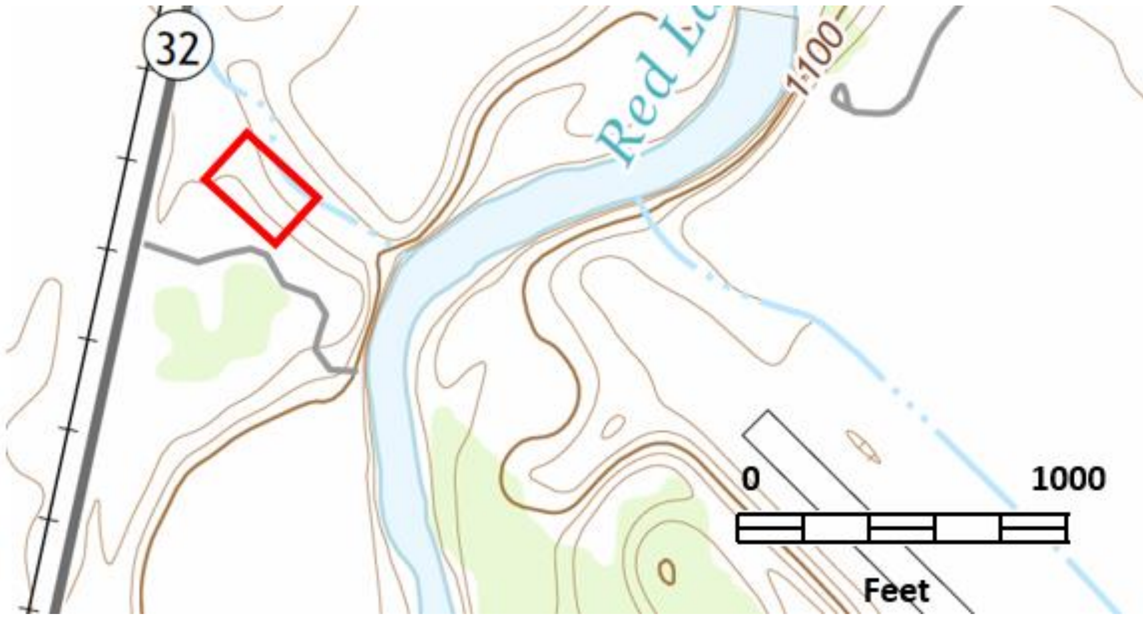


Figure A10.4: Pennington County site topography from USGS Thief River Falls Quadrangle, MN 7.5 Minute Map (2016)

Table A10.1: Pennington County Site Soil Characteristics

USCS Classification	ML - Silt with sand
SPT Correlation, N_{60} (blows / ft)	3 to 4
Moisture Content, w (%)	26.5
Undrained Shear Strength, S_u (tsf)	0.25 to 0.75
Effective Cohesion, c' (psf)	1275
Effective Friction Angle, φ' (deg)	17

Table A10.2: Pennington County Slope Characteristics Summary

Slope failure observed?	Yes
Failure type	Rotational
Evidence / indication of failure	Clearly visible failure surface
Water present near toe?	yes
Above / below roadway?	below
Approximate steepness	1.5H:1V
Observed Stabilization methods	N/A
Topsoil depth	1 ft
Approximate site UTM coordinates	14U N 5328200 E 708400

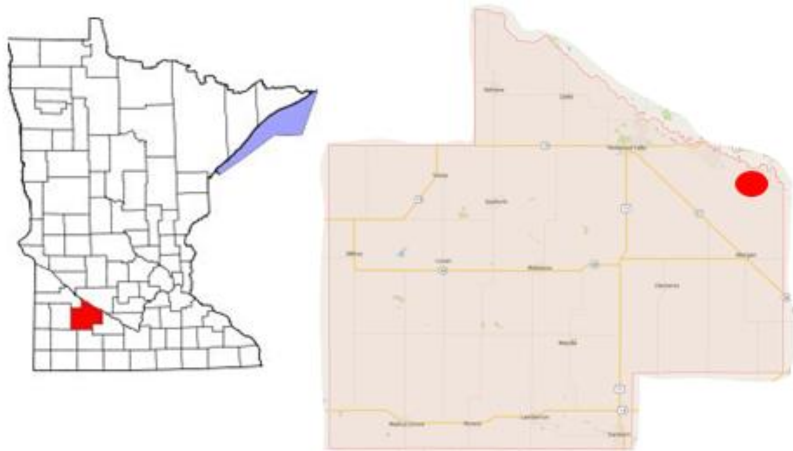


Figure A11.1: Redwood County site location

Figure A11.2: Redwood County site image

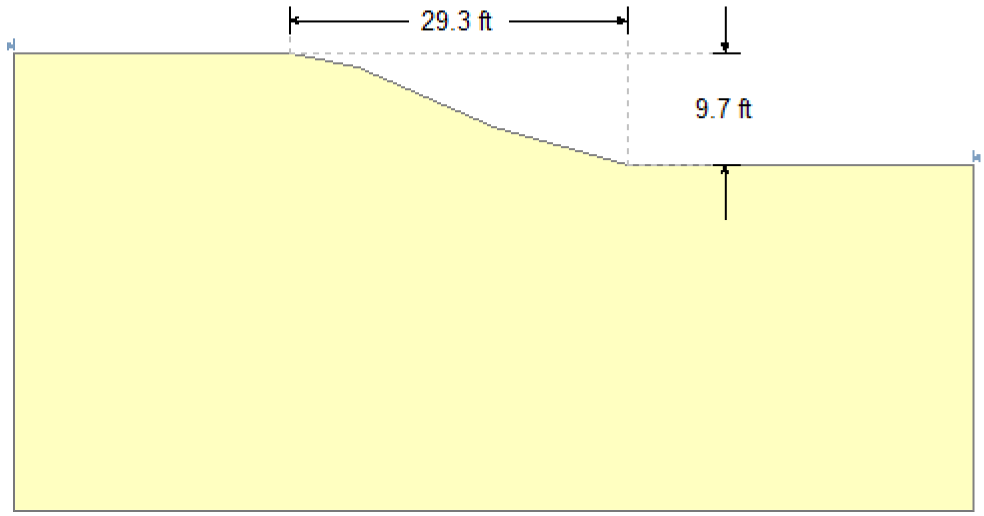


Figure A11.3: Redwood County slope profile

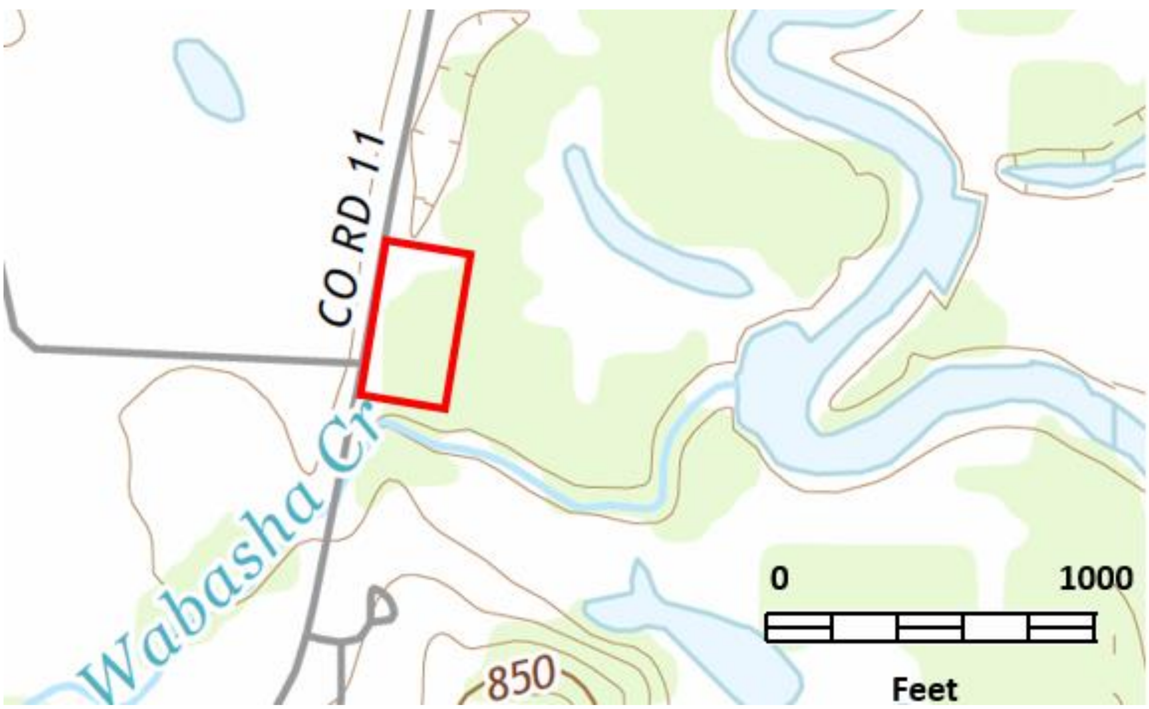


Figure A11.4: Redwood County site topography from USGS Morton Quadrangle, MN 7.5 Minute Map (2016)

Table A11.1: Redwood County Site Soil Characteristics

USCS Classification	CH - Fat clay with sand
SPT Correlation, N_{60} (blows / ft)	4 to 5
Moisture Content, w (%)	36.0
Undrained Shear Strength, S_u (tsf)	0.5
Effective Cohesion, c' (psf)	750
Effective Friction Angle, φ' (deg)	21

Table A11.2: Redwood County Slope Characteristics Summary

Slope failure observed?	No (repaired)
Failure type	N/A
Evidence / indication of failure	Pavement distress, Rip Rap cover
Water present near toe?	Yes - Minnesota River
Above / below roadway?	Below
Approximate steepness	3H : 1V
Observed Stabilization methods	Rip Rap cover, geosynthetics
Topsoil depth	0.5 ft
Approximate site UTM coordinates	15T N 4929000 E 350200

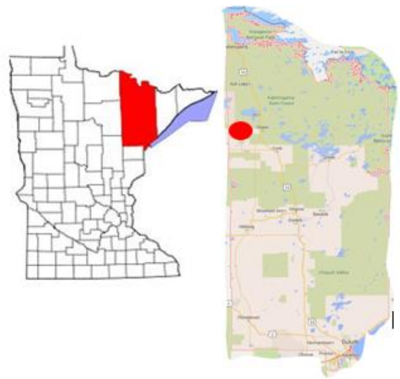


Figure A12.1: St. Louis County site location

Figure A12.2: St. Louis County site image

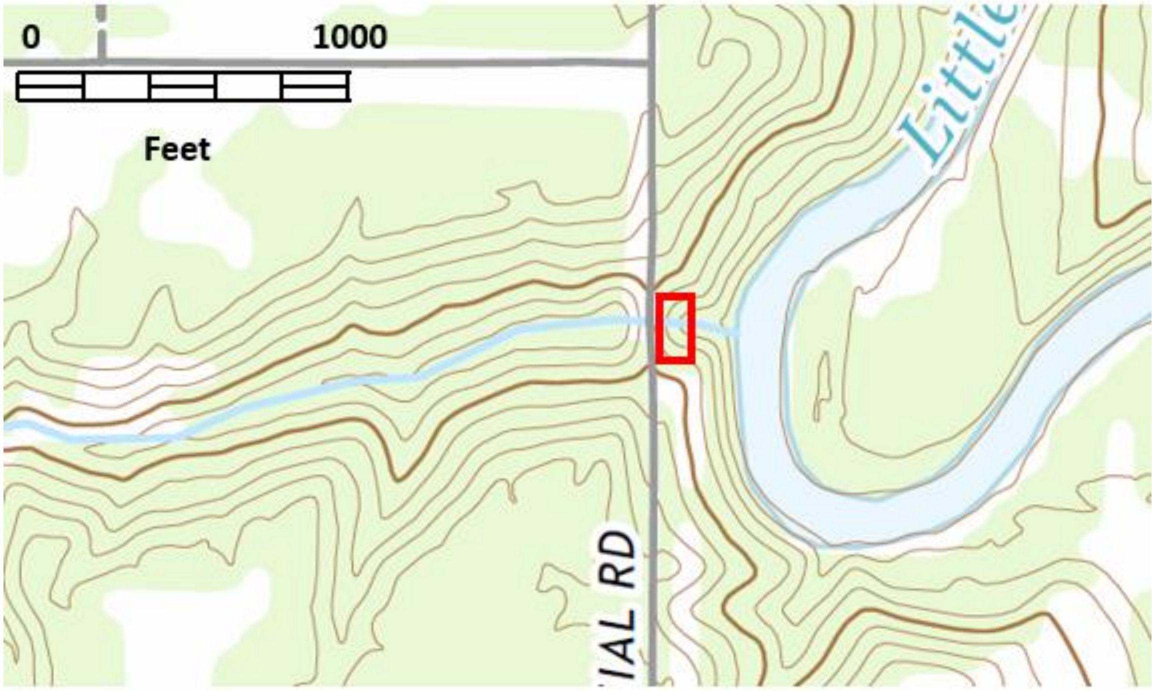


Figure A12.3: St. Louis County site topography from USGS Silverdale Quadrangle, MN 7.5 Minute Map (2016)

Approximate site UTM coordinates: 15T N 5307250 E 494000

Figure A13.1: Washington County site location

Figure A13.2: Washington County site image

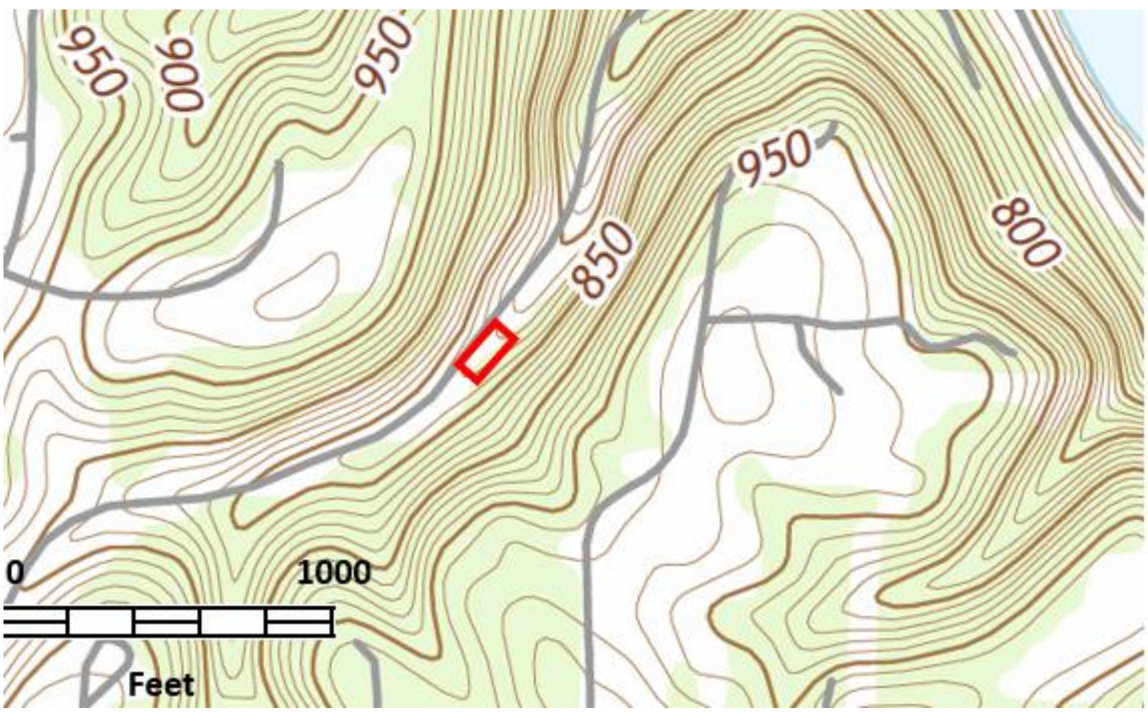


Figure A13.4: Washington County site topography from USGS Hudson Quadrangle, MN-WI 7.5 Minute Map (2016)

Approximate site UTM coordinates: 15T N 4970950 E 516900

APPENDIX B: DNR GROUNDWATER DEPTH OBSERVATION WELLS

Following is a list of DNR groundwater monitoring wells; listed are the well numbers, and each corresponding site. Few sites had monitoring wells near the slope. Groundwater monitoring data is available at:

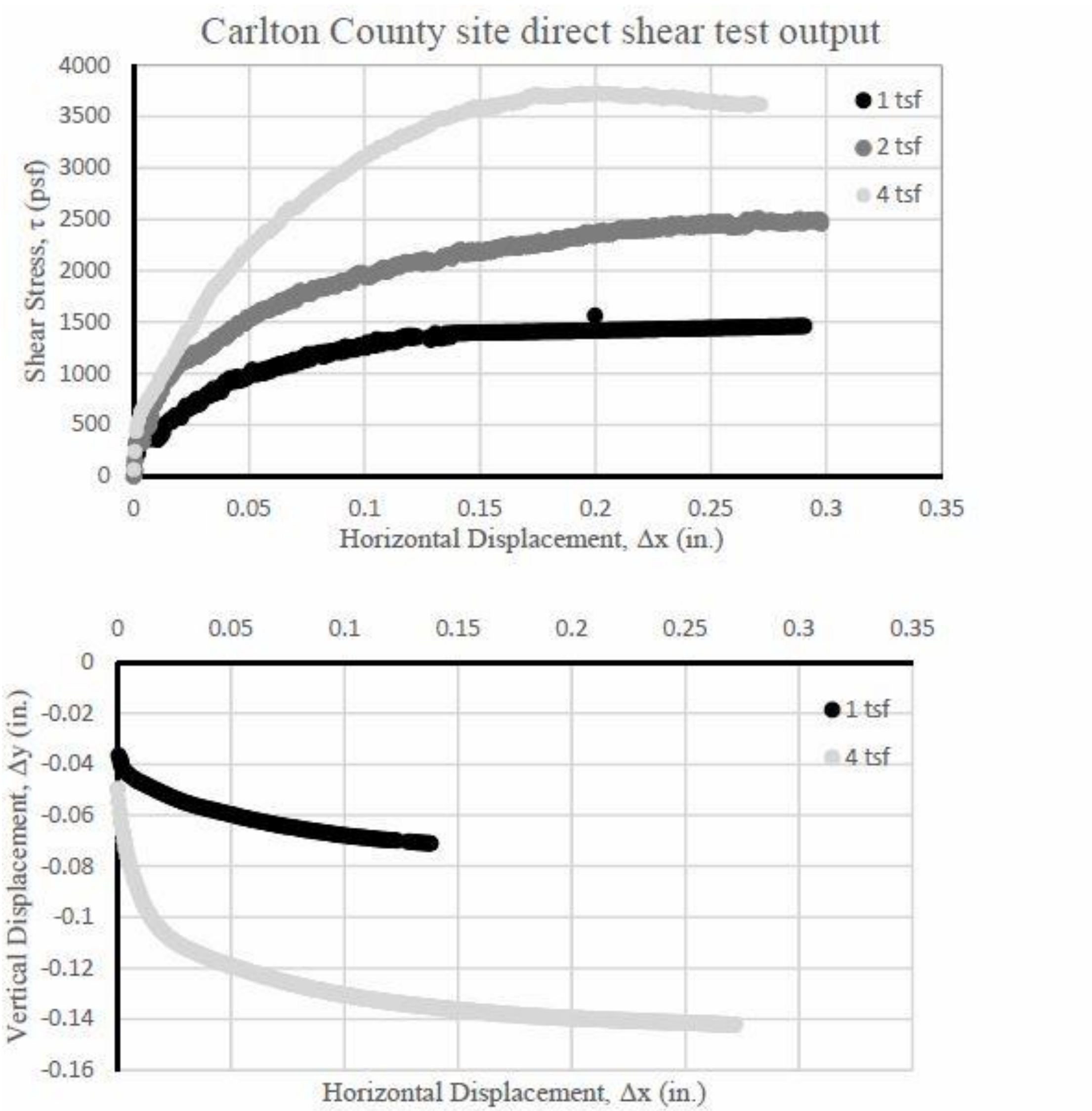
http://www.dnr.state.mn.us/waters/cgm/index.html

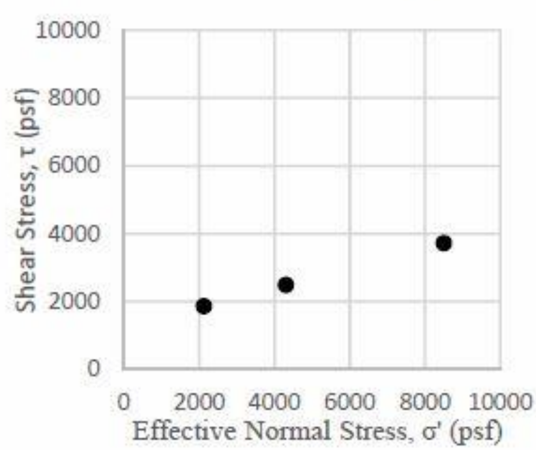
Table B1: List of DNR observation wells

Site	DNR Observation Well Number
Carlton Co.	9030
Carver Co.	70020
Fillmore Co.	23001
Houston Co.	23002
Koochiching Co.	36000
Lac Qui Parle Co.	37007
Marshall Co.	45001
Murray Co.	64000
Olmsted Co.	55001
Pennington Co.	57001
Redwood Co.	64002
St. Louis Co.	31001
Washington Co.	82063

APPENDIX C: SOIL STRENGTH CHARACTERIZATION DATA

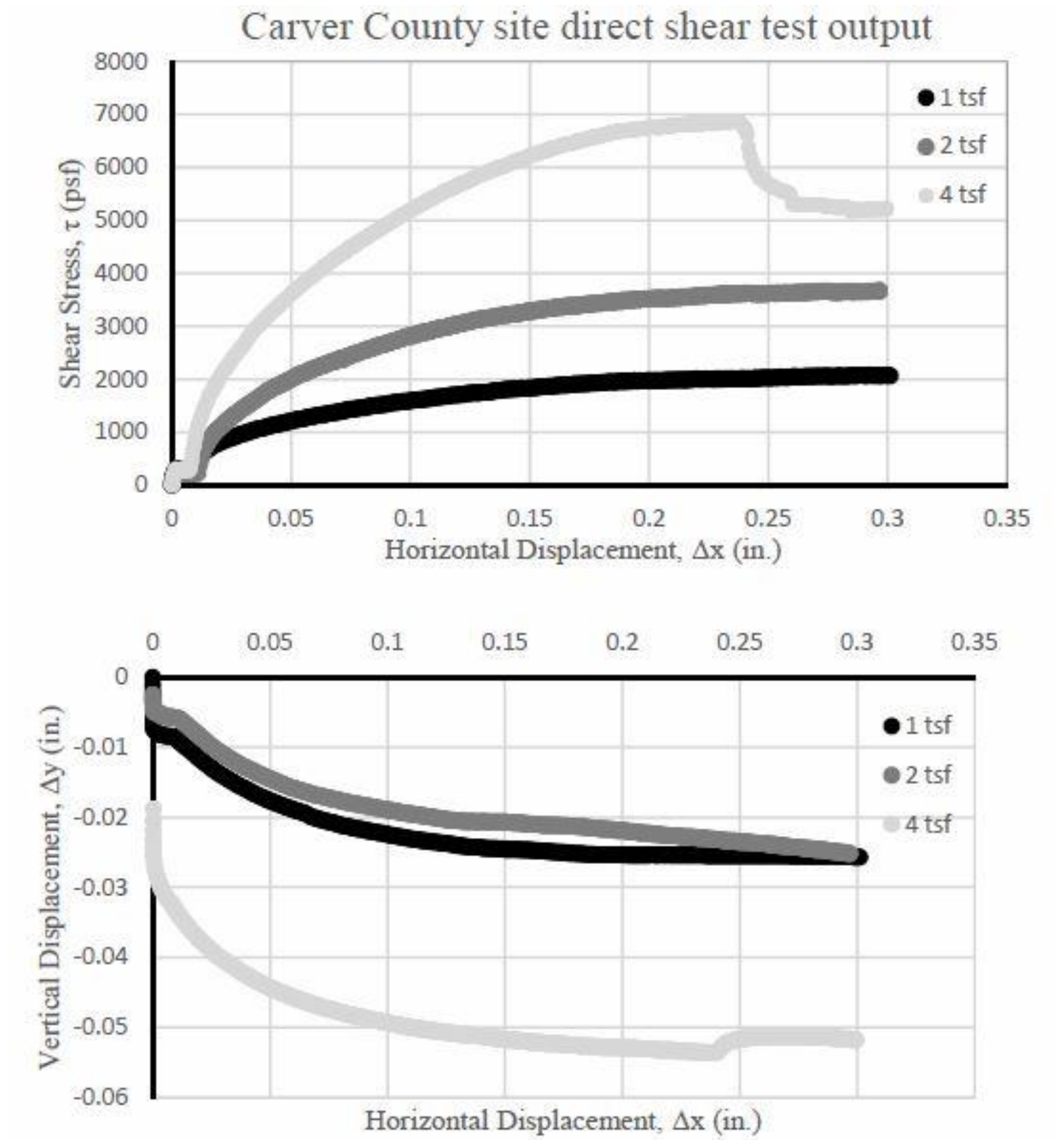
This Appendix provides raw results from the direct shear test. Each sample was tested at three confining stresses: 1 tsf, 2 tsf, and 4 tsf. The author used the plot of horizontal displacement vs. shear stress to identify the maximum shear stress for each test. Plotting the maximum shear stress vs. the corresponding normal stress allowed the calculation of c' and φ'. The vertical displacement outputs are also provided to indicate each sample's shear behavior.

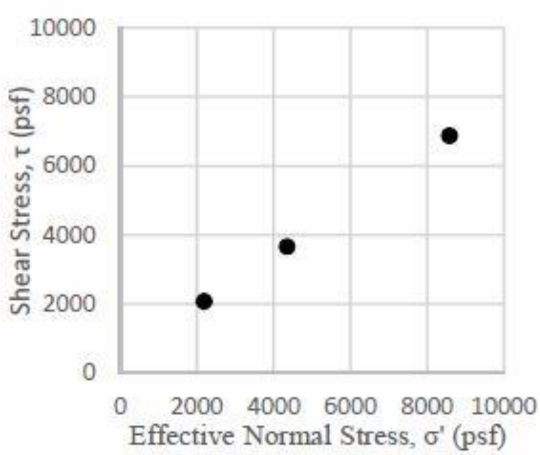




φ' (deg)	16
c' (psf)	1220
S_u (tsf)	1.25 to 1.5
DCP blows per ft.	2

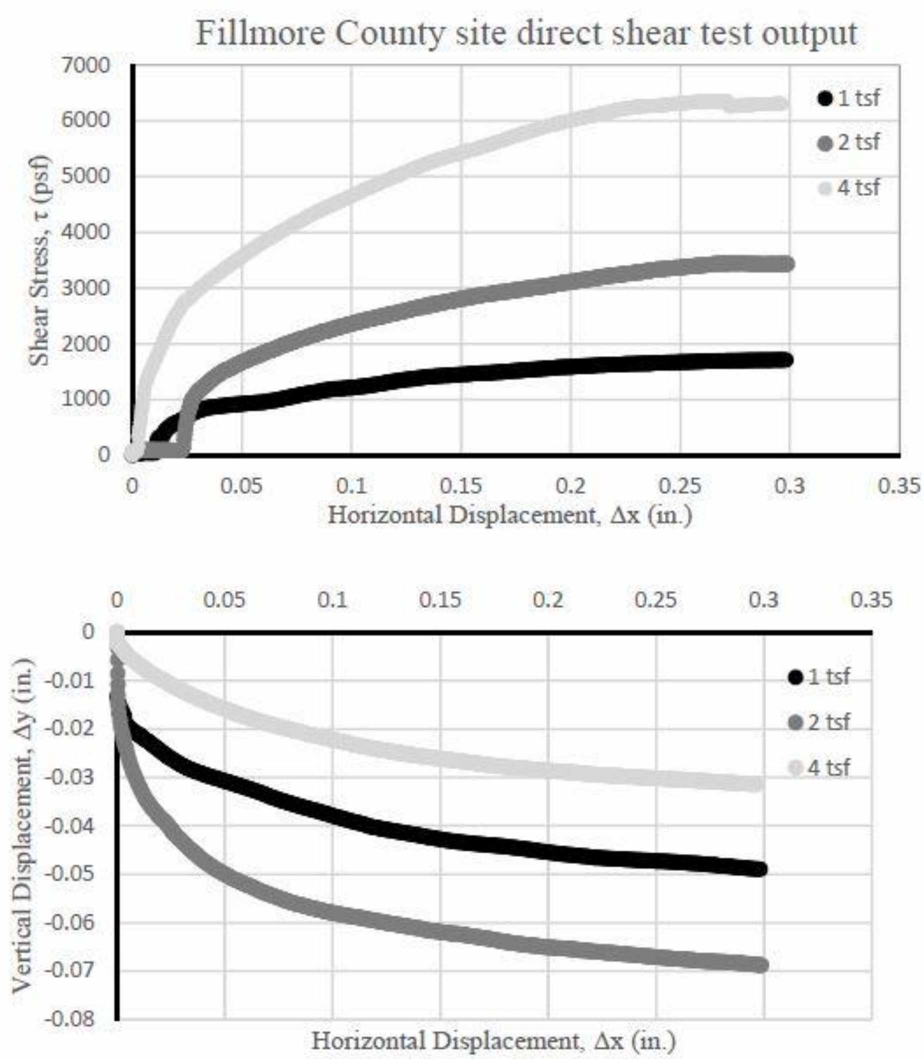
Figure C1: Carlton County site strength characterization data

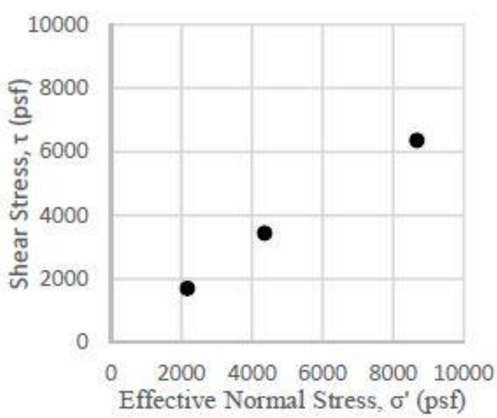




φ' (deg)	35
c' (psf)	200
S_u (tsf)	0.5 to 0.75
DCP blows per ft.	3 to 4

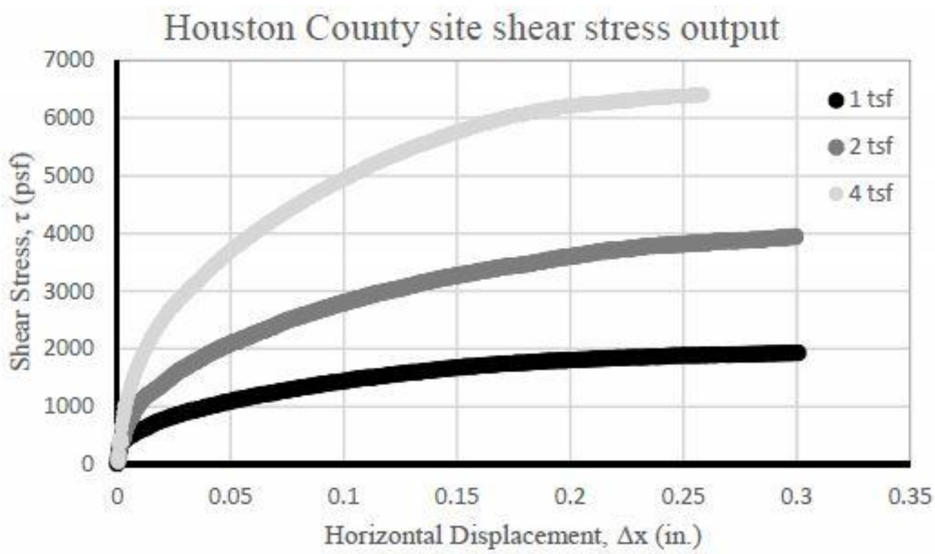
Figure C2: Carver County site strength characterization data

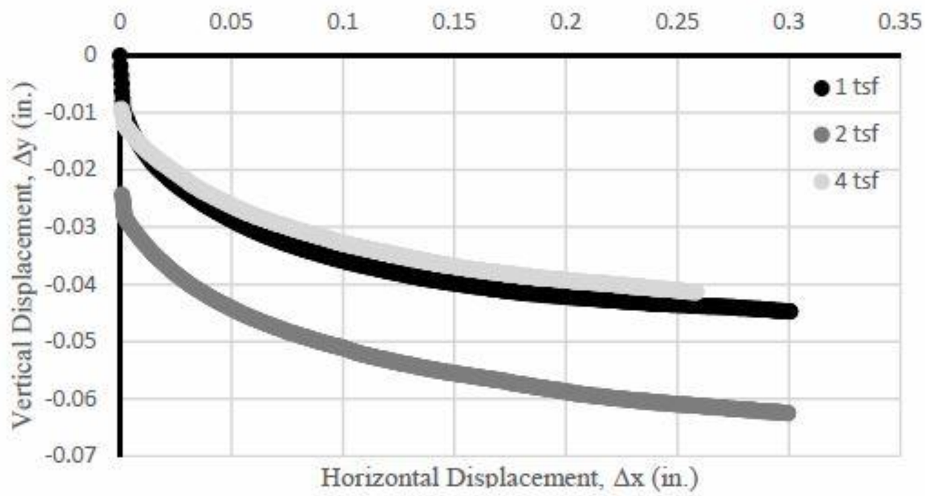


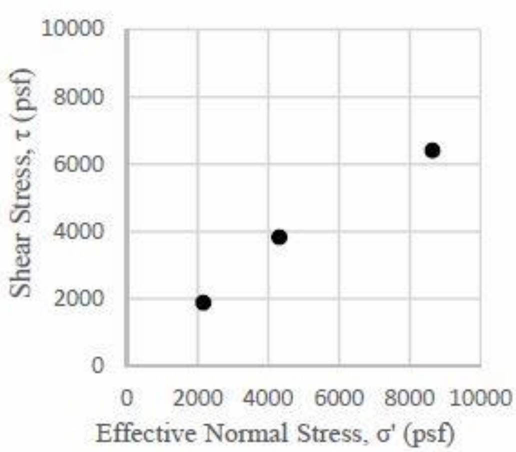


φ' (deg)	35
c' (psf)	150
S_u (tsf)	1.25 to 1.75
DCP blows per ft.	4 to 5

Figure C3: Fillmore County site strength characterization data

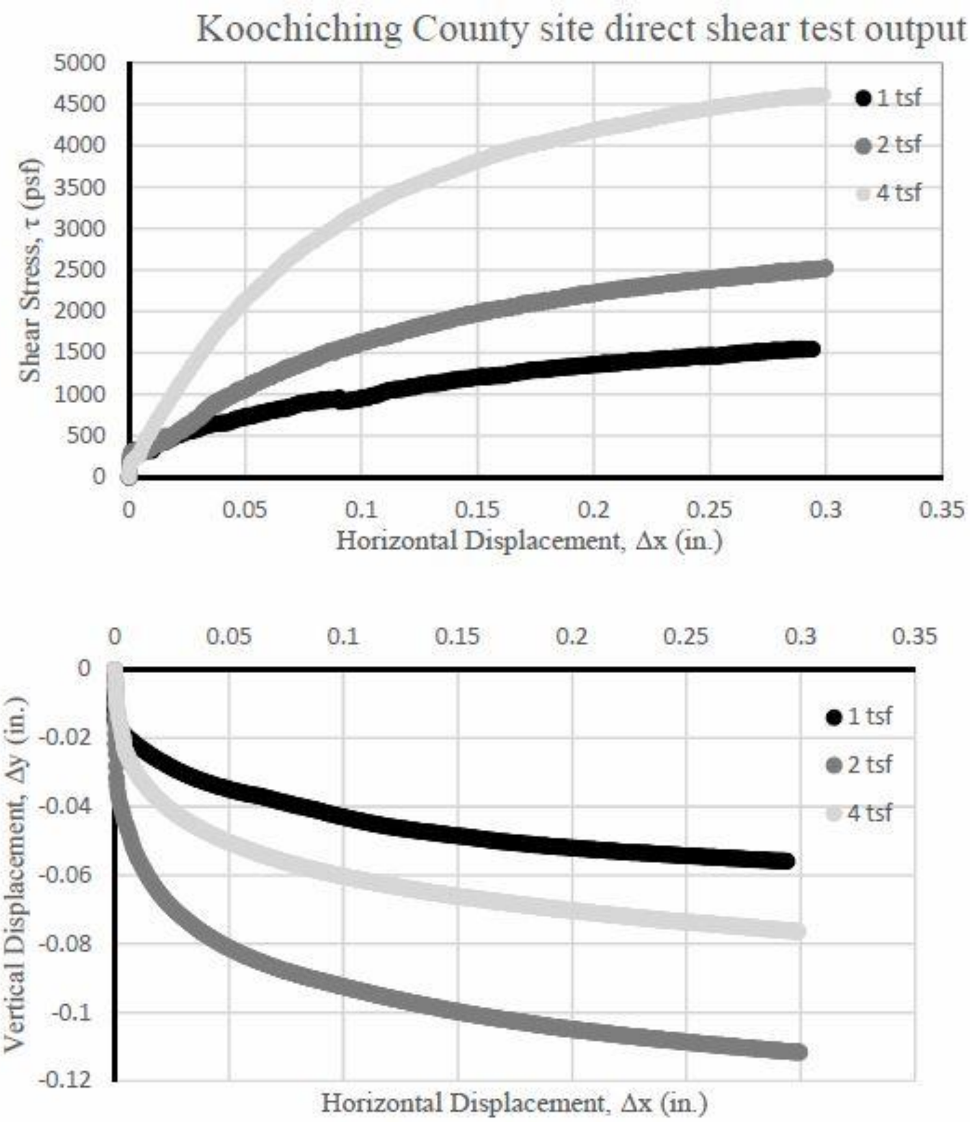


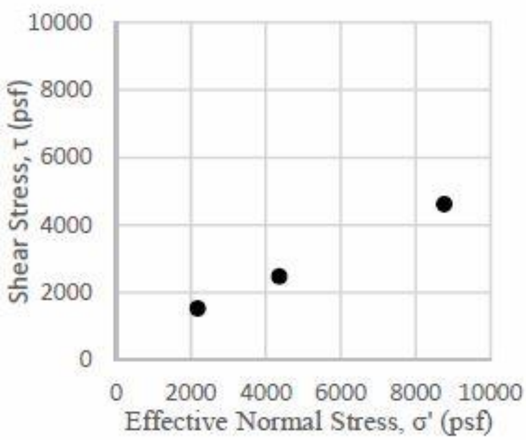




φ' (deg)	34
c' (psf)	300
S_u (tsf)	N/A
DCP blows per ft.	N/A

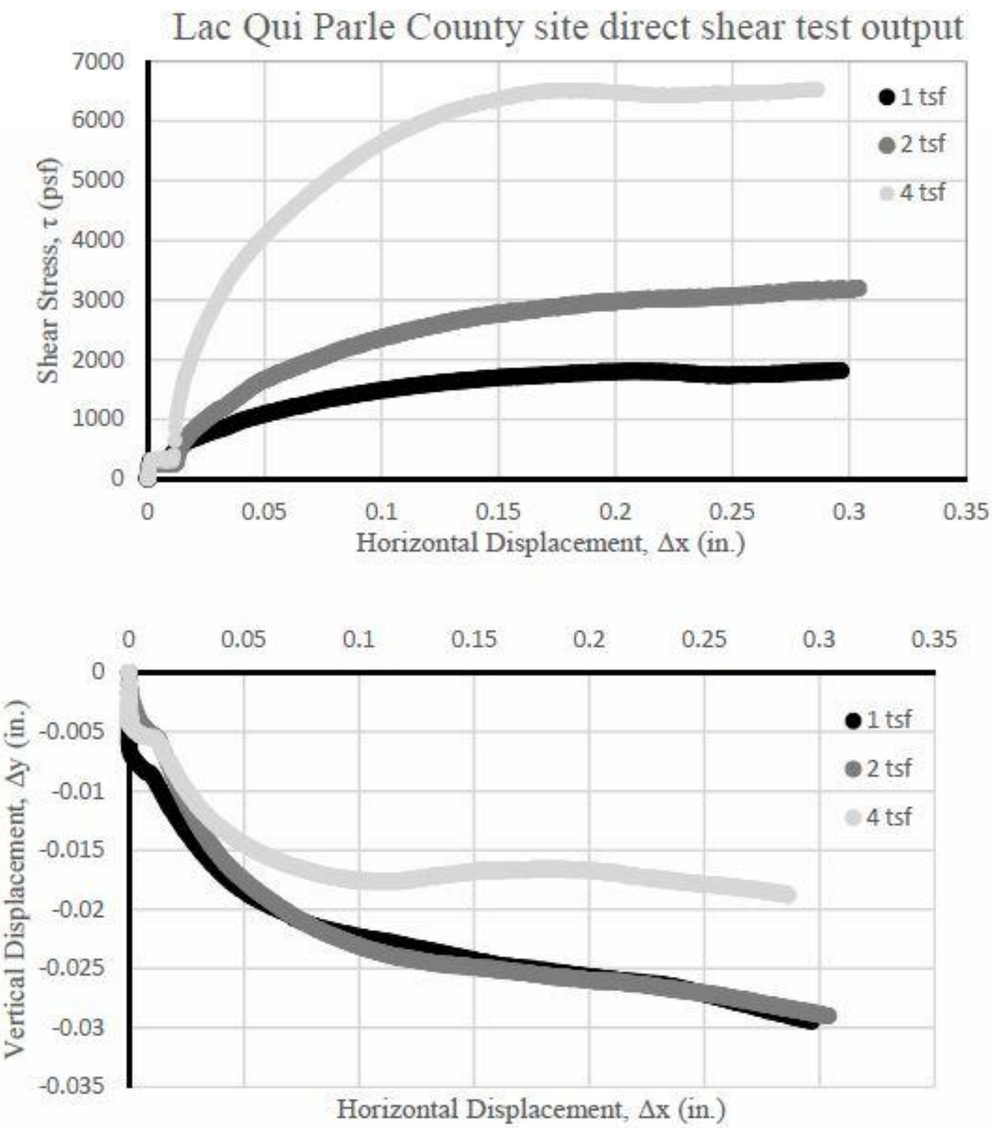
Figure C4: Houston County site strength characterization data

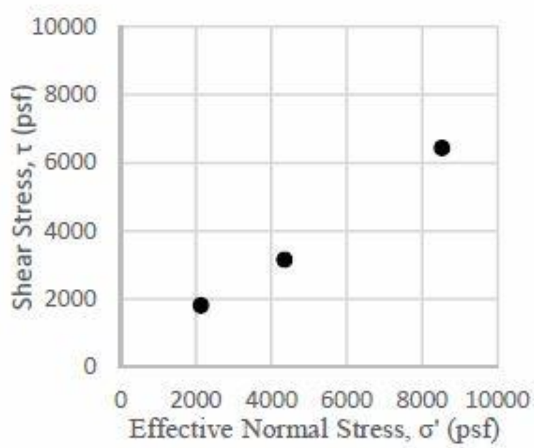




φ' (deg)	24
c' (psf)	400
S_u (tsf)	1 to 1.5
DCP blows per ft.	5

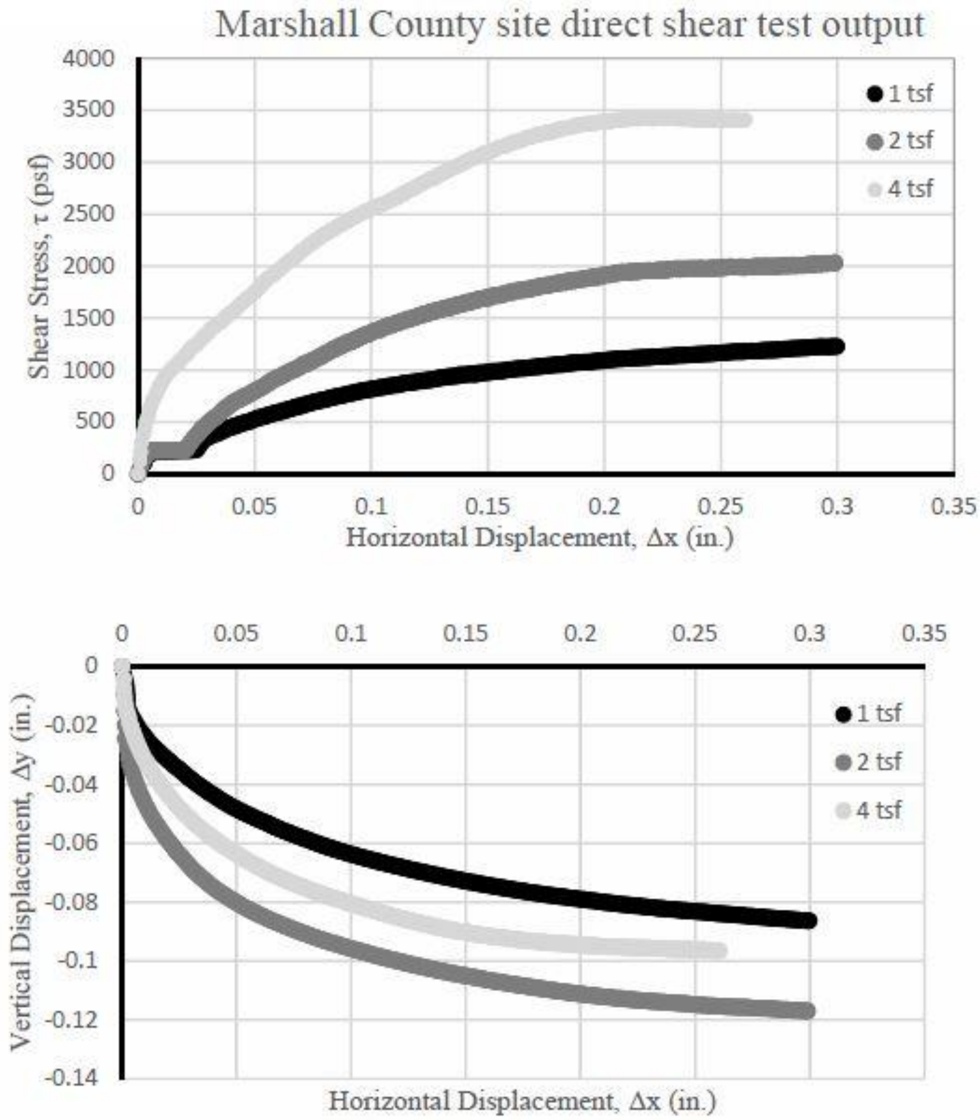
Figure C5: Koochiching County site strength characterization data

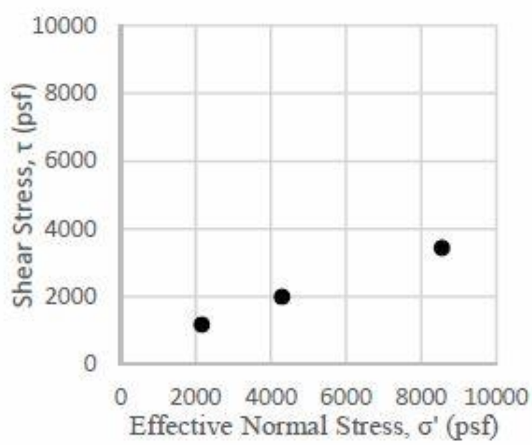




φ' (deg)	35
c' (psf)	50
S_u (tsf)	1.25 to 2
DCP blows per ft.	5 to 7

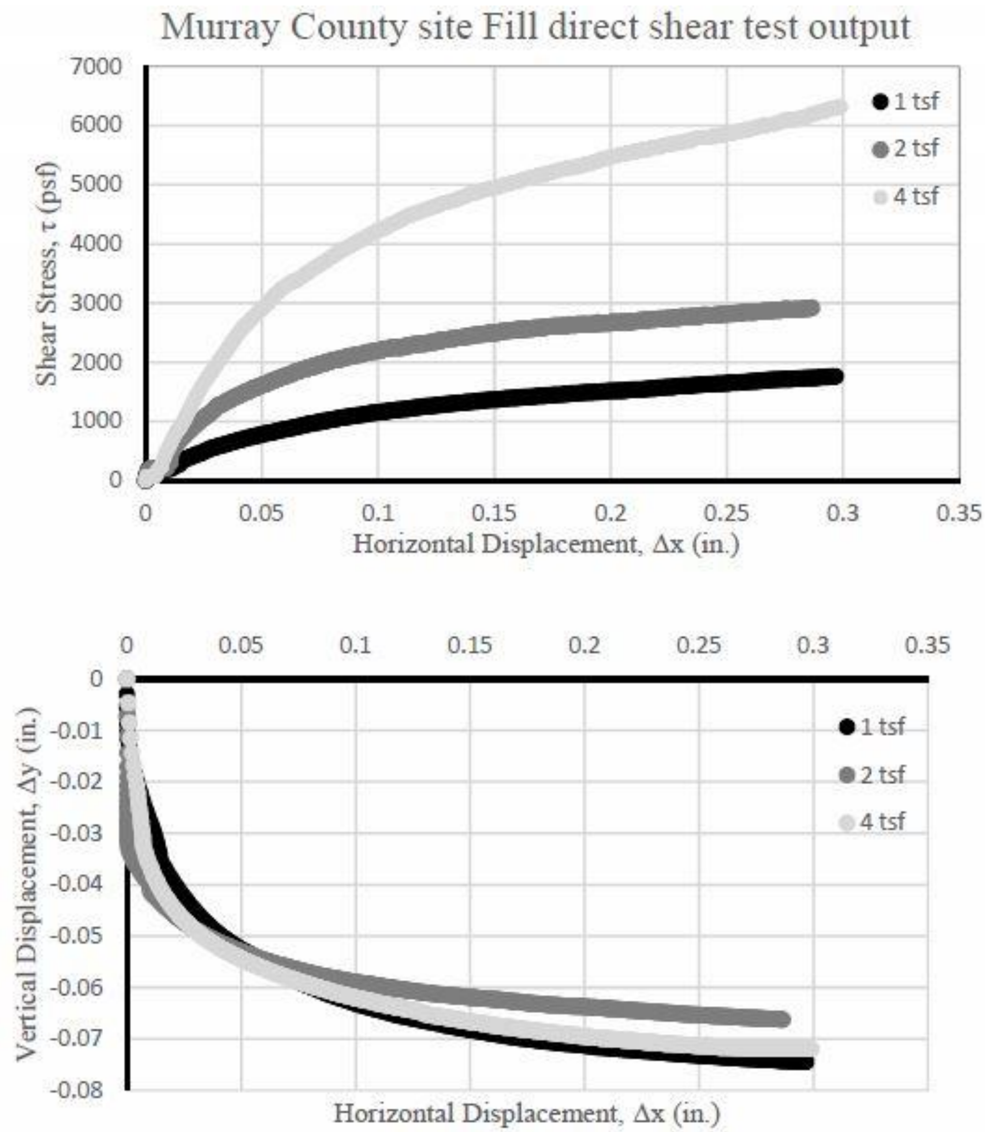
Figure C6: Lac qui Parle County site strength characterization data

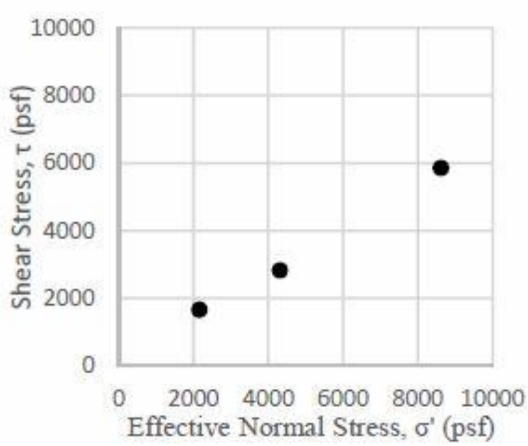




φ' (deg)	18
c' (psf)	600
S_u (tsf)	1.25 to 1.75
DCP blows per ft.	3 to 4

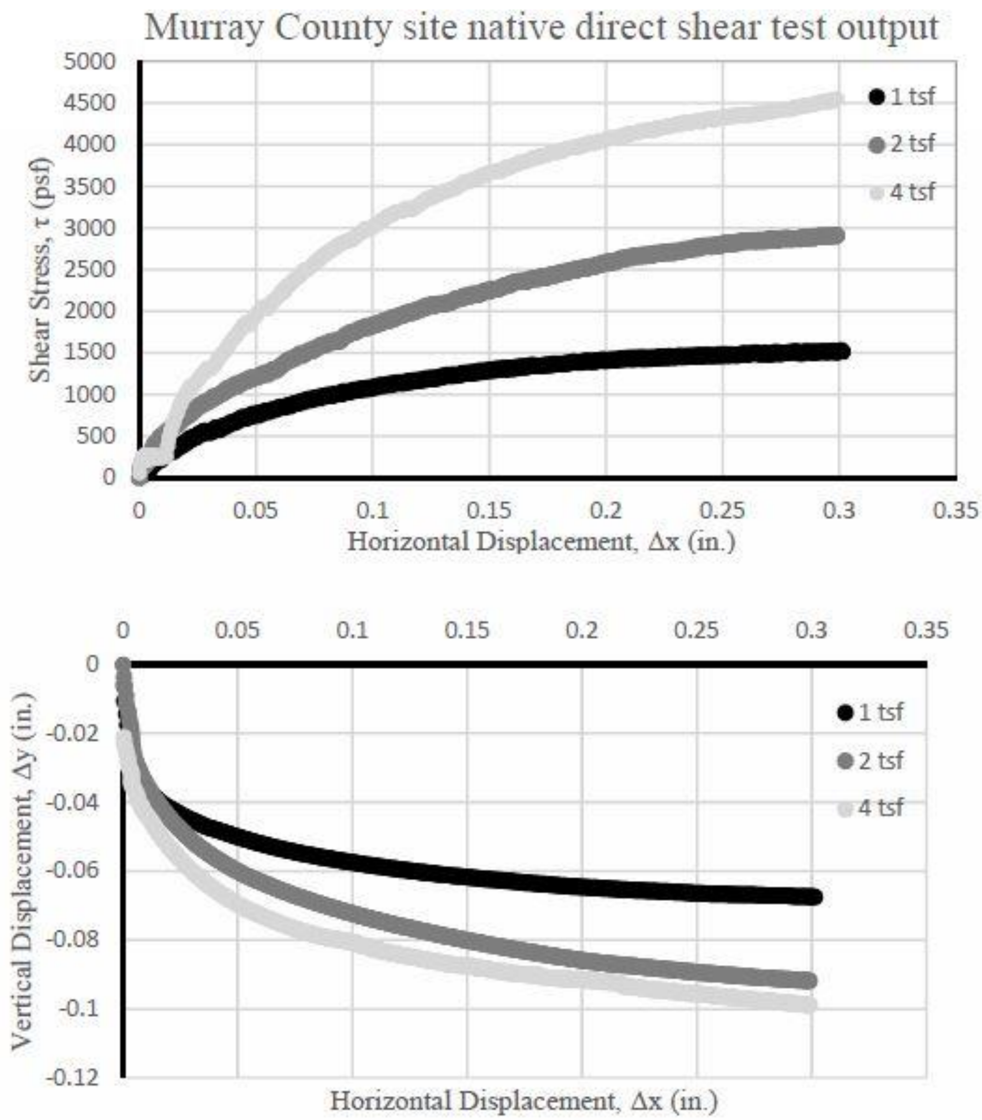
Figure C7: Marshall County site strength characterization data

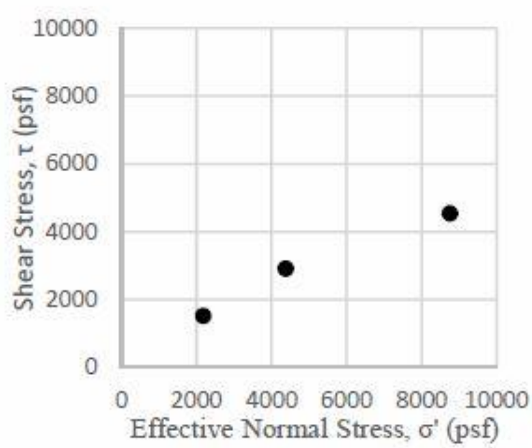




φ' (deg)	32
c' (psf)	390
S_u (tsf)	N/A
DCP blows per ft.	3 to 4

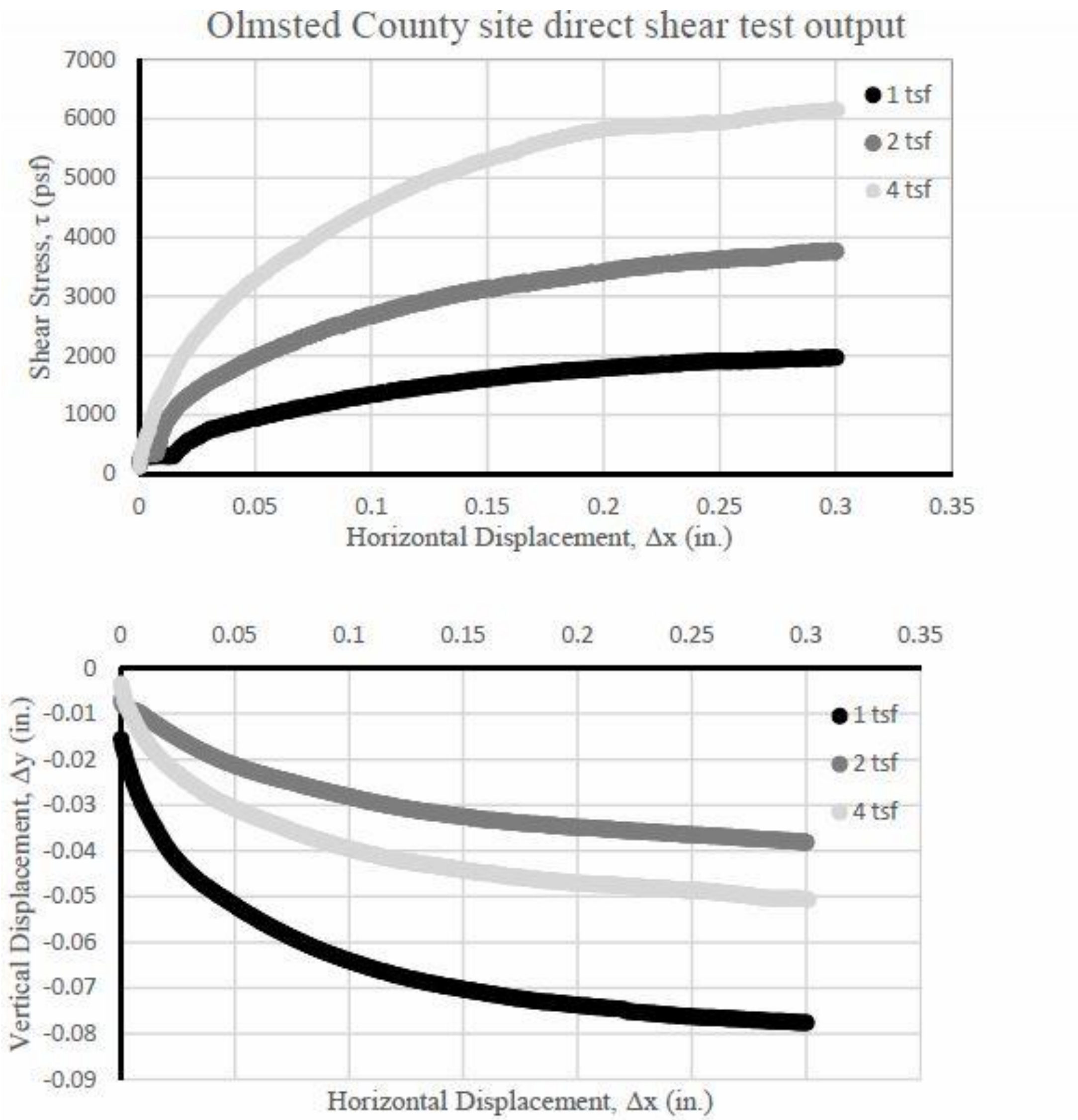
Figure C8: Murray County site fill strength characterization data

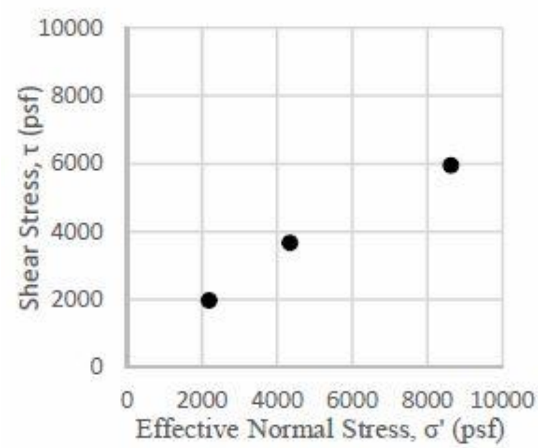




φ' (deg)	22
c' (psf)	900
S_u (tsf)	N/A
DCP blows per ft.	3 to 4

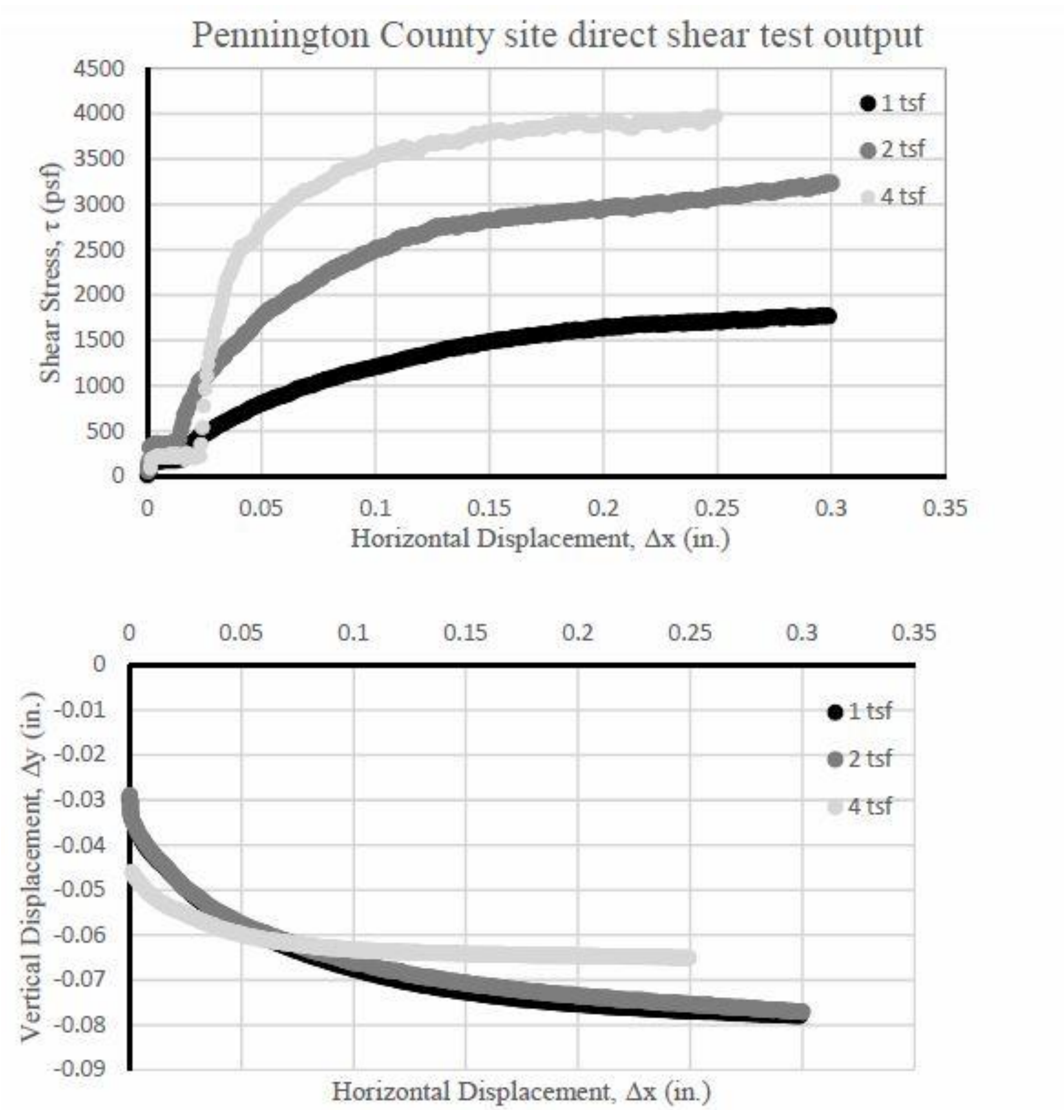
Figure C9: Murray County site native strength characterization data

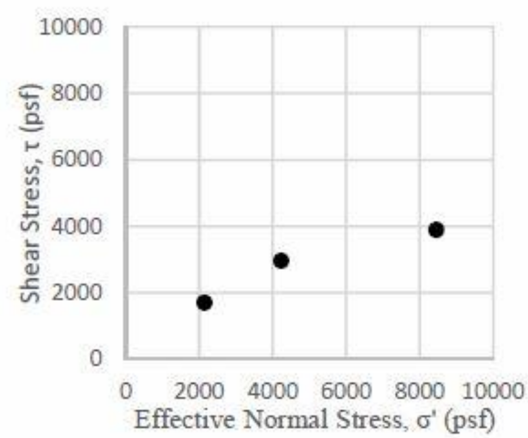




φ' (deg)	34
c' (psf)	200
S_u (tsf)	0.25 to 0.5
DCP blows per ft.	3

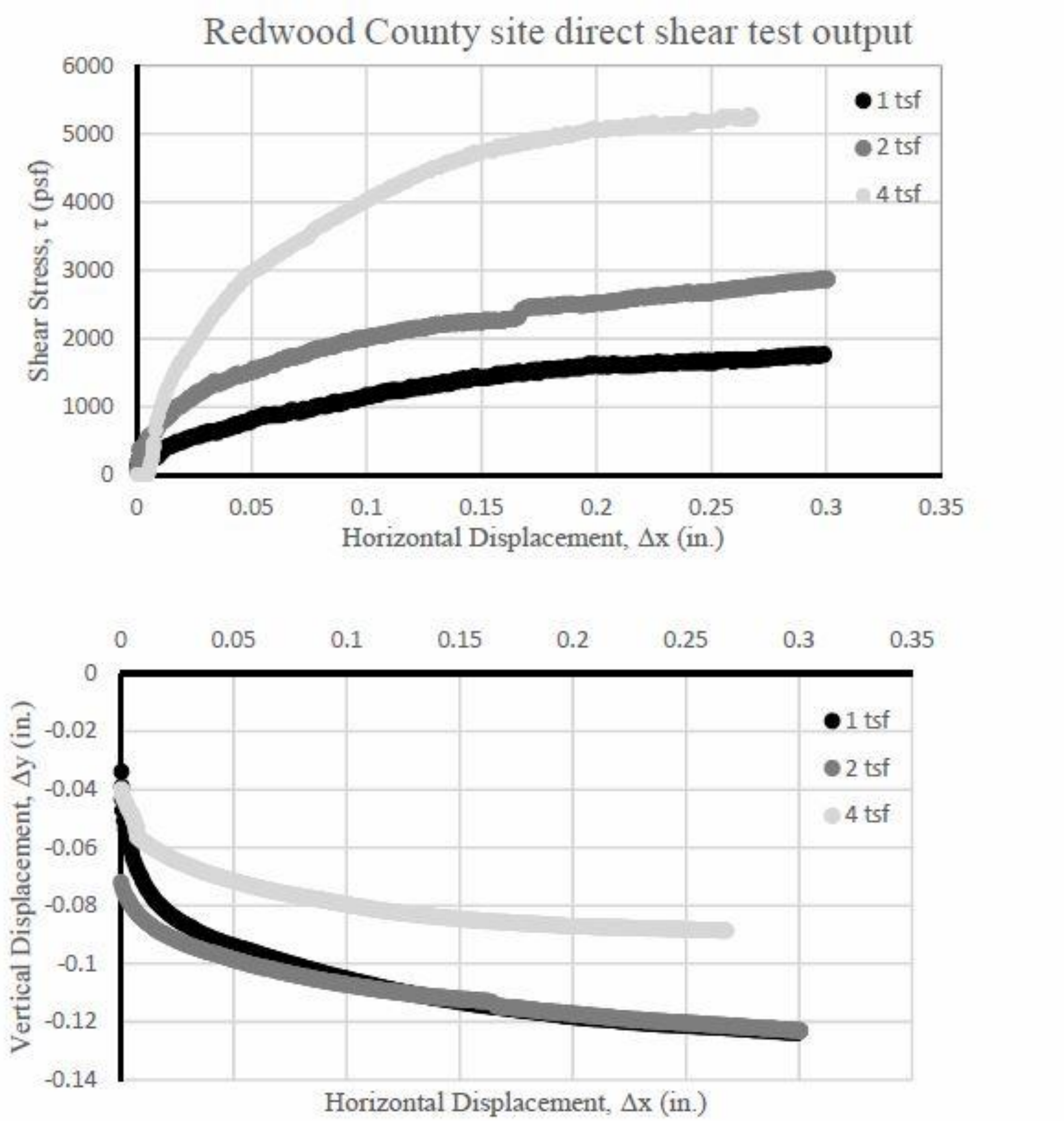
Figure C10: Olmsted County site strength characterization data

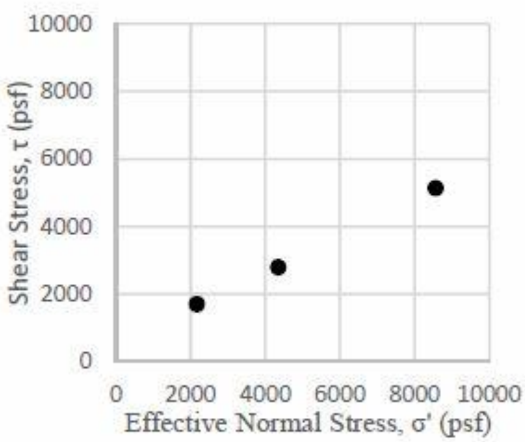




φ' (deg)	17
c' (psf)	1275
S_u (tsf)	0.25 to 0.75
DCP blows per ft.	3 to 4

Figure C11: Pennington County site strength characterization data





φ' (deg)	21
c' (psf)	750
S_u (tsf)	0.5
DCP blows per ft.	4 to 5

Figure C12: Redwood County site strength characterization data

APPENDIX D: SLOPE STABILIZATION GUIDE FOR LOCAL GOVERNMENT ENGINEERS

The final project deliverable, the slope stabilization guide, follows. The tool is intended to provide general recommendations for any given slope stabilization issue that public works engineers would encounter in Minnesota, based on observations from field investigations and LEM modeling results.

INTRODUCTION

This project recommends simple, effective methods of stabilizing at-risk slopes in Minnesota. Slope failures can block roads, pose safety hazards, and introduce preventable maintenance costs. While there is no single stabilization method appropriate for all situations, several methods have proven effective. This project uses slope stability analysis, including Limit Equilibrium Method modeling (LEM), to investigate recent slope failures in Minnesota. This study provides a consistent, logical approach to slope stabilization that is founded in geotechnical research and experience and applies to common slope failures. This project's end users are public works engineers working on slope stabilization projects. The input for analysis came from Minnesota county engineers.

This guide is the product of a Minnesota Department of Transportation research report: *MnDOT Contract No. 99008, Work Order No. 190, Slope Stabilization and Repair Solutions for Local Government Engineers.* Details, background, and complete descriptions are available in the report. Authors recommend referencing the report when using this guide.

SLOPE FAILURE OVERVIEW

Slope stability is quantified by factor of safety (FS). The FS is the ratio of *in situ* shear strength to the shear strength required for equilibrium along a given potential failure surface. Fundamentally, there are two ways to increase the FS and improve slope stability: introduce stabilizing forces (increase capacity) or limit driving forces (decrease demand). Academic research and standard engineering practice have produced many slope stabilization methods; most fit into four categories:

- Limit / manage water in slope material
- Add cover
- Excavate / change slope geometry
- Add support structure

Theoretically, a FS value less than or equal to 1.0 will correspond to slope failure. When slopes fail, visual observation can classify most failures into two types: surficial soil creep or rotational failure. Figure C.1 shows an example of each.

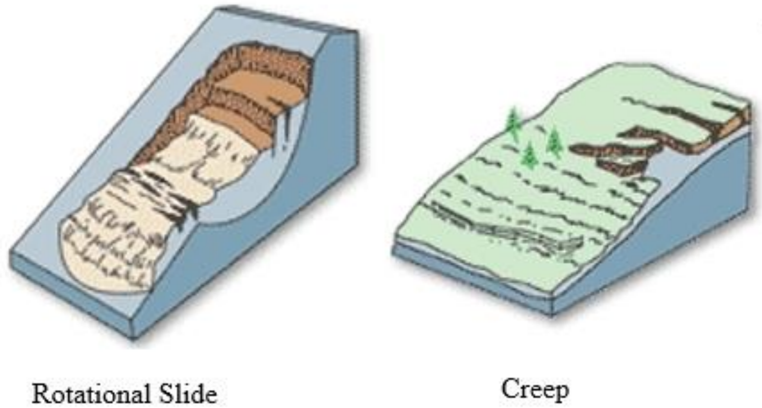


Figure D.1: Examples of common slope failure types (from Varnes, 1978)

Soil type is another important distinction. The two soil types considered are cohesive (i.e. silt and clay) and granular (i.e. sand) soils. Visual inspection may distinguish between the two types, but sometimes laboratory testing is required. Sand is typically less likely to exhibit deep rotational slides. Slopes made of cohesive material will have more drainage concerns, and are usually more susceptible to seasonal frost heave.

Water typically has a negative effect on soil's ability to resist shearing, leading to slope instability. An increase in pore pressure (due to water presence) leads to a decrease in effective stress (σ'). Because σ' governs soil strength and deformation characteristics, the presence of water leads to decreased soil shear strength. Groundwater has a significant effect on shear strength, and removing groundwater provided the greatest difference in output FS for each site. The third major site condition distinction is if poor drainage effects the slope. Drainage is considered poor if groundwater lowers soil strength and leads to failure. Cohesive soils, like clay and silt, typically have poor drainage properties. Examples of sites with poor drainage are shown in the site visit summary section of the project report.

SITE CONDITIONS AND SCENARIO DESCRIPTIONS

This guide lays out common slope failure conditions, and provides geotechnical recommendations for stabilization. Based on field observations and the LEM modeling process, researchers developed common site conditions by considering distinctions in three categories: soil type, slope failure type, and presence of groundwater. This led to eight hypothetical scenarios for researchers to make general recommendations.

A flowchart, shown in Figure D.2, helps users determine which scenario to use. The distinction "poor drainage" is interchangeable with "groundwater concerns." Users start at the center, and follow the flowchart outward.   Table D.1 provides a summary of the scenarios.

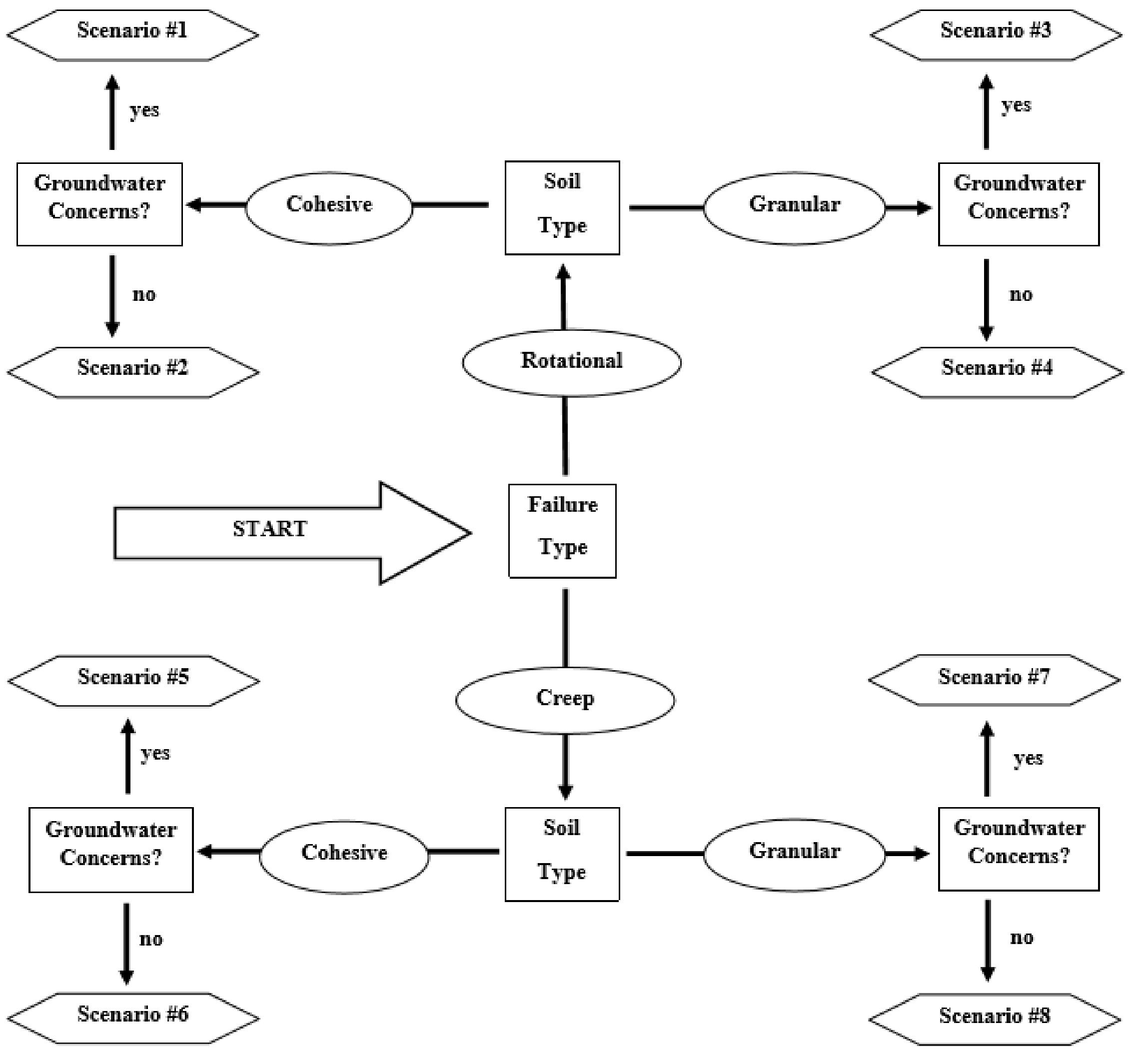


Figure D.2: Flowchart for deliverable scenarios

Table D.1: Deliverable scenarios summary

Name	Failure Type	Soil Type	Groundwater Concerns?
Scenario #1	Rotational Slide	Cohesive	Yes
Scenario #2	Rotational Slide	Cohesive	No
Scenario #3	Rotational Slide	Granular	Yes
Scenario #4	Rotational Slide	Granular	No
Scenario #5	Surficial Creep	Cohesive	Yes
Scenario #6	Surficial Creep	Cohesive	No
Scenario #7	Surficial Creep	Granular	Yes
Scenario #8	Surficial Creep	Granular	No

Table D.2 provides sources for more detailed descriptions of each stabilization method. Users are encouraged to consult sources of background information when considering a stabilization approach.

Table D.2: Sources for more information and examples of stabilization methods

Stabilization Method	Source of Background Information
Drainage pipes, wells, and channels	Cornforth (2005) Ch. 17
Dewatering	Coduto et al. (2011) Ch. 11
Vegetation	Abramson et al. (2002) Ch. 7
Buttressing / rip-rip	Abramson et al. (2002) Ch. 7
Geosynthetics	Gee (2015)
Remove and replace	Duncan and Wright (2005) Ch. 16
Re-grading and benching	Cornforth (2005) Ch. 15
Lightweight fill	Abramson et al. (2002) Ch. 7
Retaining walls	Cornforth (2005) Ch. 19
Soil nails / rock bolts / tieback anchors	Abramson et al. (2002) Ch. 7
Mechanically stabilized earth embankments	Abramson et al. (2002) Ch. 7

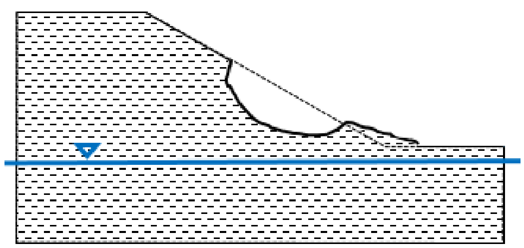
Scenario #1: Rotational failure, cohesive soil, poor drainage

Example of Scenario #1 (from Pennington Co. site)

- Rotational failure
- Cohesive soil
- Groundwater concerns

Recommended stabilization approach:

Remove-and-replace, adding drainage features and vegetative cover



Sites can be identified by visible rotational failure. Maintenance teams should consider either remove-and-replace or regrading with *in situ* soil, adding drainage features, and vegetative cover. Drainage features remove groundwater, and fill-and-regrade work adds stability. Drains should be placed near the toe of the slope. If significant rotational failure has already occurred, the slope will need to be rebuilt. Design teams should consider as low of a slope angle as possible.

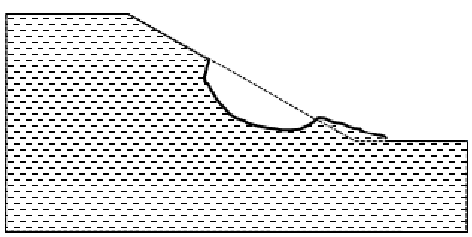
Scenario #2: Rotational failure, cohesive soil

Example of Scenario #2 (from Olmsted Co. site)

- Rotational failure
- Cohesive soil
- No groundwater concerns

Recommended stabilization approach:

Remove-and-replace, or regrade and re-compact, with vegetative cover



Failure can be identified by visual observation. Many factors can cause soil to lose strength other than groundwater effects, such as poor compaction. Regrading and re-compacting, when properly executed, increases soil strength and slope stability. Maintenance teams should evaluate the *in situ* soil properties, and either re-use the material, or use common borrow if native material has poor properties.

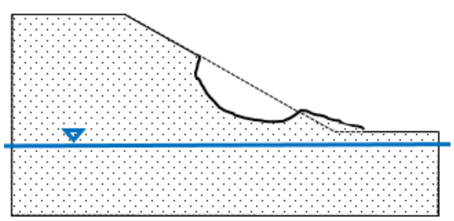
Scenario #3: Rotational failure, granular soil, poor drainage

Rotational failure in sand, similar to Scenario #3

- Rotational failure
- Granular soil
- Groundwater concerns

Recommended stabilization approach:

Remove-and-replace, or re-grade and re-compact, adding drainage features, and adequate surface cover



Surface cover is important for slopes with granular soil because erosion is a concern. Surface erosion can cause geometric inconsistencies lead to failure. Erosion can often cause washout failure. As with other rotational failures, excavation and reconstruction is necessary. Because groundwater is a concern, drainage features are recommended to remove groundwater in the slope. Researchers recommend regrading or, if necessary, replacement with sand fill.

Scenario #4: Rotational failure, granular soil

Example of Scenario #4 (from Lac Qui Parle Co. site)

- Rotational failure
- Granular soil
- No groundwater concerns

Recommended stabilization approach:

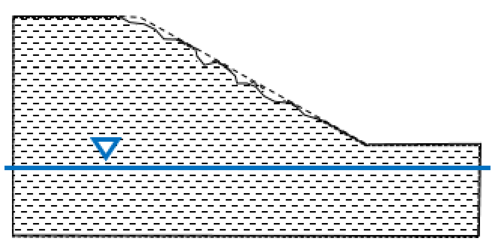
Regrade and re-compact, with vegetative cover or more involved surface cover

Because groundwater is not the primary reason for failure, the main source of strength loss must be identified and mitigated. If erosion is evident, a more involved cover (i.e. rip rap or gravel) should be considered. Slope steepness may also be a concern. Researchers recommend regrading and compacting with *in situ* material. Extra consideration should be given to adequate ground cover to protect the slope from erosion damage.

Scenario #5: Creep failure, cohesive soil, poor drainage

Example of Scenario #5 (from Koochiching Co. site)

- Creep failure
- Cohesive soil
- Groundwater concerns



Recommended stabilization approach:

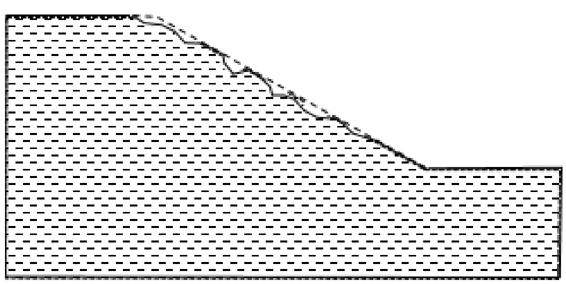
Regrade and re-compact, with drainage features; if one area of failure, remove and replace.

A given site is more likely to have drainage concerns if cohesive material is present. Failure can be identified by crooked signs or trees, and leads to pavement damage. With groundwater present, and *in situ* material being frost-susceptible cohesive soil, frost heave is a possible cause of soil movement. Drainage features are the research team's main recommendation for slope stabilization. If creep is at the top of the slope, maintenance crews can also consider replacing the top portion of the slope with free-draining sand. If the failure is near the bottom of the slope, a buttress can be an effective stabilization method.

Scenario #6: Creep failure, cohesive soil

Example of Scenario #6 (from Murray Co. site)

- Creep failure
- Cohesive soil
- No groundwater concerns



Recommended stabilization approach:

Remove, replace, and re-compact

Surface creep can be identified by bio-indicators like bent trees. The example clearly shows how soil creep at the top of a slope can lead to pavement damage. Replacing the failed portion of the slope with sand fill is the recommended option for increasing sliding resistance. In the absence of groundwater, poor compaction decreases the soil's shear strength. If *in situ* soil has adequate strength properties, regrading and re-compaction can be considered, but creep failure indicates concerns about strength of native material.

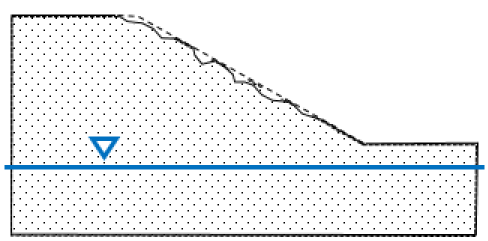
Scenario #7: Creep failure, granular soil, poor drainage

Example of Scenario #7 (from Carver Co. site)

- Creep failure
- Granular soil
- Groundwater concerns

Recommended stabilization approach:

Remove-and-replace, or re-grade and re-compact, adding drainage features, and adequate surface cover



Adequate ground cover is essential to prevent erosion in slopes with sand. Bent guardrails are evidence of soil creep, which typically causes pavement damage. Proper drainage can remove groundwater from the area, increasing resistance to soil creep. Researchers recommend installing drainage features, and replacing failed soil with properly-compacted fill, or re-compacting *in situ* material. Slope material should be protected from erosion.

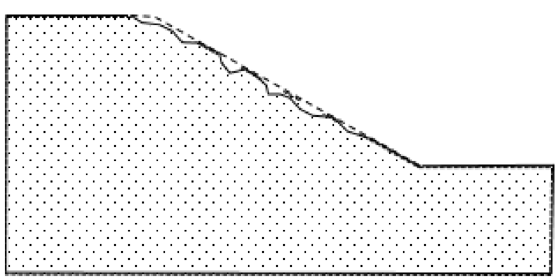
Scenario #8: Creep failure, granular soil


Soil creep in sand, similar to Scenario #8

- Creep failure
- Granular soil
- Groundwater concerns

Recommended stabilization approach:

Remove-and-replace, or re-grade and re-compact, with adequate surface cover



For granular soils, erosion is a concern. Surficial damage caused by erosion is not always soil creep, but the movement type and stabilization attempts are similar. Surface washout can undermine roadways and cause pavement damage. Ensuring adequate ground cover is important when observing surficial damage in slopes with granular fill. Damage at the top of the slope is best repaired by regrading.

APPENDIX E: SLOPE STABILITY CHARTS

The author developed slope stability charts for local government engineers. Inputs are representative of typical roadway embankment parameters for locally maintained slopes in Minnesota. Slope stability charts for factor of safety based on Bishop Method analysis, circular failure geometry parameters, and curve fitting regression analysis are presented in this Appendix.

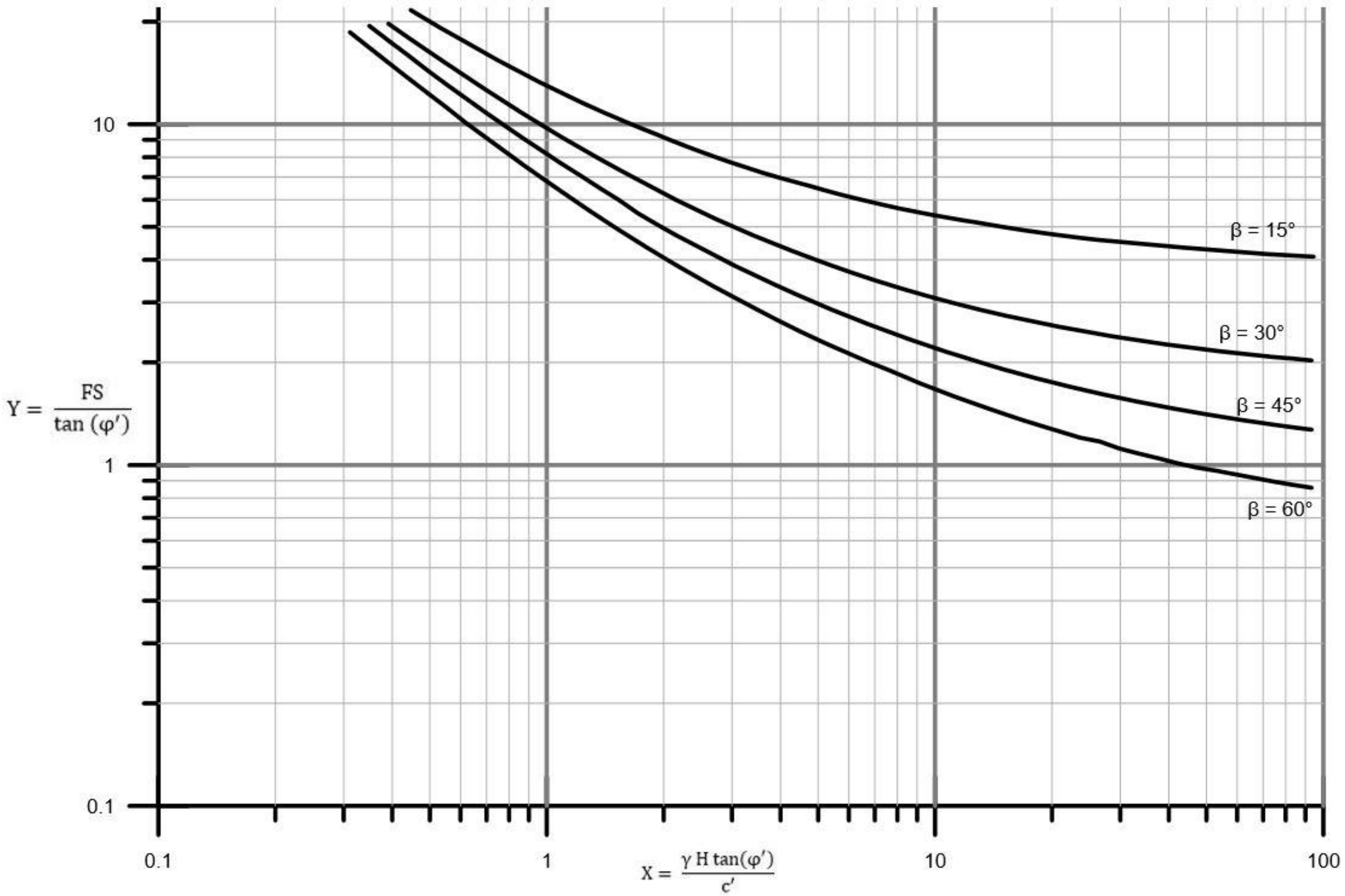


Figure E1: Slope stability chart for factor of safety

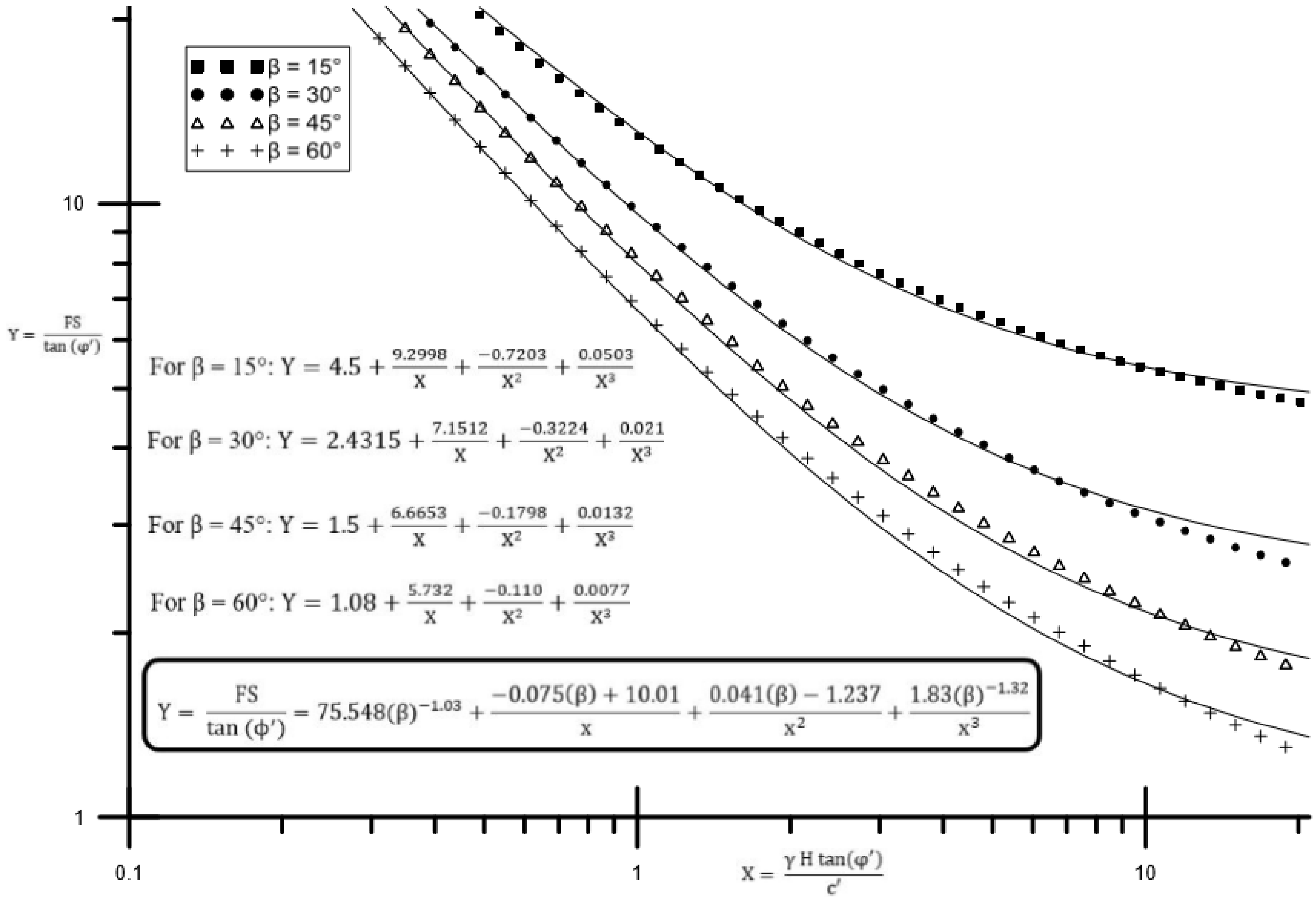


Figure E2: Curve fitting and regression analysis for stability chart

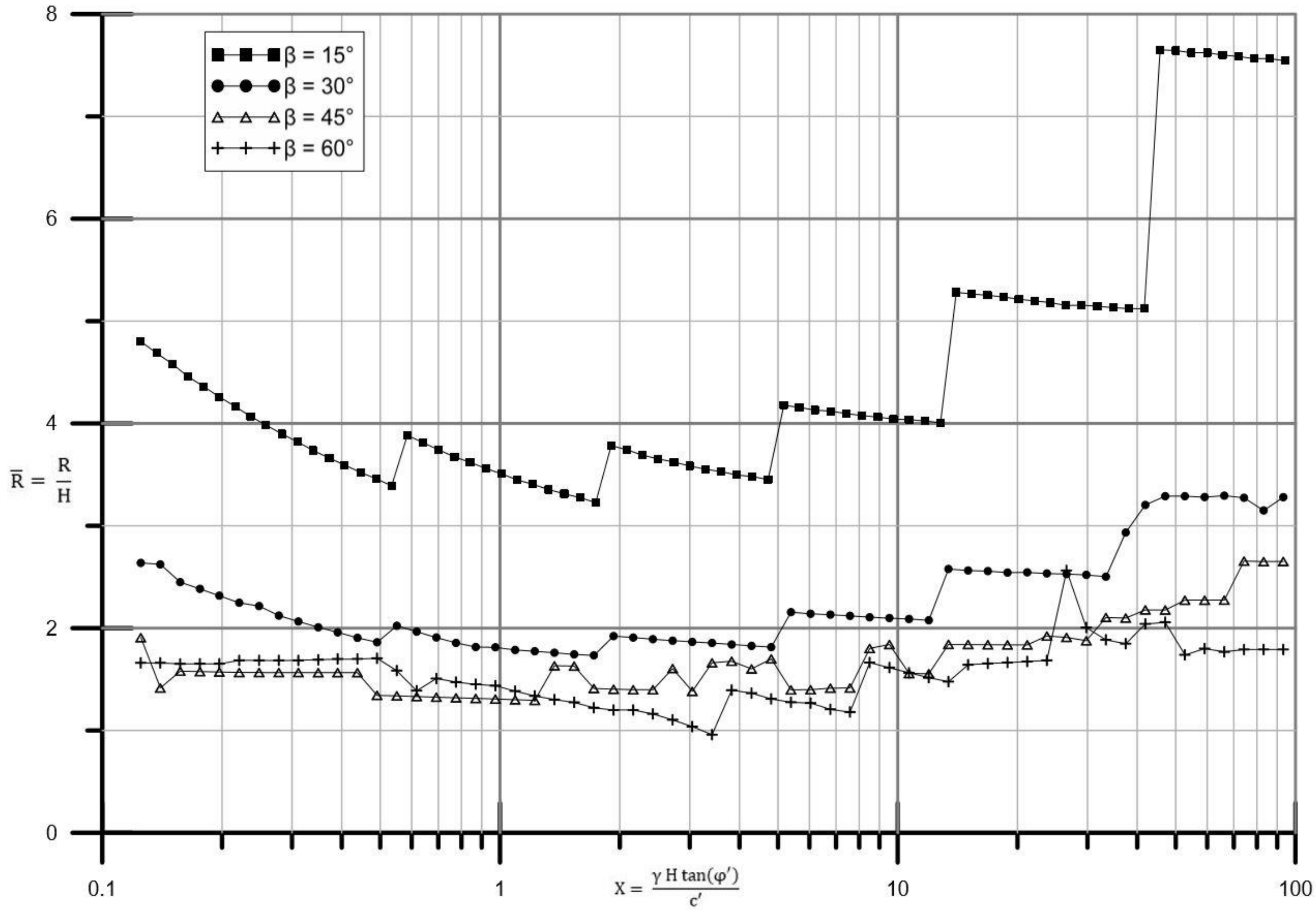


Figure E3: Slope stability chart for normalized circular failure radius, $\overline{R}$

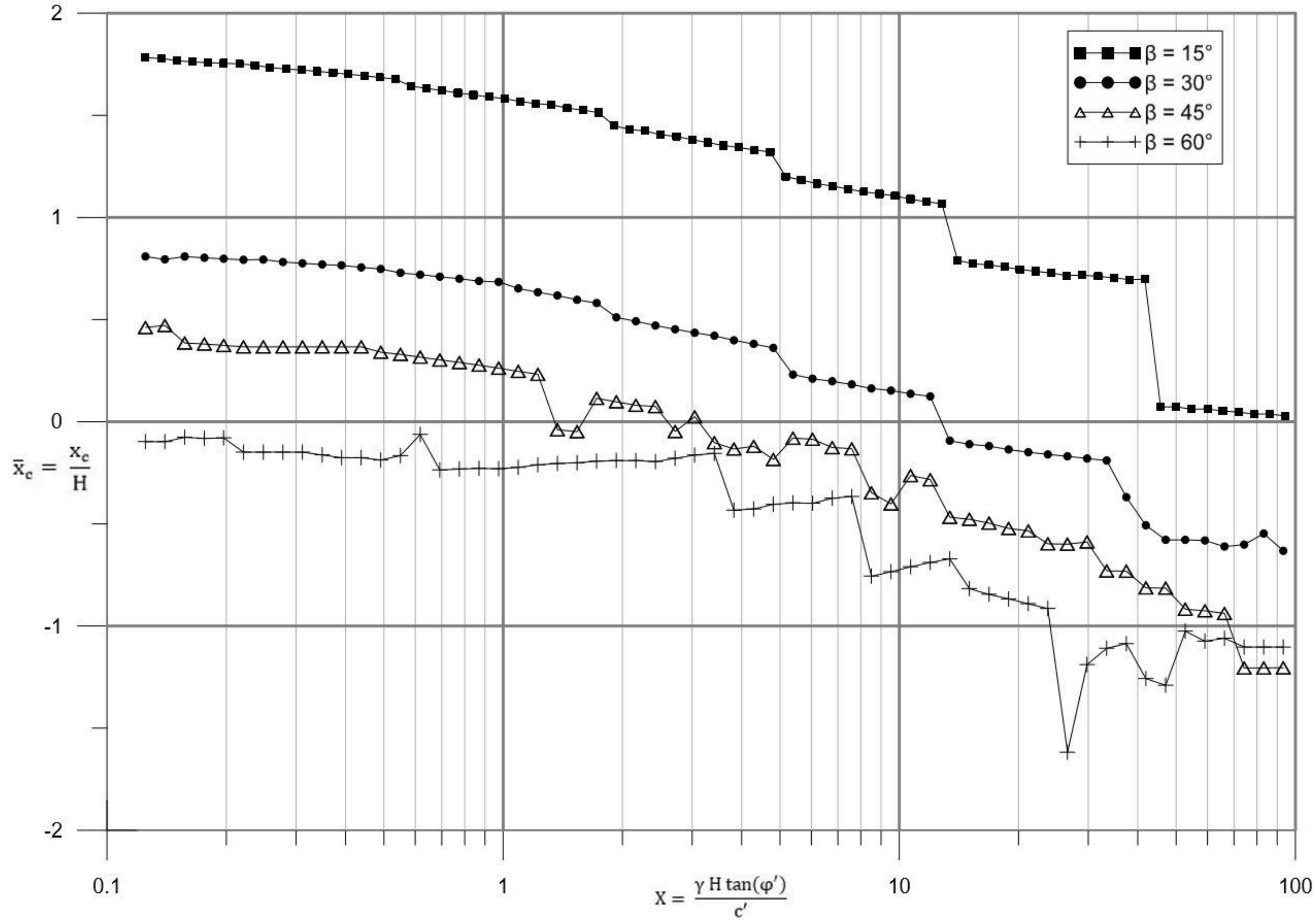


Figure E4: Slope stability chart for normalized x_c, $\bar{x}_c$

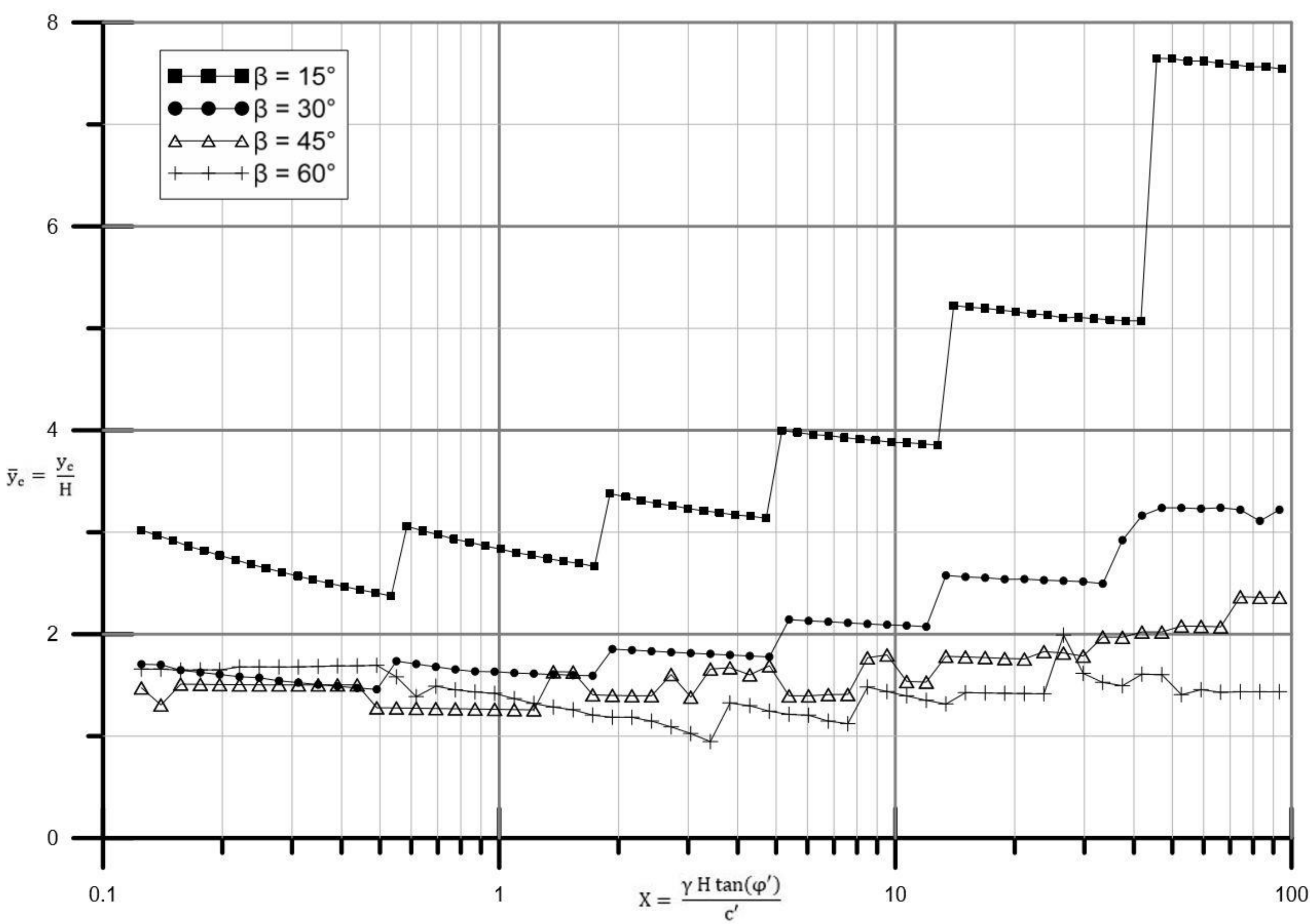


Figure E5: Slope stability chart for normalized y_c, $\bar{y}_c$